ARAB REPUBLIC OF EGYPT

Ministry of Housing, Utilities
and Urban Development

Housing and Building National Research Center

EGYPTIAN CODE OF PRACTICE FOR STEEL CONSTRUCTION (LOAD AND RESISTANCE FACTOR DESIGN) (LRFD)

(205)
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Permanent Committee for the Code of Practice
for Steel Construction and Bridges

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جمهورية مصر العربية
وزارة
الإسكان والمرافق والتنمية العمرانية
مكتب الوزير
الرقم البريدي ١١٥١٦

قرار وزاري
رقم (٣٥٩) لسنة ٢٠٠٧

وزير الإسكان والمرافق والتنمية العمرانية:

- بعد الإطلاع على القانون رقم ٦ لسنة ١٩٦٤ بشأن أسس التصميم وشروط تنفيذ الأعمال الإنشائية وأعمال البناء.
- وعلى القرار الوزاري رقم ٤٩٢ لسنة ١٩٩٦ بتشكيل اللجنة الرئيسية لأسس تصميم وشروط تنفيذ الأعمال الإنشائية وأعمال البناء .
- وعلى القرار الجمهوري رقم ٦٣ لسنة ٢٠٠٥ في شأن إعادة تنظيم المركز القومي لبحوث الاسكان والبناء.
- وعلى القرار الجمهوري رقم ٥٢ لسنة ٢٠٠٦ .
- وعلى ما ارتأته اللجنة الدائمة لإعداد أسس التصميم وإشتراطات تنفيذ المنشآت المعدنية المشكلة بالقرار الوزاري رقم ١٦١ لسنة ١٩٩٧ .
- وعلى مذكرة المركز القومي لبحوث الإسكان والبناء بتاريخ ٢٠٠٧/٩/٩ .

قــــرر

مادة (١) : يتم العمل بالكود المصري لأسس تصميم واشتراطات تنفيذ المنشآت المعدنية على أساس الأحمال والمقاومة المعيارية بجانب الكود المصري لأسس تصميم وشروط تنفيذ المنشآت والكباري المعدنية الصادر بالقرار الوزاري رقم ٢٧٩ لسنة ٢٠٠١ على أساس إجهادات التشغيل.

مادة (٢) : تتولى اللجنة الدائمة لإعداد كود أسس التصميم وشروط تنفيذ المنشآت والكباري المعدنية لاقتراح التعديلات والإضافات التي تراها لازمة بغرض التحديث كلما دعت الحاجة لذلك وتعتبر التعديلات والإضافة بعد إصدارها جزءاً لا يتجزأ من الكود.

مادة (٣) : يتولى المركز القومي لبحوث الإسكان والبناء العمل على نشر الكود والتعريف به والتدريب عليه.

مادة (٤) : تلتزم الجهات المعنية والمذكورة في القانون رقم ٦ لسنة ١٩٦٤ بتنفيذ ما جاء بهذا الكود.

مادة (٥) : ينشر هذا القرار في الوقائع المصرية، ويعمل به اعتباراً من اليوم التالي لمضي ستة أشهر من تاريخ نشره .

وزير الإسكان والمرافق
والتنمية العمرانية
مهندس /
أحمد المغربي

صدر في ٢٠٠٧/٩/٢٠

SYMBOLS

A	Cross-sectional area, cm^2
a	Distance, cm
A_1	Area of steel bearing concentrically on a concrete support, cm^2
A_2	Total cross-sectional area of a concrete support, cm^2
A_B	Loaded area of concrete, cm^2
A_b	Nominal body area of a fastener, cm^2
A_c	Area of concrete, cm^2
A_D	Area of an upset rod based on the diameter of its threads, cm^2
A_e	Effective net area, cm^2
A_f	Area of flange, cm^2
A_{fe}	Effective tension flange area, cm^2
A_{fg}	Gross area of flange, cm^2
A_{fn}	Net area of flange, cm^2
A_g	Gross area, cm^2
A_{gt}	Gross area subject to tension, cm^2
A_{gv}	Gross area subject to shear, cm^2
A_n	Net area, cm^2
A_{nt}	Net area subject to tension, cm^2
A_{nv}	Net area subject to shear, cm^2
A_{pb}	Projected bearing area, cm^2
a_r	Ratio of web area to compression flange area
A_r	Area of reinforcing bars, cm^2
A_s	Area of steel cross section, cm^2
A_{sc}	Cross-sectional area of stud shear connector, cm^2
A_{sf}	Shear area on the failure path, cm^2
A_w	Web area, cm^2
B	Factor for bending stress in tees and double angles Distance, cm
b	Compression element width, Distance, cm
$\bar{b}$	Appropriate width
B_1, B_2	Factors used in determining M_u for combined bending and axial forces
b_e	Reduced effective width for slender compression elements, cm
b_{eff}	Effective edge distance, cm

b_f	Flange width, cm
c	Distance, cm
C'_m, C_m	Coefficients applied to bending term in interaction formula
c_1, c_2, c_3	Numerical coefficients
C_b	Bending coefficient dependent on moment gradient
C_p	Ponding flexibility coefficient for primary member in a flat roof
C_{PG}	Plate-girder coefficient
C_s	Ponding flexibility coefficient for secondary member in a flat roof
C_v	Ratio of "critical" web stress, according to linear buckling theory, to the shear yield stress of web material
C_w	Warping constant, cm^6
D	Diameter - Dead load – Factor - Depth
d	Pin diameter - Nominal fastener diameter - Diameter
d_1, d_2	Distances, cm
d_b	Beam depth, cm
d_c	Column depth, cm
d_L	Depth at larger end of unbarred tapered segment, cm
d_o	Depth at smaller end of unbarred tapered segment, cm
E	Modulus of elasticity of steel (**E** = 2100 t/cm^2)
e	Base of natural logarithm = 2.71828...
E_c	Modulus of elasticity of concrete, t/cm^2
E_m	Modified modulus of elasticity, t/cm^2
EQ	Earthquake load
f	Computed compressive stress in the stiffened element, t/cm^2
F_{BM}	Nominal strength of the base material to be welded, t/cm^2
f'_c	Specified compressive strength of concrete, kg/cm^2
F_{cr}	Critical buckling stress, t/cm^2
F_{crft}, F_{cry}, F_{crz}	Flexural-torsional buckling stresses for double-angle and tee-shaped compression members, t/cm^2
F_e	Elastic buckling stress, t/cm^2
F_{ex}	Elastic flexural buckling stress about the major axis, t/cm^2
FE_{XX}	Classification number of weld metal
F_{ey}	Elastic flexural buckling stress about the minor axis, t/cm^2
F_{ez}	Elastic torsional buckling stress, t/cm^2
F_L	Stress portion defined in section 5.1.3
F_{my}	Modified yield stress for composite columns, t/cm^2

F_n	Nominal shear rupture strength, t/cm^2
F_r	Compressive residual stress in flange, t/cm^2
F_u	Specified minimum tensile strength, t/cm^2
f_{un}	Required normal stress, t/cm^2
f_{uv}	Required shear stress, t/cm^2
f_v	Required shear stress due to factored loads in bolts, t/cm^2
F_w	Nominal strength of the weld electrode material, t/cm^2
F_y	Specified minimum yield stress, t/cm^2
F_{yf}	Specified minimum yield stress of the flange, t/cm^2
F_{yr}	Specified minimum yield stress of reinforcing bars, t/cm^2
F_{yst}	Specified minimum yield stress of the stiffener material, t/cm^2
F_{yw}	Specified minimum yield stress of the web, t/cm^2
G	Shear modulus of elasticity for steel, t/cm^2
	Relative rigidity Factor
g	Transverse center-to-center spacing (gauge) between fastener gauge lines, cm
H	Horizontal force, t – Flexural constant
h	Distance, Depth of unequal angle, cm
h_c	Distance, cm
h_r	Nominal rib height, cm
H_s	Length of stud connector after welding, cm
I	Moment of inertia, cm^4
I_d	Moment of inertia of the steel deck, cm^4 per m
I_p	Moment of inertia of primary members, cm^4
I_s	Moment of inertia of secondary members, cm^4
I_{st}	Moment of inertia of a transverse stiffener, cm^4
I_{yc}	Moment of inertia about y axis referred to compression flange, cm^4
J	Torsional constant for a section, cm^4
j	Factor for minimum moment of inertia for a transverse stiffener
k	Distance from outer face of flange to web toe of fillet, cm
K	Effective length factor for prismatic member, Buckling length factor
K_σ	Plate buckling factor
k_v	Web plate buckling coefficient
K_γ	Effective length factor for a tapered member

K_z	Effective length factor for Torsional buckling
L	Story height - Live load - Length, Unsupported length
ℓ , L_b, L_c	Lengths, cm
ℓ_b	Effective buckling length of compression chord
L_e	Edge distance, cm
L_p	Limiting laterally unbraced length for full plastic bending capacity, uniform moment case (**C_b = 1.0**), cm
	Column spacing in direction of girder, m
L_{pd}	Limiting laterally unbraced length for plastic analysis, cm
L_r	Limiting laterally unbraced length for inelastic lateral-torsional buckling, cm
	Roof live load
L_s	Column spacing perpendicular to direction of girder, m
M	Ratio of web to flange yield stress or critical stress in hybrid beams
M_{nx}, M_{ny}	Flexural strength for use in alternate interaction equations for combined bending and axial force, t.m
M'_p	Moment, for use in alternate interaction equations for combined bending and axial force, t.m
M_1	Smaller moment at end of unbraced length of beam or beam-column, t.m
M_2	Larger moment at end of unbraced length of beam or beam-column, t.m
M_a	Applied torque, t.cm
	Absolute value of moment at quarter point of the unbraced beam segment, t.cm
M_b	Absolute value of moment at centerline of the unbraced beam segment, t.cm
M_c	Absolute value of moment at three-quarter point of the unbraced beam segment, t.cm
M_{cr}	Elastic buckling moment, t.m
M_{lt}	Required flexural strength in member due to lateral frame translation only, t.m
M_{max}	Absolute value of maximum moment in the unbraced beam segment, t.m
M_n	Nominal flexural strength, t.m
M_{nt}	Required flexural strength in member assuming there is no lateral translation of the frame, t.m
M_o	maximum factored design moment between supports due to transverse loading, t.cm
M_p	Plastic bending moment, t.cm

M_r	Limiting buckling moment, t.cm
M_u	Required flexural strength, t.cm
M_y	Moment corresponding to onset of yielding, t.m
N	Bearing length, cm
N_L	Larger value of compression force
N_r	Number of stud connectors
N_s	Smaller value of compression force
P_{e1}, P_{e2}	Elastic Euler buckling load for braced and unbraced frame; respectively, t
P_n	Nominal axial strength (tension or compression), t
P_p	Bearing load on concrete, t
P_u	Required axial strength (tension or compression), t
P_y	Yield strength, t
Q	Full reduction factor for slender compression elements
Q_i	Nominal load effect
Q_m	Mean load effect
Q_n	Nominal strength of one stud shear connector, t
Q_s	Reduction factor for slender unstiffened compression elements
Q_u	Reduction factor for slender stiffened compression elements
r	Radius of gyration, cm
R_e	Hybrid girder factor
r_i	Minimum radius of gyration of individual component in a built-up member, cm
r_{ib}	Radius of gyration of individual component relative to centroidal axis parallel to member axis of buckling, cm
r_m	Radius of gyration of the steel section, pipe, or tubing in composite columns, cm
R_m	Mean resistance, t
R_n	Nominal strength (resistance), t
r_o	Polar radius of gyration about the shear center, cm
r_{ox}, r_{oy}	Radius of gyration about x and y axes at the smaller end of a tapered member, respectively, cm
R_{PG}	Plate girder bending strength reduction factor
R_v	Web shear strength, t
r_x, r_y	Radius of gyration about x and y axes, respectively, cm
r_{yc}	Radius of gyration about y axis referred to compression flange, or if reverse curvature bending, referred to smaller flange, cm

S	Elastic section modulus, cm^3
	Spacing of secondary members, m
	Longitudinal center-to-center spacing (pitch) of any two consecutive holes, cm
S_{eff}	Effective section modulus about major axis, cm^3
S_{xt}, S_{xc}	Elastic section modulus referred to tension and compression flanges, respectively, cm^3
T	Tension force due to service loads, t
	Bolt pretension force, t
	Thickness of connected part
t	Thickness of plate, cm
T_b	Specified pretension load in high-strength bolt, t
t_f	Flange thickness, cm
T_u	Required tensile strength due to factored loads, t
t_w	Web thickness, cm
U	Reduction coefficient, used in calculating effective net area
V_n	Nominal shear strength, t
V_R	Coefficient of variation of resistance
V_u	Required shear strength, t
W	Wind load
w	Unit mass of concrete, Kg/m^3
	Distance, cm
	Plate width; distance between welds, cm
w_r	Average width of concrete rib or haunch, cm
$\bar{x}$	Connection eccentricity, cm
x	Subscript relating symbol to strong axis bending
X_1, X_2	Beam buckling factors
x_o, y_o	Coordinates of the shear center with respect to the centroid, cm
y	Subscript relating symbol to weak axis bending
Z	Plastic section modulus, cm^3
	Distance from the smaller end of tapered member, cm
Δ_{oh}	Lateral inter-storage sidesway, cm
ζ	Exponent for alternate beam-column interaction equation
α	Separation ratio for built-up compression members
η	Exponent for alternate beam-column interaction equation
ϕ	Resistance factor

ϕ_b	Resistance factor for flexure
ϕ_c	Resistance factor for compression
ϕ_{sf}	Resistance factor for shear on the failure path
ϕ_t	Resistance factor for tension
ϕ_v	Resistance factor for shear
λ	Slenderness ratio
λ_c	Slenderness parameter
λ_e	Equivalent slenderness parameter
λ_{eff}	Effective slenderness ratio
λ_p	Limiting slenderness parameter for compact element
λ_r	Limiting slenderness parameter for non compact element
	Reduction factor
ψ, β, ω, η	Factors
δ_o	Maximum deflection due to transverse loading, cm
$\overline{\lambda_p}$	Normalized plate slenderness
ρ	Mass density
δ	Flexibility of the U-frame
ν	Poisson's ratio
α	Coefficient of thermal expansion
γ_i	Load factor

CHAPTER 1

GENERAL PROVISIONS

1.1 SCOPE AND APPLICATION

The Load and Resistance Factor Design (LRFD) is an improved approach to the design of structural steel for buildings. It involves explicit consideration of limit states, multiple load factors, resistance factors, and implicit probabilistic determination of reliability. The LRFD method is devised to offer the designer greater flexibility, more rationality, and possible overall economy. Steel structures and structural members shall be designed to be serviceable to permit proper functioning during the service life of structure and safe from collapse during construction.

The Egyptian Code of Practice for Steel Construction, Load and Resistance Factor Design; governs the design of structural members and frames that consist primarily of structural steel components, including the detail parts, welds, bolts, or other fasteners required in fabrication and erection. Meanwhile, the use of the Egyptian Code of Practice for Steel Construction and Bridges - Allowable Stress Design (ASD)-is permitted.

1.2 LIMITS OF APPLICABILITY
1.2.1 Types of Steel Structures

As used in this code, the term steel structure refers to the steel elements of the framed structures essential to transmit the imposed loads, where elements may be tension members, compression members, beams, columns, beam – columns, hangers, and connections. For steel structures other than framed-type structures e.g.

- Shell-type structures; silos, tanks, etc.
- Suspension-type structures; guyed towers, suspension roofs, etc.
- Special structures; offshore structures, etc.

and wherever applicable, the Egyptian Code of Practice for Steel Construction and Bridges - Allowable Stress Design (ASD) - shall be used.

1.2.2 Types of Construction Connections

Two basic types of construction connections and associated design assumptions are permissible under the conditions stated herein, and each will govern in a specific manner the strength of members and the types and strength of their connections.

Fully rigid, commonly designated as rigid-frame (continuous frame), assumes that connections have sufficient rigidity to maintain the angles between intersecting members.

The type of construction assumed in the design shall be indicated on the design documents. The design of all connections shall be consistent with the assumption.

When the connection restraint is ignored, commonly designated "simple framing," it is assumed that for the transmission of gravity loads the ends of the beams and girders are connected for shear only and are free to rotate. For simple framing the following requirements apply:

1- The connections and connected members shall be adequate to resist the factored gravity loads as simple beams.
2- The connections and connected members shall be adequate to resist the factored lateral loads.
3- The connections shall have sufficient inelastic rotation capacity to avoid overload of fasteners or welds under combined factored gravity and lateral loading.

The designer should, when incorporating connection restraint into the design, take into account the reduced connection stiffness on the stability of the structure and its effect on the magnitude of second order effects.

Construction utilizing rigid frame connections may be designed in LRFD using either elastic or plastic analysis provided that appropriate Code provisions are satisfied.

1.3 MATERIAL
1.3.1 Structural Steel
1.3.1.1 General

The materials generally used in steel construction are described below. Other grades of steels can be used provided that they are precisely specified and that their characteristics, such as yield stress, ultimate strength, ductility and weldability, enable the present code to be put into application.

1.3.1.2 Identification

1.3.1.2.1 Certified report or manufacturer's certificates, properly correlated to the materials used, intended or other recognized identification markings on the product, made by the manufacturer of the steel material, fastener or other item to be used in fabrication or erection, shall serve to identify the material or item as to specification, type or grade.

1.3.1.2.2 Unless otherwise approved, structural steel not satisfactorily identified as to material specification shall not be used unless tested in an approved testing laboratory. The results of such testing, taking into account both mechanical and chemical properties, shall form the basis for classifying the steel as to specifications, and for the determination of the limiting stresses.

1.3.1.2.3 Unidentified steel, if service conditions are acceptable according to criteria contained in relevant local or international reference standards, may be used for unimportant members or details, where the precise physical properties and weldability of the steel would not affect the strength of the structure.

1.3.1.3 Heavy Sections

For Heavy rolled sections or built-up sections made of plates exceeding 50mm thick, to be used as members subjected to primary tensile stresses due to tension or flexure, toughness need not to be specified if splices are made by bolting. If such members are spliced using complete-joint penetration welds, the steel shall be specified in the contract documents to be supplied with charpy V-notch testing in accordance with relevant local or international reference standards.

1.3.2 Mechanical Properties

The mechanical properties of structural steel shall comply with the requirement given in Section 1.3.3. Under normal conditions of usual temperatures, calculations shall be made for all grades of steel based on the following properties:

Mass Density	$\rho = 7.85$	t/m^3
Modulus of Elasticity	$E = 2100$	t/cm^2
Shear Modulus	$G = 810$	t/cm^2
Poisson's Ratio	$\nu = 0.3$	
Coefficient of Thermal Expansion	$\alpha = 1.2 \times 10^{-5}$	$/^{\circ}C$

1.3.3 Grades of Steel

Materials conforming to the Egyptian Standard Specification No.260/2004 and its modifications (Ministry of Industry) are approved for use under this code. For high strength steel without a definite yield point, the yield stress is determined by the 0.2% offset value. The term "yield stress" is used in this code as a generic term to denote either the yield point or the yield stress.

Grade of Steel	Minimum Values of Yield Stress (F_y) and Ultimate Strength (F_u)			
	Thickness (t)			
	$t \leq 40$ mm		40mm< $t \leq 100$ mm	
	F_y (t/cm^2)	F_u (t/cm^2)	F_y (t/cm^2)	F_u (t/cm^2)
St 37	2.40	3.70	2.15	3.40
St 44	2.80	4.40	2.55	4.10
St 52	3.60	5.20	3.35	4.90

1.3.4 Cast Steel and Forged Steel

Cast iron, cast steel, and forged steel shall conform to the Egyptian Standard Specification No.260/2004 and its modifications (Ministry of Industry).

1.3.4.1 Cast Steel

Steel castings shall be of one of the two following grades in accordance with the purpose for which they are to be used, as specified on the drawings and as prescribed in the special specification.

a- Castings of grade C St 44 for all medium-strength carbon steel castings; for general use and in parts not subjected to wearing on their surfaces.

b- Castings of grade C St 55 for all high-strength-carbon steel castings which are to be subjected to higher mechanical stresses than C St 44; for use in parts subjected to wearing on their surface such as pins, hinges, parts of bearings and machinery.

Steel for castings shall be made by the open-hearth process (acid or basic) or electric furnace process, as may be specified. On analysis it must show not more than 0.06% of sulphur or phosphorus.

1.3.4.2 Forged Steel

The following prescriptions apply to carbon steel forging.

The forging shall be of the following grades according to the purpose for which they are used:

a- Forging of grade F St 50, annealed or normalized; for mild steel forging of bearings, hinges, trunnions, shafts, bolts, nuts, pins, keys, screws, worms; tensile strength from 5.0 to 5.6 t/cm^2 and minimum yield point stress 2.4 t/cm^2 are used.

b- Forging of grade F St 56, normalized, annealed or normalized and tempered; for various carbon steel machinery, structural forging of pinions, levers, cranks, rollers, tread plates; tensile strength from 5.6 to 6.3 t/cm^2 and minimum yield point stress 2.8 t/cm^2 are used. The grade required shall be specified on the drawings or in the special specification.

Carbon steel for forging shall be made by the open hearth or an electric process, acid or basic, as may be specified.

The steel shall contain not more than 0.05% of sulphur or of phosphorus, 0.35% of carbon, 0.8% of manganese, 0.35% of silicium.

1.3.5 Bolts, Nuts and Washers

- Bolts, nuts and washers shall conform to relevant local or international reference standard.

- Bolts of grades lower than 4.6 or higher than 10.9 shall not be used unless test results prove their acceptability in a particular application.
- Carbon and alloy steel nuts for bolts for high-pressure and high-temperature service, shall conform to relevant local or international reference standard.

1.3.6 Anchor Bolts and Threaded Rods

Anchor bolts and threaded steel rods shall conform to relevant local or international reference standard.

1.3.7 Filler Metal and Flux for Welding

Welding electrodes and fluxes shall conform to relevant local or international reference standards. Manufacturer's certification shall constitute sufficient evidence of conformity with the standard. Electrodes (filler metals) that are suitable for the intended application shall be selected. Weld metal notch toughness is generally not critical for building construction.

1.3.8 Stud Shear Connectors

Steel stud shear connectors shall conform to the requirements of relevant local or international reference standards. Manufacturer's certification shall constitute sufficient evidence of conformity with the code.

1.4 LOADS AND LOAD COMBINATIONS

The nominal loads shall be the minimum design loads stipulated by the applicable code under which the structure is designed or dictated by the conditions involved. The loads shall be those stipulated in the Egyptian Code of Practice for Loads and Forces for Structural Elements (ECPLF–latest edition).

1.4.1 Loads, Load Factors, and Load Combinations

The nominal loads D, L, L_r, W, and EQ are the code (ECPLF– latest edition) loads to be considered, Where:

D = dead load due to the weight of the structural elements and the permanent features on the structure

L = live load due to occupancy and movable or vibrating equipments including its dynamic effect (impact, vibration...etc.)

L_r = roof live load

W = wind load

EQ = earthquake load

The required strength of the structure and its elements must be determined from the appropriate critical combination of factored loads. The most critical effect may occur when one or more loads are not acting. The following load combinations and the corresponding load factors shall be investigated:

$$1.4D \quad (1.1)$$

$$1.2D + 1.6L + 0.5\,L_r \quad (1.2)$$

$$1.2D + 1.6\,L_r + (0.5L \text{ or } 0.8\,W) \quad (1.3)$$

$$1.2D + 1.3W + 0.5L + 0.5\,L_r \quad (1.4)$$

$$1.2D \pm 1.0EQ + 0.5L \quad (1.5)$$

$$0.9D \pm (1.3W \text{ or } 1.0EQ) \quad (1.6)$$

The load factors and load combinations recognize that when several loads act in combination with the dead load (e.g., dead plus live plus wind), only one of these takes on its maximum lifetime value, while the other load is at its "arbitrary point-in-time value" (i.e., at a value which can be expected to be on the structure at any time).

For garages, areas occupied as places of public assembly, and all areas where the live load is greater than 500 kg/m^2, the load factor for L in combinations 1.3, 1.4, and 1.5 shall be equal 1.0.

1.5 DESIGN BASIS

1.5.1 Limit States

Limit state is a condition in which a structure or structural component becomes unfit. Limit states of strength vary from member to member, and several limit states may apply to a given member. Two kinds of limit states apply for structures; limit states of strength which define safety against the extreme loads during the intended life of the structure, and limit states of serviceability which define the functional requirements.

The following limit states of strength are the most common; onset of yielding, formation of a plastic hinge, formation of a plastic mechanism, overall frame or member instability, lateral-torsional buckling, local buckling, tensile fracture, development of fatigue cracks, deflection instability, alternating plasticity, and excessive deformation. The most common serviceability limit states include unacceptable elastic deflections and drift, unacceptable vibrations, and permanent deformations.

LRFD is a method of proportioning structures so that no applicable limit state is exceeded when the structure is subjected to all appropriate factored load combinations. The designation LRFD reflects the concept of factoring both loads and resistance. The term "resistance" includes both strength limit states and serviceability limit states.

1.5.2 Required Strength at Factored Loads

The required strength of structural members and connections shall be determined by structural analysis for the appropriate factored load combinations given in Section 1.4.

Design by either elastic or plastic analysis is permitted, except that design by plastic analysis is permitted only for steels with specified yield stresses not exceeding 4.5 t/cm^2 and is subject to the appropriate provisions of the code.

Beams and girders composed of compact sections, as defined in Section 2.3.1, and satisfy the unbraced length requirements of Section 5.1.3.1 which are continuous over supports or are rigidly framed to columns may be proportioned for nine-tenths (0.90) of the negative moments produced by gravity loading at points of support, provided that the maximum positive moment is increased by one-tenth (0.10) of the average negative moments. This reduction is not permitted for hybrid beams, or moments produced by loading on cantilevers. If the negative moment is resisted by a column rigidly framed to the beam or girder, the one-tenth reduction may be used in proportioning the column for combined axial force and flexure, provided that the axial force does not exceed ϕ_c times $0.15\ A_g F_y$.

Where:

A_g = gross area, cm^2

F_y = specified minimum yield stress, t/cm^2

ϕ_c = resistance factor for compression

1.5.3 Design for Strength

1.5.3.1 Basic Concept

The general format of the LRFD Code is given by the formula:

$$\Sigma \gamma_i Q_i \leq \phi R_n \qquad (1.7)$$

Where:

Σ	=	summation
i	=	type of load, i.e., dead load, live load, wind, etc.
Q_i	=	nominal load effect
γ_i	=	load factor corresponding to Q_i
$\Sigma \gamma_i Q_i$	=	required strength
R_n	=	nominal resistance
ϕ	=	resistance factor corresponding to R_n
ϕR_n	=	design strength

The term $\Sigma \gamma_i Q_i$ represents the required resistance (required strength) computed by structural analysis based upon assumed loads, and the term ϕR_n represents a limiting structural capacity provided by the selected members (i.e. design strength). The resistance factors ϕ and the load factors γ account for unavoidable inaccuracies in the theory, variations in the material properties and dimensions, and uncertainties in the determination of loads.

The design strength of each structural component or assemblage must equal or exceed the required strength for each applicable limit state based on the corresponding applicable factored load combination as stipulated in Section 1.4. Nominal design strengths R_n and resistance factors ϕ are given in the following chapters.

1.5.3.2 Basis of LRFD Probability Theory

The load effects Q and the resistance R are assumed to be statistically independent random variables as shown in Fig. 1.1. As long as the resistance R is greater than the effects of the loads Q, a margin of safety for the particular limit state exists. However, because Q and R are random variables, there is some small probability that R may be less than Q, $(R < Q)$.

If the expression $R < Q$ is divided by Q and the result expressed logarithmically, the result will be a single frequency distribution curve combining the uncertainties of both R and Q. The probability of attaining a limit state $(R < Q)$ is equal to the probability that $\ln(R/Q) < 0$ and is represented by the shaded area in the diagram shown in Fig. 1.2.

The distance from the origin to the mean is measured as the number of standard deviations of the function $\ln(R/Q)$, as shown in Fig. 1.2, this is stated as β times $\sigma \ln(R/Q)$, the standard deviation of $\ln(R/Q)$. The factor β therefore is called the "reliability index".

The distribution shape of each of the many variables (material, loads, etc.) has an influence on the shape of the distribution of $\ln(R/Q)$. Often only the means and the standard deviations of the many variables involved in the makeup of the resistance and the load effect can be estimated. However, this information is enough to build an approximate design criterion which is independent of the knowledge of the distribution, by stipulating the following design condition:

$$\beta \sigma \ln (R / Q) \approx \beta \sqrt{V_R^2 + V_Q^2} \leq \ln(R_m / Q_m) \quad \ldots\ldots 1.8$$

Where:

$V_R = \sigma_R/R_m$ and $V_Q = \sigma_Q/Q_m$ (σ_R and σ_Q are the standard deviations, R_m and Q_m are the mean values, V_R and V_Q are the coefficients of variation, of the resistance R and the load effect Q; respectively).

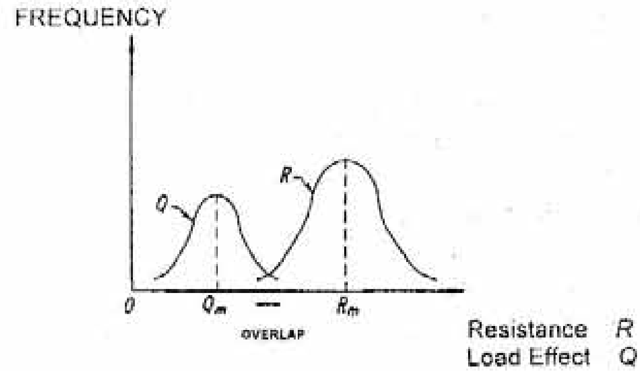


Figure 1.1 Frequency Distribution of Load Effect *Q* and Resistance *R*

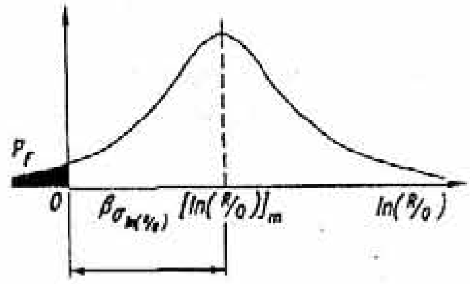


Figure 1.2 Definition of Reliability Index

For structural elements and the usual loadings R_m, Q_m, and the coefficients of variation, V_R and V_Q, can be estimated, so a calculation of

$$\beta = ln(R_m / Q_m) \qquad 1.9$$

will give a comparative value of the measure of reliability of a structure or component. Computer methods as well as charts can be used to determine the resistance factors ϕ. These factors can also be approximately determined by the following:

$$\phi = \left(\frac{R_m}{R_n}\right) e^{(0.55 \beta V_R)} \qquad 1.10$$

Where:

R_m = mean resistance

R_n = nominal resistance according to the equations in this code

V_R = coefficient of variation of the resistance

1.5.4 Design for Serviceability and Other Considerations

The overall structure and the individual members, connections, and connectors shall be checked for serviceability. Nominally, serviceability should be checked at the unfactored loads. For combinations of gravity and wind or seismic loads some additional reduction factor may be warranted. Provisions for design for serviceability are given in Chapter 14.

1.6 DESIGN DOCUMENTS

The design plans shall show a complete design with sizes, sections, and relative locations of the various members. Floor levels, column centers and offsets shall be dimensioned. Drawings shall be drawn to a scale large enough to show the information clearly.

Design documents shall indicate the type or types of construction and include the required strengths (moments and forces) if necessary for preparation of shop drawings.

Where joints are to be assembled with high strength bolts, the design documents shall indicate the connection type (i.e. snug-tight bearing, fully tensioned bearing, direct tension, or slip-critical).

Cambers of trusses, beams, and girders, if required, shall be specified in the design documents. The requirements for stiffeners and bracing shall be shown in the design documents.

Weld lengths shown on the drawings shall be the net effective lengths.

1.7 UNITS

- For calculation, the following units are recommended:

  - forces and loads t, t/m, t/m^2
  - stresses and strengths t / cm^2
  - moments (bending) t.cm

- S.I. units could be used along with the above recommended units.

CHAPTER 2

DESIGN REQUIREMENTS

This chapter contains provisions that are common to the code as a whole.

2.1 GROSS AREA, NET AREA & EFFECTIVE AREA

2.1.1 Gross Area

The gross area A_g of a member at any point is the sum of the products of the thickness and the gross width of each element measured normal to the axis of the member.

2.1.2 Net Area

The net area A_n of a member is the sum of the products of the thickness and the net width of each element computed as follows:

a- In computing the net area for tension, the diameter of a bolt hole shall be taken in accordance with section 8.3.

b- For net area subject to shear the width of the bolt hole shall be taken equal to the nominal clearance only.

c- For a chain of holes extending across a part in any diagonal or zigzag line, the critical net area is based on the net width and load transfer at a particular chain. The net width of the parts shall be obtained by deducing from the gross width the sum of the diameters or slot dimensions of all holes in the chain, and adding, for each gauge space in the chain, the quantity ($S^2/4g$) as shown in Fig. 2.1.

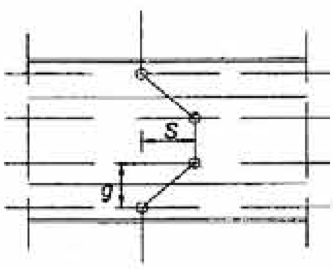


Figure 2.1 Critical Sections in a Tension Member

Where:

S = the staggered pitch, i.e., the longitudinal distance measured parallel to the direction of stress in the member, center – to – center of fasteners in any consecutive lines, cm

g = the gauge, i.e., the transverse distance, measured at right angles to the direction of stress in the member, center – to - center spacing of fasteners in any consecutive lines, cm

- For angles with staggered holes in two legs, the gauge length "g" for holes in opposite adjacent legs shall be the sum of the gauges from the back of the angles less the thickness.

- In determining the net area across plug or slot welds the weld metal shall not be considered as adding to the net area.

2.1.3 Effective Net Area for Tension Members (A_e)

This section deals with the effect of shear lag, which is applicable to both welded and bolted tension members. The reduction coefficient U is applied to the net area A_n of bolted members and to the gross area A_g of welded members. As the length of connection ℓ is increased, the shear lag effect is diminished. Equation 2.1 expresses this concept empirically.

$$A_e = A\,U \qquad \text{..........} \qquad 2.1$$

Where:

A = A_n = net area for bolted members

A = A_g = gross area for welded members

U = reduction coefficient

= $1-(\bar{X}\ /\ell) \leq 0.9$ 2.2

$\bar{X}$ = connection eccentricity

ℓ = length of connection in the direction of loading

2.1.3.1 Determination of the Connection Eccentricity ($\bar{X}$)

For any given profile and connected elements $\bar{X}$ is a fixed geometric property. It is defined as the distance from the connection plane or face of the member, to the centroid of the member resisting the connection force as shown in Fig. 2.2.

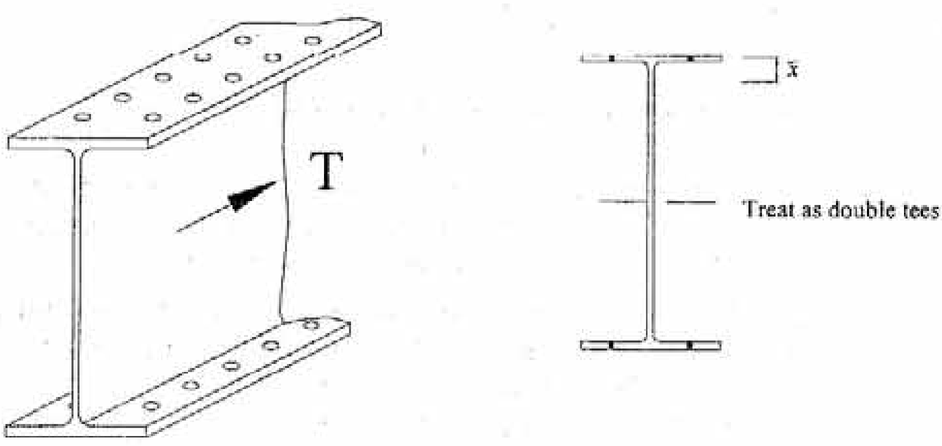


Figure 2.2a Determination of $\bar{X}$ for I-Sections Connected at Flanges

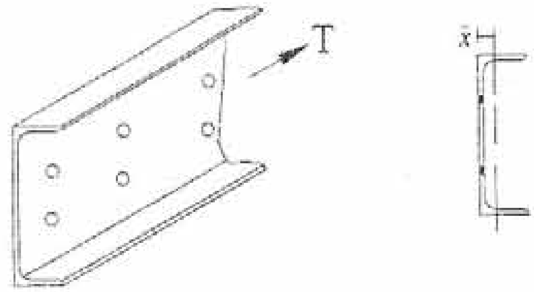


Figure 2.2b Determination of $\bar{X}$ for Channel-Sections Connected at Web

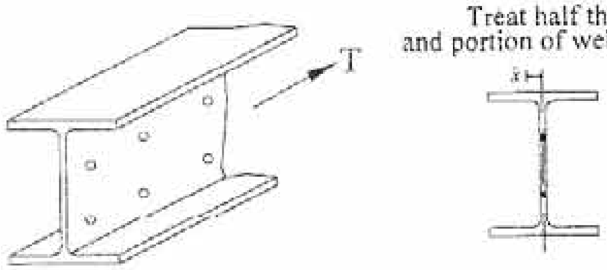


Figure 2.2c Determination of $\bar{X}$ for I-Sections Connected at Web

2.1.3.2 Determination of the Connection Length (ℓ)

The length ℓ is dependent upon the number of fasteners or equivalent length of weld required to develop the given tensile force, and this in turn is dependent upon the mechanical properties of the member and the capacity of the fasteners or weld used.

For Bolted Connections

The determination of the connection length (ℓ) for bolted connections shall satisfy the following:

a- For one gauge line the length ℓ is the distance, parallel to the line of force between the first and last fasteners (refer to Fig. 2.3a).

b- For staggered bolts, the out – to – out distance is used for ℓ as shown in Fig. 2.3b.

c If all lines have only one bolt the effective area A_e shall be taken equal to the net area A_n of the connected element.

d- When the tension load is transmitted directly to each of the cross sectional elements by fasteners, the effective area A_e shall be taken equal to the net area A_n.

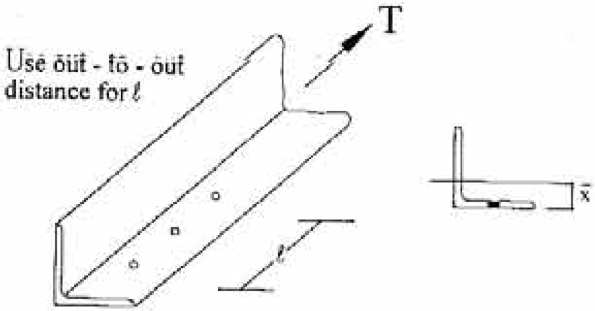


Figure 2.3a Determination of ℓ and $\bar{X}$ for One Gauge Line

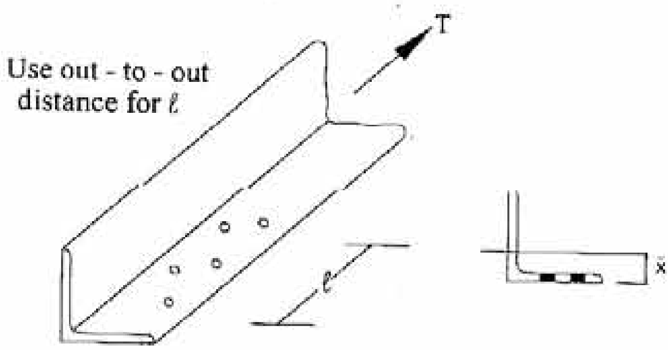


Figure 2.3b Determination of ℓ and $\bar{X}$ for Staggered Holes

The determination of the connection length ℓ for welded connections shall satisfy the following:

a- For longitudinal welded connections, ℓ is the greater of the weld lines parallel to the line of force as illustrated in Fig. 2.4(a).
b- For combination of longitudinal and transverse welds (ℓ) is the greater length of longitudinal welds because the transverse weld has little effect on the shear lag phenomena, refer to Fig. 2.4(b).
c- For transverse weld only, the effective area A_e shall be taken equal to the gross area A_g of the connected element, refer to Fig. 2.4(c).
d- When the tension load is transmitted to a plate by longitudinal welds along both edges at the end of the plate, no need to apply Equation 2.2 where the following reduction coefficients are to be utilized:

For $\ell \geq 2w$ .... $U = 1.0$

For $2w > \ell \geq 1.5w$ .... $U = 0.87$

For $1.5w > \ell \geq w$ .... $U = 0.75$

Where:

ℓ = length of longitudinal weld, cm

w = plate width (distance between welds), cm

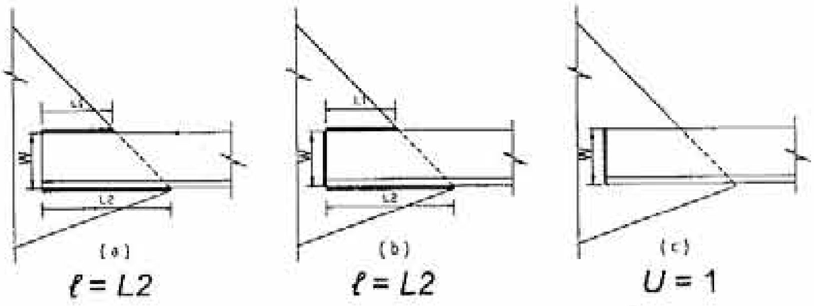


Figure 2.4 Determination of ℓ in Welded Connections

e- When the tension load is transmitted directly to each of the cross sectional elements by welds, the effective area A_e shall be taken equal to the gross area A_g.

2.1.3.3 Approximation for the Determination of the Reduction Coefficient "U"

Conservative approximate values of the reduction coefficient U may be utilized according to Tables 2.1 and 2.2 for bolted and welded connections respectively as follows:

Table 2.1 Approximate U Coefficient for Bolted Connections

No.	Configuration	Conditions	U	A_e
1	For all types of cross sections where the tension load is transmitted directly by all the cross sectional elements	---------	1.0	A_n
2	Gusset plates, splice plates or any short connecting element	$n \geq 1$	1.0	A_n $\geq 0.85\ A_g$
3	Rolled I-sections, C-sections, T-sections cut from I-sections, built up sections, connected by the flanges with three or more fasteners	$b/d < 0.67$ $b/c \geq 0.67$	0.85 0.90	$0.85\ A_n$
4	Rolled I-sections, C-sections, T-sections cut from I-sections, built up sections, connected by the web with four or more fasteners		0.70	$0.85\ A_n$
5	Single angles	$n < 4$ $n \geq 4$	0.60 0.80	$0.85\ A_n$
6	All sections	$n \geq 2$	0.75	$0.75\ A_n$

Where: b = flange width, d = section depth
n = number of fasteners per line in the direction of loading in the connection

Table 2.2 Approximate U Coefficient for Welded Connections

No.	Configuration	Conditions	U	A_e
1	For all types of cross sections where the tension load is transmitted directly by all the cross sectional elements	---------	1.0	A_g
2	Gusset plates, splice pates or any short connecting element	---------	1.0	A_g
3	Rolled I-sections T- sections cut from I-sections	$b/d \geq 0.67$ The connection is to the flanges	0.9	$0.9\ A_g$
4	Rolled I-sections, C-sections, T-sections cut from I-sections, Built up sections. All angle sections configurations, Other sections	$b/d \leq 0.67$ The connection is either to the flanges	0.85	$0.85\ A_g$
5	Flat plates having loads transmitted by longitudinal weld along both edges // to load (Fig. 2.4)	$\ell/w \geq 2$	1.0	A_g
		$2 > \ell/w \geq 1.5$	0.87	$0.87A_g$
		$1.5 > \ell/w \geq 1.0$	0.75	$0.75A_g$
6	Circular hollow sections welded to a single concentric gusset plate	$\ell/D \geq 1.3$ $1 < \ell/D < 1.3$	1.0 $1-\frac{D}{\pi\ \ell}$	

Where: b = flange width, d = section depth, ℓ = length of connection, w = plate width, D = diameter of circular hollow section

2.2 STABILITY

General requirements for stability of the structure as well as individual members are provided. This includes the second order effect of axial loading on bending stresses as well as the proper determination of the critical buckling load and corresponding K-factor.

2.2.1 Slenderness Ratios

2.2.1.1 General

a- General stability shall be checked for the structure as a whole and for each individual member.

b- The slenderness ratio of a member shall be taken as

$$\lambda = \frac{KL}{r} \qquad \text{.......... 2.3}$$

c- The slenderness parameter $\lambda_c = \lambda\sqrt{\frac{F_y}{\pi^2 E}}$

Where:

λ = the slenderness ratio

K = the buckling length factor
For a compression member, K depends on the rotational restraint at the member ends and the means available to resist lateral movements
For tension members, $K = 1.0$

L = the unsupported length for tension or compression members

r = the radius of gyration of the gross section corresponding to the axis of buckling

2.2.1.2 Maximum Slenderness Ratios (λ_{max})

The slenderness ratio of compression and tension members, shall not exceed λ_{max} of Table 2.3

Table 2.3 Maximum Slenderness Ratio for Axially Loaded Members

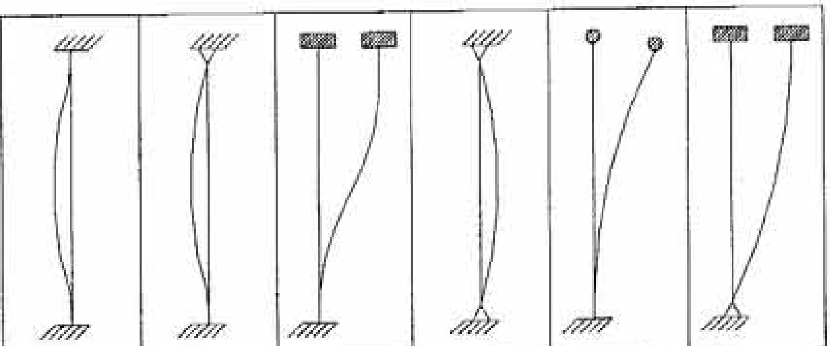
Member	λ_{max}
Compression members	180
Bracing systems and secondary compression members	200
Tension members	300

The use of rods and cables in bracing systems or as a main tension member is prohibited in this code.

2.2.1.3 Buckling Length Factor (K)

a- The recommended values for the buckling length factor K (Equation 2.3) are given in Table 2.4 for members with well-defined (idealized) end conditions.

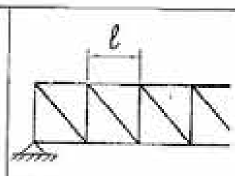
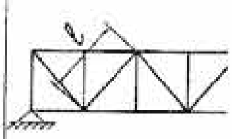
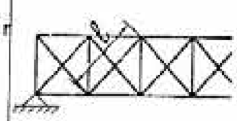
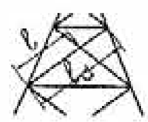
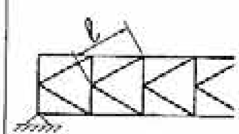
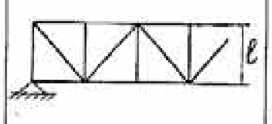
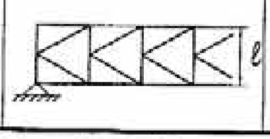
Table 2.4 Buckling Length Factor for Members with Well Defined End Conditions

BUCKLING MODE						
K	0.65	0.80	1.20	1.00	2.10	2.00
END CONDITIONS		ROTATION PREVENTED, TRANSLATION PREVENTED				
		ROTATION PERMITTED, TRANSLATION PREVENTED				
		ROTATION PREVENTED, TRANSLATION PERMITTED				
		ROTATION PERMITTED, TRANSLATION PERMITTED				

b- **Trusses**

The effective buckling length (KL) of a compression member in a truss is obtained from Table 2.5 or determined from an elastic critical buckling analysis of the truss.

Table 2.5 Effective Buckling Lengths of Compression members

Member		In-Plane	Out-of-Plane: Compression Chord — Effectively Braced	Out-of-Plane: Compression Chord — Unbraced
Chords		ℓ	ℓ	0.75 span
Diagonals -Single Triangulated web system		ℓ	ℓ	$1.2\,\ell$
-Multiple Intersected web rectangular system adequately connected		$0.5\,\ell$	$0.75\,\ell$	ℓ
-Multiple Intersected web trapezoidal system adequately connected		ℓ	$0.8\,\ell_d$	——
- K-system		ℓ	$1.2\,\ell$	$1.5\,\ell$
Vertical members -Single triangulated web system		ℓ	ℓ	$1.2\,\ell$
-K-intersected web system		$0.5\,\ell$	$(0.75+0.25\frac{N_s}{N_L})\ell$	$(0.90+0.30\frac{N_s}{N_L})\ell$

N_s = Smaller value of compression force
N_L = Larger value of compression force

- For a simply supported truss, with laterally unsupported compression chords and with no cross-frames but with each end of the truss adequately restrained (Fig. 2.5), the effective buckling length (KL), shall be taken equal to 0.75 of the truss span.

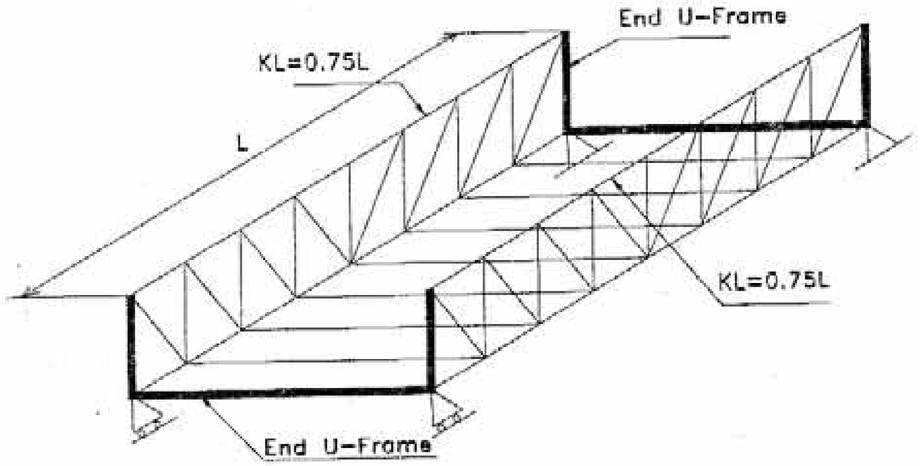


Figure 2.5 Truss with the Compression Chord Laterally Unbraced

- For trusses where the compression chord is laterally restrained by U-frames composed of the cross girders and verticals of the trusses, the effective buckling length of the compression chord (ℓ_b) is:

$$\ell_b = 2.5 \cdot \sqrt[4]{E \cdot I_y \cdot a \cdot \delta} \geq a \quad \text{.......... 2.4}$$

Where:

E = the Young's modulus, t/cm^2

I_y = the moment of inertia of the chord member about the Y-Y axis shown in Fig. 2.6, cm^4

a = the distance between the U-frames, cm

δ = the flexibility of the U-frame: the lateral deflection near mid-span at the level of the considered chord's centroid due to a unit load acting laterally at each chord connected to the U-frame. The unit load is applied only at the point at which δ is being calculated. The direction of each unit load shall produce a maximum value for δ, cm

The U-frame is considered to be free and unconnected at all points except at each point of intersection between cross girder and vertical of the truss where this joint is considered to be rigidly connected.

In case of symmetrical U-frame with constant moment of inertia for each of the cross girder and the verticals through their own length, δ may be taken from:

$$\delta = \frac{d_1^3}{3EI_1} + \frac{d_2^2 B}{2EI_2} \quad \text{.......... 2.5}$$

Where:

d_1 = the distance from the centroid of the compression chord to the nearest face of the cross girder of the U-frame, cm

d_2 = the distance from the centroid of the compression chord to the centroidal axis of the cross girder of the U-frame, cm

I_1 = the second moment of area of the vertical member forming the arm of the U-frame about the axis of bending, cm^4

I_2 = the second moment of area of the cross girder about the axis of bending, cm^4

B = the distance between centers of consecutive main girders connected by the U-frame, cm

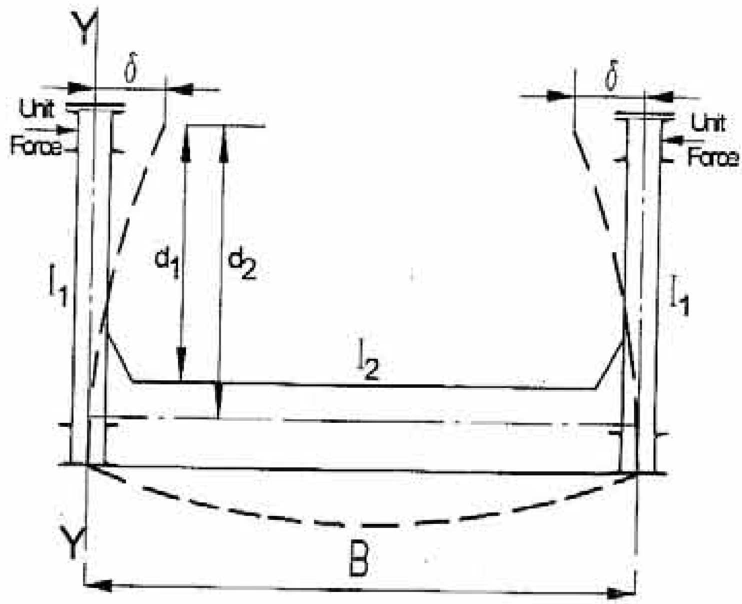


Figure 2.6 Lateral Restraints of Truss Chords by U-Frame

c- Columns in Rigid Frames

i- Braced Frames

In trusses and frames where lateral stability is provided by diagonal bracing, shear walls, or equivalent means, the effective buckling length factor K for compression members shall be taken as unity, unless structural analysis shows that a smaller value may be used. The alignment charts may be used to determine the effective buckling length.

The vertical bracing system for a braced multistory frame shall be determined by structural analysis to be adequate to prevent buckling of the structure and to maintain the lateral stability of the structure, including the overturning effects of drift, under the factored loads given in Section 1.4.

The vertical bracing system for a multistory frame may be considered to function together with in-plane shear-resisting exterior and interior walls, floor slabs, and roof decks, which are properly secured to the structural frames. The columns, girders, beams, and diagonal members, when used as the vertical bracing system, may be considered to comprise a vertically cantilevered simply connected truss in the analysis of frame buckling and lateral stability. Axial deformation of all members in the vertical bracing system shall be included in the lateral stability analysis.

In structures designed on the basis of plastic analysis, the axial force in these columns caused by factored gravity plus factored horizontal loads shall not exceed the value of $0.85\phi_c A_g F_y$.

ii- Unbraced Frames

In frames where lateral stability depends upon the bending stiffness of rigidly connected beams and columns, the effective buckling length factor K of compression members shall be determined by structural analysis. The alignment charts may be used to determine the effective buckling length. The destabilizing effects of gravity loaded columns whose simple connections to the frame do not provide resistance to lateral loads shall be included in the design of the moment-frame columns. Stiffness reduction adjustment due to column inelasticity is permitted.

Analysis of the required strength of unbraced multistory frames shall include the effects of frame instability and column axial deformation under the factored loads given in Section 1.4.

In structures designed on the basis of plastic analysis, the axial force in these members caused by factored gravity plus factored horizontal loads shall not exceed the value of $0.75\phi_c A_g F_y$.

iii- Effective Buckling Length of Columns in Rigid Frames

The evaluation of effective buckling length factor (K) for columns in rigid frames may be obtained from the alignment charts in Fig. 2.7. These charts are a function of the ratio of the moment of inertia to length of members, (I/L) values.

A conservative approach is to assume that all columns in the portion of the frame under consideration reach their individual buckling loads simultaneously. These charts are based on a slope-deflection analysis. In Fig. 2.7, the subscripts A and B refer to the points at the two ends of the column under consideration. G is defined as:

$$G = \frac{\sum(I/L)\,columns}{\sum(I/L)\,girders} \qquad \text{2.6}$$

Where:

- The summation (Σ) indicates a summation of all members rigidly connected to that joint (A or B) and lying in the plane in which buckling of the column is being considered.

- (I) is the moment of inertia of each member, taken about the axis perpendicular to the plane of buckling.

- (L) is the unsupported length for both columns and girders.

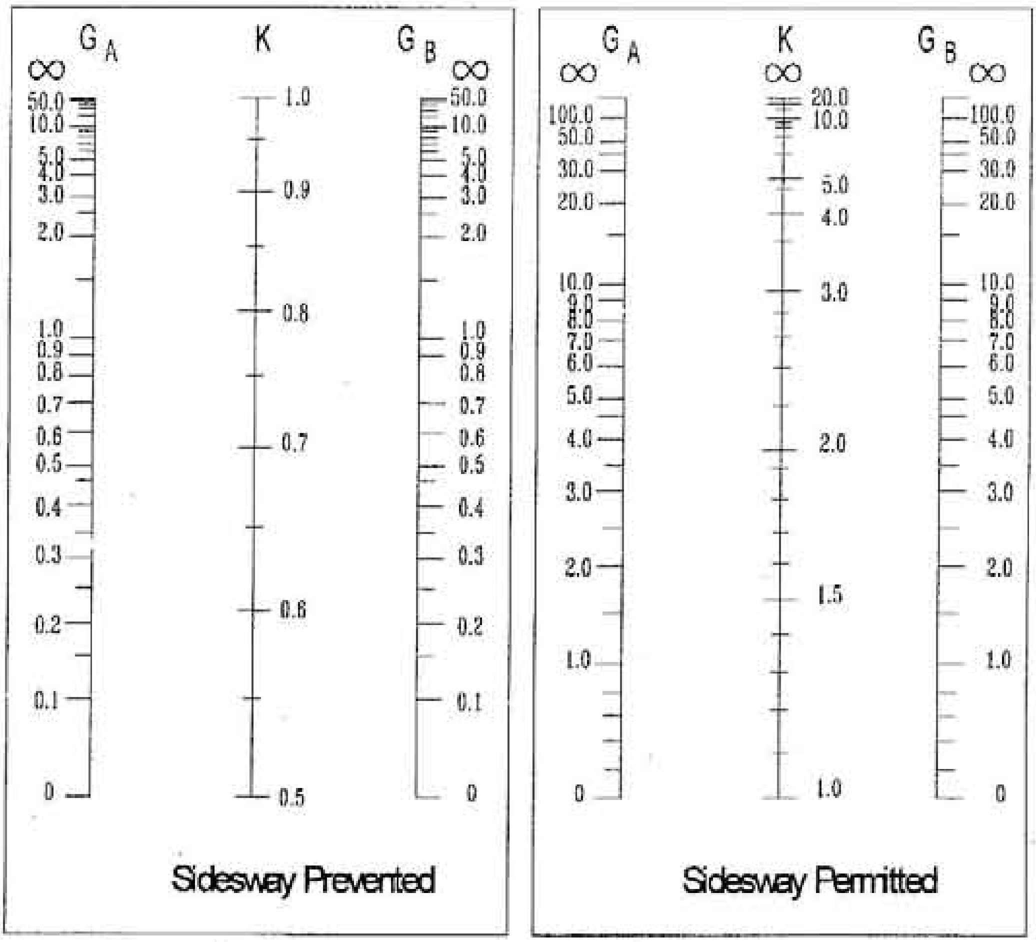


Figure 2.7 Alignment Charts for Buckling Length Factor (K) of Columns in Rigid Frames

Equation 2.6 shall be modified for the following special cases (Table 2.6a and 2.6b)

- For column base connected to a footing or foundation by a frictionless hinge, G is theoretically infinity, but should be taken as 10 in design practice.
- For column base rigidly attached to properly designed footing, G approaches a theoretical value of zero, but should be taken as 1.0 in design practice.
- The girder stiffness (I/L) should be multiplied by a factor when certain conditions at the far end are known to exist

Table 2.6a Recommended G values for Columns with Special End Conditions

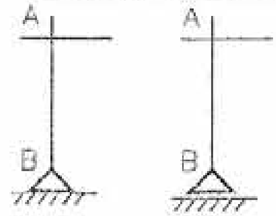
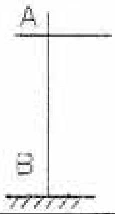
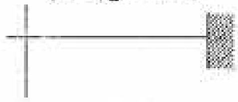
Column Base Condition		
G_B	$G_B = 10.0$	$G_B = 1.0$

Table 2.6b Beams with Special End Conditions

Sidesway prevented	$(I/L)_g$ X 1.5 	$(I/L)_g$ X 2.0 
Sidesway permitted	$(I/L)_g$ X 0.5 	$(I/L)_g$ X 0.67 

- For the case of sidesway prevented, the appropriate multiplication factors are 1.5 if the far end of the girder is hinged, and 2.0 if the far end of the girder is fixed.
- For the case of sidesway permitted, the appropriate multiplication factors are 0.5 if the far end of the girder is hinged, and 0.67 if the far end of the girder is fixed.
- When a column of slenderness ratio KL/r is inelastic (i.e. $\lambda_c < 1.1$) and beams are elastic, an adjustment is to be made in the restraint factor G by multiplying it with the reduction factor β_s given in Table 2.7.

Table 2.7 Reduction β_s Factor for Inelastic Columns

λ_c	0.10	0.20	0.30	0.40	0.50	0.60	.70	0.80	0.90	1.00	1.10
β_s	0.01	0.06	0.13	0.23	0.35	0.48	0.61	0.75	0.86	0.95	1.00

Having determined G_A and G_B for a column, the buckling length factor (K) is obtained by constructing a straight line between the appropriate points of the scales of G_A and G_B. The intersection of the straight line with the middle scale gives the K value. If the conditions used to develop this chart are not met, corrections have to be made.

iv. Leaning Columns

Weak columns (with high axial forces) sized for vertical loads only; (based on assumed K=1.0); are called leaning columns. Such columns have no shear resistance to lateral loads. Leaning columns when used in unbraced frames receive lateral stability from the stiffness of strong columns (with low axial force) having rigid-resisting moment connections. As such, an adjusted distribution of stiffness to the i^{th} strong column of a storey is given by:

$$K_i^{\prime} = \sqrt{\frac{\sum P_c}{P_{ci}} \frac{I_i}{\sum \frac{I_i}{K_i^2}}} \geq \sqrt{\frac{5}{8}} K_i \qquad \text{.......... 2.7}$$

$$P_{ci} = A_i F_c$$

Where:

K'_i and K_i	=	the adjusted and unadjusted buckling length factor for column resisting sidesway.
A_i and I_i	=	the cross sectional area and moment of inertia: respectively of the considered column
F_c	=	the axial compressive stress = $0.58F_{cr}$ as defined in Chapter 4
P_{ci}	=	the axial compressive strength of the i^{th} rigidly connected column
ΣP_c	=	the axial compressive strength of all columns in a storey

2.2.1.4 Buckling Length of Compression Flange of Beams

a- Simply Supported Beams

The effective buckling length of compression flange of simply supported beams shall be considered as follows:

i- Compression Flange with No Intermediate Lateral Support

Table 2.8 defines the effective buckling length of compression flange of simply supported beams having no intermediate supports.

ii- Compression Flange with Intermediate Lateral Support

Table 2.9 defines the effective buckling length of compression flange of simply supported beams having intermediate lateral supports.

Table 2.8 Buckling Length of Compression Flange of Simply Supported Beams Having no Intermediate Lateral Supports

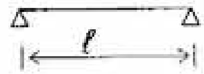
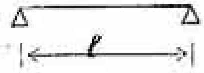
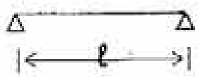
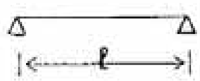
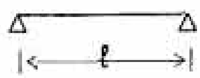
Compression Flange End Restraint Conditions	Beam Type	Buckling Length ($K\ell$)
End of compression flange unrestrained against lateral bending		ℓ
End of compression flange partially restrained against lateral bending		0.85 ℓ
End of compression flange fully restrained against lateral bending		0.70 ℓ

Table 2.9 Buckling Length of Compression Flange of Simply Supported Beams Having Intermediate Lateral Supports

Compression Flange End Restraint Conditions	Beam Type	Buckling Length ($K\ell$)
Beams where there is no bracing to support the compression flange laterally, but where cross beams and stiffeners forming U-frames provide lateral restraint		The effective buckling length is according to section 2.2.1.3
Beams where there is an effective lateral bracing to the compression flange		Distance between centers of intersection of the bracing with the compression chord
Beams where the compression flange is unbraced but supported by rigid cross girders		Distance between centers of cross girders
Beams where the compression flange is supported by continuous reinforced concrete or steel deck, where the frictional or connection of the deck to the flange is capable to resist a lateral force of 2% of the flange force at the point of the maximum bending moment		K = 0

b- Cantilever Beams with Intermediate Lateral Supports

The effective buckling length of compression flange of cantilever beams with intermediate lateral supports shall be similar to that of simply supported beams having lateral supports as given in Section 2.2.1.4.

c- Cantilever Beams without Intermediate Lateral Supports

The effective buckling length of compression flange of cantilever beams without intermediate lateral supports shall be according to Table 2.10. The loading condition (normal or destabilizing) is defined by the point of application of the load. Destabilizing load conditions exist when a load is applied to the top flange of a beam or cantilever and both the load and the flange are free to deflect laterally (and possibly rotationally also) relative to the centroid of the beam. The type of restraint provided to the cantilever tip is detailed in Fig. 2.8.

Table 2.10 Effective Buckling Length of Compression Flange of Cantilever Beams without Intermediate Supports

Restraint Conditions		Loading Conditions	
At support	**At tip**	**Normal**	**Destabilizing**
Continuous with lateral restraint only (refer to Fig. 2.9)	Free	3.0 ℓ	7.5 ℓ
	Laterally restrained on top flange only	2.7 ℓ	7.5 ℓ
	Torsionally restrained only	2.4 ℓ	4.5 ℓ
	Laterally and torsionally restrained	2.1 ℓ	3.6 ℓ
Continuous with lateral and torsional restraint (refer to Fig.2.10)	Free	1.0 ℓ	2.5 ℓ
	Laterally restrained on top flange only	0.9 ℓ	2.5 ℓ
	Torsionally restrained only	0.8	1.5 ℓ
	Laterally and torsionally restrained	0.7 ℓ	1.2 ℓ
Built- in laterally and torsionally (refer to Fig.2.11)	Free	0.8 ℓ	1.4 ℓ
	Lateral restraint on top flange only	0.7 ℓ	1.4 ℓ
	Torsionally restrained only	0.6 ℓ	0.6 ℓ
	Laterally and torsionally restrained	0.5 ℓ	0.5 ℓ

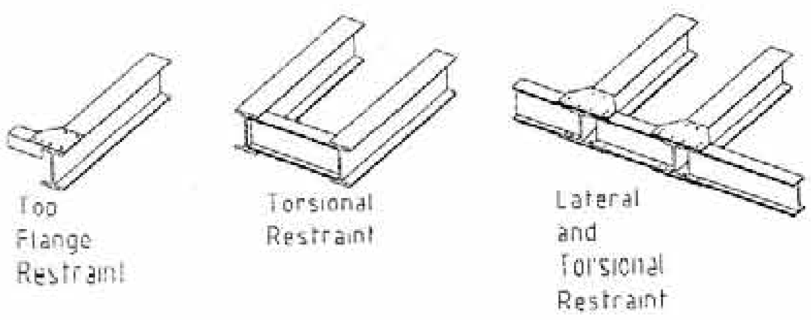


Figure 2.8 Type of Restraint Provided to the Cantilever Tip

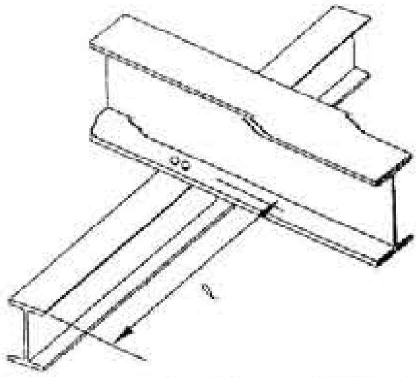


Figure 2.9 Continuous Cantilever With Lateral Restraint Only

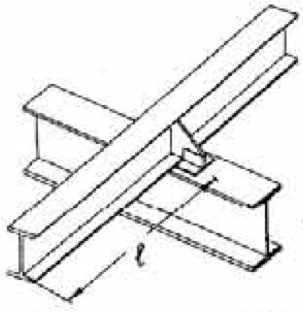


Figure 2.10 Continuous Cantilevers with Lateral and Torsional Restraint

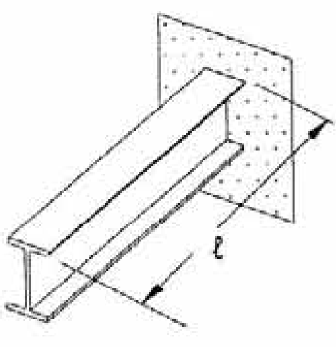


Figure 2.11 Cantilever Built-In Laterally and Torsionally

2.2.2 Second Order Effect

Second order P-δ and P-Δ (Fig. 2.12) effects shall be considered in the design of members subjected to combined axial compression and simple bending. The required axial strength P_u and flexural strength M_u must account for the elastic second-order effects. P_u and M_u can be calculated directly from a second-order elastic analysis or approximately from the superposition of two first-order elastic analyses (Fig. 2.13) according to the following procedure.

$$M_u = B_1 M_{nt} + B_2 M_{lt} \quad \text{.......... 2.8a}$$
$$P_u = P_{nt} + B_2 P_{lt} \quad \text{.......... 2.8b}$$

Where:

M_{nt} = required flexural strength in member assuming there is no lateral translation of the frame as shown in Fig. 2.13b, m.ton

M_{lt} = required flexural strength in member as a result of lateral translation of the frame only as shown in Fig. 2.13c, m.ton

P_{nt} = required axial strength in member as a result of lateral translation of the frame only as shown in Fig. 2.13b, ton

P_{lt} = required axial strength in member as a result of lateral translation of the frame only as shown in Fig. 2.13c, ton

The factor B_1 (defined by Eq. 2.9) is required to estimate the P-δ effects on the nonsway moments, M_{nt} , in axially loaded members, while the factor B_2 (defined by eq. 2.12 or 2.13) is required to estimate the P-Δ effect in frame components of unbraced, moment, M_{lt}, and/or combined framing systems. The P-δ and P-Δ effects are shown graphically in Fig. 2.12 for a beam column.

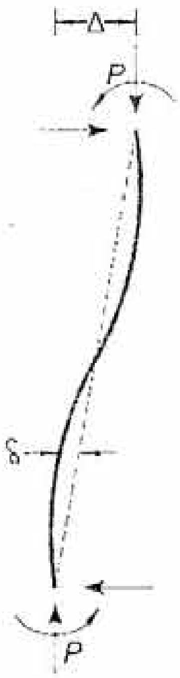


Fig. 2.12 P-δ and P-Δ effects in beams-columns

$$B_1 = \frac{C_m}{1 - \frac{P_u}{P_{e_1}}} \geq 1.0 \quad \text{......} \quad 2.9$$

$$P_{e_1} = \frac{\pi^2 E I}{(KL)^2} \quad \text{......} \quad 2.10$$

Where:

K = ≤ 1.0, calculated in the plane of bending for the nt-case (braced frame)

C_m = moment modification factor for nt-case, and is to be taken according to the following:

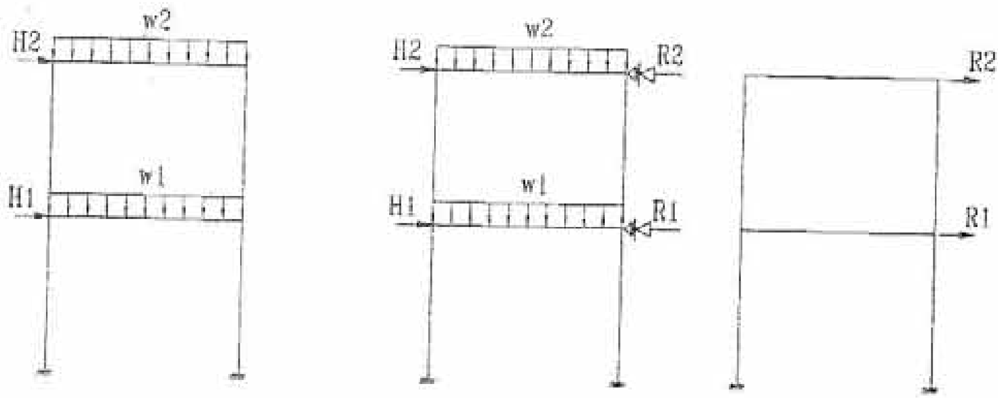
- For beam-columns without transverse loading between their ends in the plane of bending,

  $C_m = 0.6 - 0.4\ (M_1/M_2)$ 2.11

  Where:
  $M_2 > M_1$; and the end moments M_1 and M_2 carry a sign in accordance with the end rotational direction; i.e., positive moment ratio for reverse curvature and negative moment ratio for single curvature.
- For beam-columns with transverse loading between their ends, C_m may be taken as:

a- For members with moment restraint at the ends, $C_m = 0.85$

b- For members with simply supported ends, $C_m = 1.0$



Original Frame (a) = Non-Sway Frame (nt) Analysis (b) + Sway Frame (lt) Analysis (c)

Figure 2.13 Approximate Determinations of Second Order Elastic Moments

$$B_2 = \frac{1}{1 - \left(\frac{\sum P_u}{\sum H}\right)\left(\frac{\Delta_{oh}}{L}\right)} \geq 1.0 \quad \text{.......... 2.12}$$

OR

$$B_2 = \frac{1}{1 - \left(\frac{\sum P_u}{\sum P_{e_2}}\right)} \geq 1.0 \quad \text{.......... 2.13}$$

Where:

$\sum P_u$ = required axial strength of all columns in a storey, i.e. the total factored gravity load above that level, tons

Δ_{oh} = lateral inter-storey sidesway, cm

$\sum H$ = sum of all story horizontal forces producing Δ_{oh}, tons

L = story height, cm

$$P_{e_2} = \frac{\pi^2 E\, I}{(KL)^2} \quad \text{.......... 2.14}$$

Where K ≥1.0, calculated in the plane of bending for the Δ-case (unbraced frame).

P_{e2} should be calculated for columns which have contribution to the story side sway, therefore, contributions from leaning columns should not be included in this summation.

A rigorous second-order elastic analysis is recommended for accurate determination of the frame internal forces when B1 is larger than about 1.2 or B2 is larger than about 1.5.

In structures designed on the basis of plastic analysis, the required flexural strength, M_u, shall be determined from the second-order plastic analysis that satisfies the requirements of Section 2.3.2. In structures designed on the basis of elastic analysis, M_u for beam-columns, connections, and connected members shall be determined from the second-order elastic analysis or from the following approximate second-order analysis procedure:

a- For beam-columns with transverse loadings, simply-supported at both ends, the second order moment can be approximated by the following equation:

$$C_m = 1 + \omega\left(\frac{P_u}{P_{el}}\right) \qquad \text{.......... 2.15}$$

Where:

$$\omega = \frac{\pi^2 \delta_o EI}{M_o L^2} - 1 \qquad \text{.......... 2.16}$$

Where:

δ_o = maximum deflection due to transverse loading, cm

M_o = maximum factored design moment between supports due to transverse loading, t.cm

b- For beam-columns with transverse loading and restrained ends, limiting cases are shown in Table 2-11. The values of C_m are always used with the maximum moment in the member. A more conservative value may be used for transversely loaded members with unrestrained ends, for C_m = 1.0 and for restrained ends C_m = 0.85.

Table 2.11 Values of ω and C_m for Various Cases of Loading and End-Restraint

Case	ω	C_m
Hinged-hinged distributed load	0	1.0
Fixed-hinged distributed load	- 0.4	$1-0.4\left(\frac{P_u}{P_{e_1}}\right)$
Fixed-fixed distributed load	- 0.4	$1-0.4\left(\frac{P_u}{P_{e_1}}\right)$
Hinged-hinged concentrated load	- 0.2	$1-0.2\left(\frac{P_u}{P_{e_1}}\right)$
Fixed-hinged concentrated load	-0.3	$1-0.3\left(\frac{P_u}{P_{e_1}}\right)$
Fixed-fixed concentrated load	-0.2	$1-0.2\left(\frac{P_u}{P_{e_1}}\right)$

2.3 LOCAL BUCKLING

2.3.1 Classification of Steel Sections

Structural sections (other than cold-formed sections - covered by Chapter 13) shall be classified depending on the maximum width-thickness ratios of their elements subjected to compression as follows:

2.3.1.1 Class 1 (Compact sections)

Those sections that can achieve plastic moment capacity without local buckling of any of their compression elements. Compact sections are capable of developing a fully plastic stress distribution before the onset of local buckling. For a section to qualify as compact, its flanges must be continuously connected to the web or webs. The limiting width to thickness ratios (λ_p) of compression elements are given in Table 2.12.

2.3.1.2 Class 2 (Non-compact sections)

Those sections that can achieve yield moment capacity without local buckling of any of their compression elements. The limiting width to thickness ratios (λ_r) of compression elements are given in Table 2.12. It should be noted that for tapered flanges of rolled sections, the thickness is the nominal value halfway between the free edge and the corresponding face of the web.

Table 2.11 Values of ω and C_m for Various Cases of Loading and End-Restraint

Case	ω	C_m
Hinged-hinged distributed load	0	1.0
Fixed-hinged distributed load	- 0.4	$1-0.4\left(\frac{P_u}{P_{e_1}}\right)$
Fixed-fixed distributed load	- 0.4	$1-0.4\left(\frac{P_u}{P_{e_1}}\right)$
Hinged-hinged concentrated load	- 0.2	$1-0.2\left(\frac{P_u}{P_{e_1}}\right)$
Fixed-hinged concentrated load	-0.3	$1-0.3\left(\frac{P_u}{P_{e_1}}\right)$
Fixed-fixed concentrated load	-0.2	$1-0.2\left(\frac{P_u}{P_{e_1}}\right)$

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Those sections that can achieve yield moment capacity without local buckling of any of their compression elements. The limiting width to thickness ratios (λ_r) of compression elements are given in Table 2.12. It should be noted that for tapered flanges of rolled sections, the thickness is the nominal value halfway between the free edge and the corresponding face of the web.

2.3.1.3 Class 3 (Slender sections)

Those sections that cannot achieve yield moment capacity without local buckling of any of its compression elements. When any of the compression elements of a cross-section is classified as class 3, the whole cross section shall be designed as class 3 cross section.

Slender sections (which do not meet the non- compact section requirements of Table 2.12) shall be designed same as non-compact sections except that the section properties used in design shall be based on the effective widths b_e of compression elements as specified in Table 2.13 for stiffened elements and Table 2.14 for unstiffened elements. The effective width is calculated using a reduction factor ρ as $b_e = \rho\ \bar{b}$

Where, for stiffened members:

$$\rho = (1.1\,\overline{\lambda_p} - 0.16 - 0.1\psi)/\,\overline{\lambda_p}^{\,2} \quad \leq 1 \qquad \text{2.17}$$

and

$\overline{\lambda_p}$ = normalized plate slenderness given by:

$$\overline{\lambda_p} = \frac{\bar{b}/t}{44}\left[\sqrt{F_y/K_\sigma}\right] \qquad \text{2.18}$$

and for unstiffened members:

$$\rho = (\overline{\lambda_p} - 0.15 - 0.05\psi)/\,\overline{\lambda_p}^{\,2} \quad \leq 1 \qquad \text{2.19}$$

and

$$\overline{\lambda_p} = \frac{\bar{b}/t}{59}\left[\sqrt{F_y/K_\sigma}\right] \qquad \text{2.20}$$

K_σ = plate buckling factor which depends on the stress ratio ψ as shown in Tables 2.13 and 2.14

$\bar{b}$ = appropriate width, (refer to Table 2.12) as follows
= d_w for webs
= b for internal flange elements (except rectangular hollow sections)
= b-$3t$ for flanges of rectangular hollow sections
= c for out standing flanges
= b for equal leg angles
= b or $(b + h)/2$ for unequal leg angles
= b for stem of T-section

t = relevant thickness

Refer to Chapter 13 for members made of cold formed sections with their components generally having flat slender thin plates. The individual plate elements are classified as stiffened, unstiffened and multiple stiffened elements depending on the stiffening arrangement provided. The effective design width for compression cold formed elements with edge stiffeners or multiple stiffened elements and the stiffener requirements are detailed in Chapter 13.

For the flexural design of cold-formed I-shaped sections, channels, rectangular and/or circular sections and other shapes or members in axial compression that have slender compression cold formed elements, refer to Chapter 13. For plate girders with slender web elements, refer to Chapter 6.

2.3.2 Design by Plastic Analysis

Design by plastic analysis is permitted when flanges subject to compression involving hinge rotation and all webs have a width-thickness ratio less than or equal to the limiting value for compact sections given in Table 2.12. Design by plastic analysis is subject to the limitations in Section 1.5.2.

2.4 BRACING AT SUPPORTS

At points of support for beams, girders and trusses, restraint against rotation about their longitudinal axis shall be provided.

2.5 END RESTRAINT

When designed on the assumption of full or partial end restraint due to continuous, semi-continuous, or cantilever action, the beams, girders, and trusses, as well as the sections of the members to which they connect, shall be designed to carry the factored forces and moments that are introduced, as well as all other factored forces, without exceeding the design strengths prescribed in this code, except that some inelastic but self-limiting deformation of a part of the connection is permitted.

2.6 MINIMUM THICKNESS OF PLATES

The minimum thickness (in mm) to be used in structural steelwork (except cold-formed steel sections) shall be as follows:

For plates= 5mm

For gusset plates of main trusses = 8mm

An addition shall be made to the sectional areas required to resist the computed stress, so as to allow for corrosion, when climate influences or other conditions may set up such a corrosion or when the steelwork is not accessible for painting on both sides. In such cases, the minimum thickness as given above shall be increased by at least 1 mm.

components generally having flat slender thin plates. The individual plate elements are classified as stiffened, unstiffened and multiple stiffened elements depending on the stiffening arrangement provided. The effective design width for compression cold formed elements with edge stiffeners or multiple stiffened elements and the stiffener requirements are detailed in Chapter 13.

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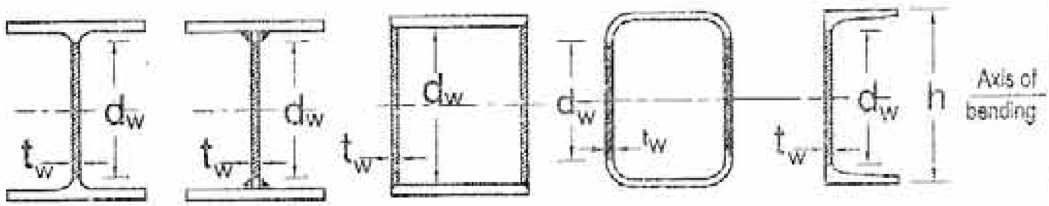
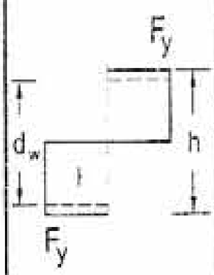
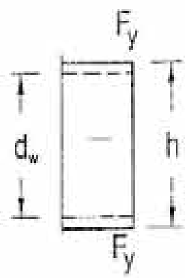
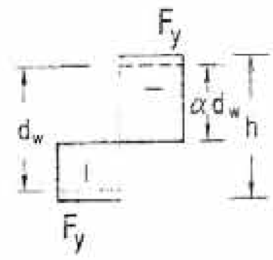
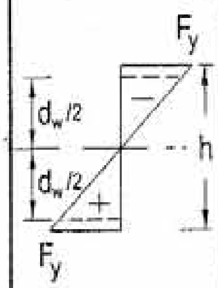
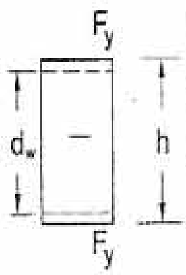
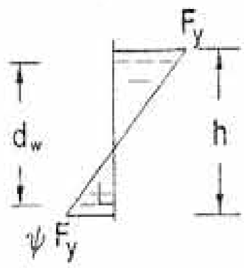
Table 2.12a Maximum Width to Thickness Ratios for Stiffened Compression Elements

(a) Webs: (Internal elements perpendicular to axis of bending)

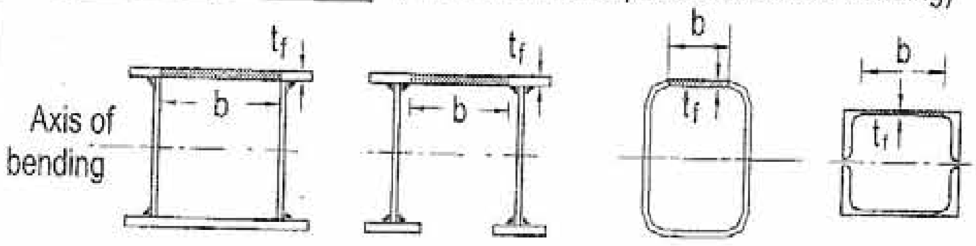


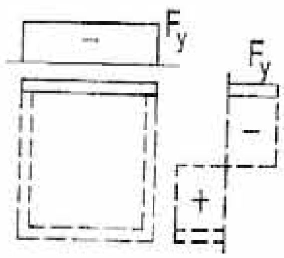
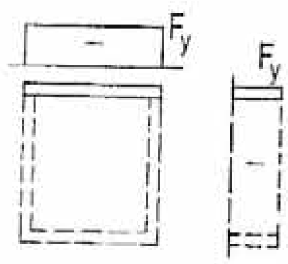
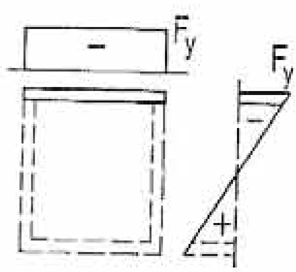
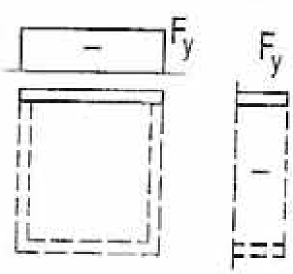
Class / Type	Web Subject to Bending	Web Subject to Compression	Web Subject to Bending and Compression	
1. Compact Stress distribution in element.				
Not for single channel (λ_p)	$\alpha = 0.5$	$\alpha = 1.0$	$\alpha > 0.5$	$\alpha \leqslant 0.5$
	$\frac{d_w}{t_w} \leqslant \frac{127}{\sqrt{F_y}}$	$\frac{d_w}{t_w} \leqslant \frac{58}{\sqrt{F_y}}$	$\frac{d_w}{t_w} \leqslant \frac{699/\sqrt{F_y}}{13\alpha - 1}$	$\frac{d_w}{t_w} \leqslant \frac{63.6/\alpha}{\sqrt{F_y}}$
2.Non-Compact Stress distribution in element. (λ_r)				
	$\psi = -1$	$\psi = 1$	$\psi > -1$	$\psi \leqslant -1$
	$\frac{d_w}{t_w} \leqslant \frac{222}{\sqrt{F_y}}$	$\frac{d_w}{t_w} \leqslant \frac{64}{\sqrt{F_y}}$	$\frac{d_w}{t_w} \leqslant \frac{222/\sqrt{F_y}}{2+\psi}$	$\frac{d_w}{t_w} \leqslant \frac{111(1-\psi)\sqrt{-\psi}}{\sqrt{F_y}}$

F_y in t/cm^2

Table 2.12b Maximum Width to Thickness Ratios for Stiffened Compression Elements

(b) Internal Flange Elements: (Internal elements parallel to axis of bending)

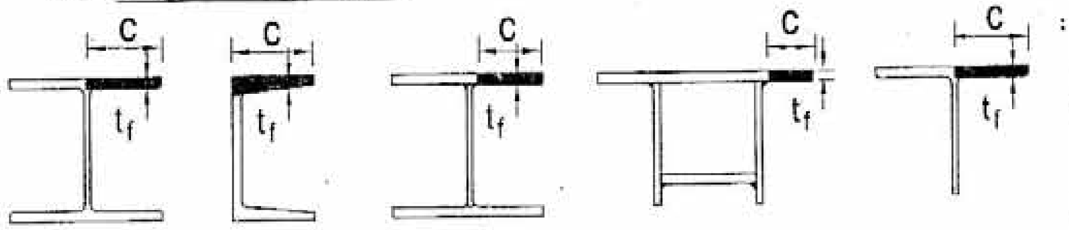


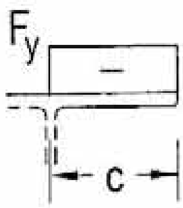
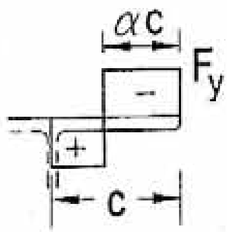
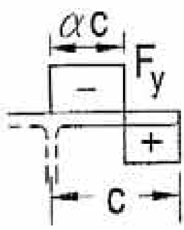
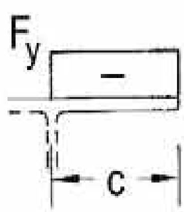
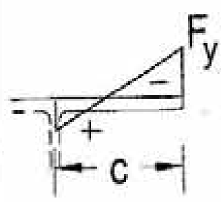
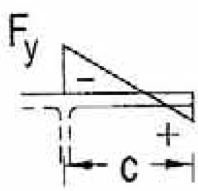
Class / Type	Section in Bending	Section in Compression
1. Compact Stess distribution in element and across section (λ_p)	 $\frac{b}{t_f} \leqslant \frac{58}{\sqrt{F_y}}$	 $\frac{b}{t_f} \leqslant \frac{58}{\sqrt{F_y}}$
2. Non-Compact Stess distribution in element and across section (λ_r)	 $\frac{b}{t_f} \leqslant \frac{64}{\sqrt{F_y}}$	 $\frac{b}{t_f} \leqslant \frac{64}{\sqrt{F_y}}$

F_y in t/cm^2

Table 2.12c Maximum Width to Thickness Ratios for Unstiffened Compression Elements

(c) Outstand Flanges

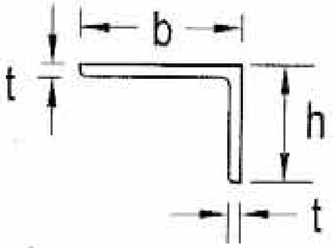
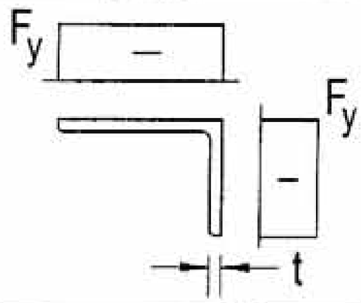
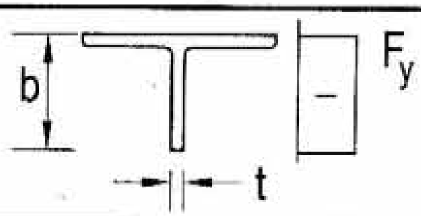
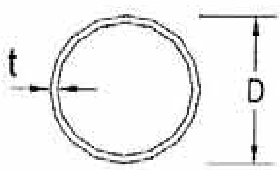


Class / Type	Flange Subject to Compression Due to M_x	Flange Subject to Compression and Bending: Tip in Comprssion	Flange Subject to Compression and Bending: Tip in Tension
1. Compact Stress distribution in element (not for single channel or T section) (λ_p)			
Rolled	$\frac{c}{t_f} \leqslant 16.9/\sqrt{F_y}$	$\frac{\alpha c}{t_f} \leqslant 16.9/\sqrt{F_y}$	$\frac{\alpha c}{t_f} \leqslant 16.9/\sqrt{\alpha F_y}$
Welded	$\frac{c}{t_f} \leqslant 15.3/\sqrt{F_y}$	$\frac{\alpha c}{t_f} \leqslant 15.3/\sqrt{F_y}$	$\frac{\alpha c}{t_f} \leqslant 15.3/\sqrt{\alpha F_y}$
2. Non-Compact Stress distribution in element (λ_r)			
Rolled	$\frac{c}{t_f} \leqslant 33/\sqrt{F_y}$	$\frac{c}{t_f} \leqslant 50\sqrt{K_\sigma/F_y}$	
Welded	$\frac{c}{t_f} \leqslant 28/\sqrt{F_y}$	$\frac{c}{t_f} \leqslant 43\sqrt{K_\sigma/F_y}$	

F_y in t/cm^2

For K_σ see Tables 2.14 & 2.15

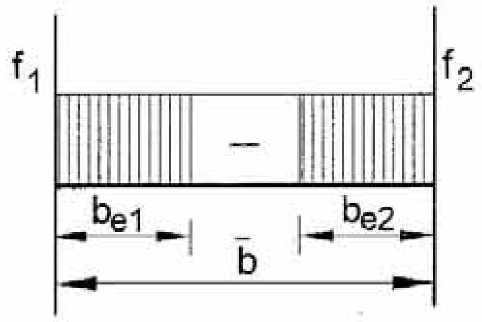
Table 2.12d Maximum Width to Thickness Ratios for Compression Elements

(d) Angles: Refer also to (Table 2.12c) "Outstand flanges"	 (Does not apply to angles in continuous contact with other components)
Class	Section In Compression
Stress distribution across section	
Non-compact (λr)	$b/t \le 23/\sqrt{F_y}$; $(b+h)/2t \le 17/\sqrt{F_y}$ (*)
Class	Section In Compression
(e) T-section:	
Non-compact (λr)	$b/t \le 30/\sqrt{F_y}$
(f) Tubular section:	
Class	Section In Bending and/or Compression
1. Compact (λp)	$D/t \le 165/F_y$
2. Non-Compact (λr)	$D/t \le 211/F_y$

F_y in t/cm^2 (*) For unequal angles

Table 2.13 Effective Width and Buckling Factor for Stiffened Compression Elements

Stress Distribution						Effective Width b_e $\rho = (\bar{\lambda}_p - 0.15 - 0.05\psi) / \bar{\lambda}_p^2 \leq 1$
For $1 > \psi \geq -1$: $k_\sigma = \dfrac{16}{[(1+\psi)^2 + 0.112(1-\psi)^2]^{0.5} + (1+\psi)}$						$-1 > \psi > -2$
$\psi = f_2 / f_1$	1	$1 > \psi > 0$	0	$0 > \psi > -1$	-1	
Buckling Factor k_σ	4.0	$\dfrac{8.2}{1.05 + \psi}$	7.81	$7.81 - 6.29\psi + 9.78\psi^2$	23.9	$5.98(1-\psi)^2$

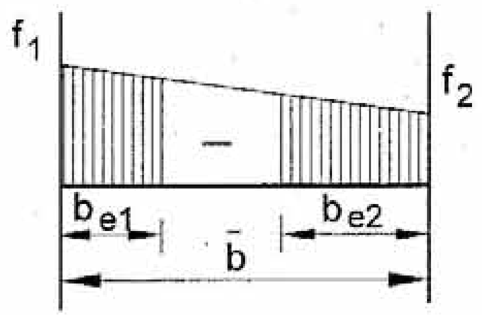


$\psi = 1$:

$b_e = \rho \bar{b}$

$b_{e_1} = 0.5\, b_e$

$b_{e_2} = 0.5\, b_e$

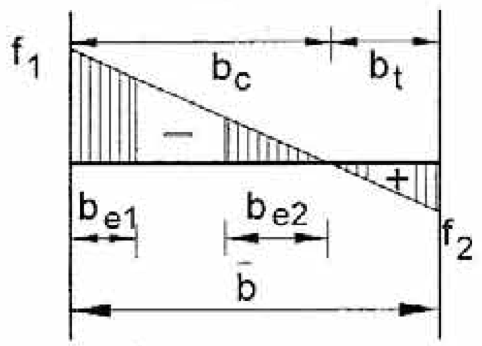


$1 > \psi \geq 0$:

$b_e = \rho \bar{b}$

$b_{e_1} = 2\, b_e / (5 - \psi)$

$b_{e_2} = b_e - b_{e1}$



$\psi < 0$:

$b_e = \rho\, b_c = \rho\, \bar{b} / (1 - \psi)$

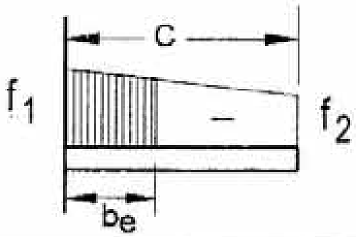
$b_{e_1} = 0.4\, b_e$

$b_{e_2} = 0.6\, b_e$

Table 2.14 Effective Width and Buckling Factor for Unstiffened Compression Elements

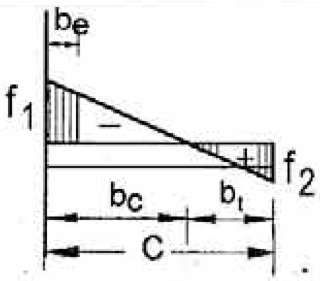
Stress Distribution	Effective Width b_e $\rho = (\bar{\lambda}_p - 0.15 - 0.05\,\psi) / \bar{\lambda}_p^2 < 1$

$\psi = f_2 / f_1$	1	$1 > \psi > 0$	0	$0 > \psi > -1$	-1
Buckling factor k_σ	0.43	$\dfrac{0.578}{\psi + 0.34}$	1.70	$1.7 - 5\,\psi + 17.1\,\psi^2$	23.8



$1 > \psi > 0$:

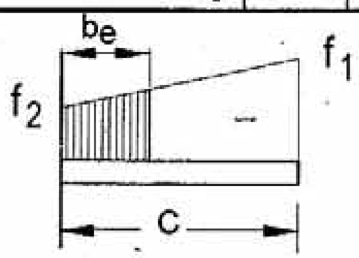
$$b_e = \rho C$$



$\psi < 0$

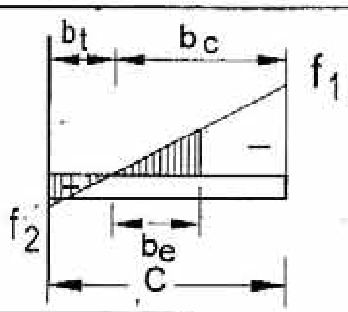
$$b_e = \rho\, b_c = \rho C / (1 - \psi)$$

$\psi = f_2 / f_1$	1	0	-1	$1 > \psi > -1$
Buckling factor k_σ	0.43	0.57	0.85	$0.57 - 0.21\,\psi + 0.07\,\psi^2$



$1 > \psi > 0$:

$$b_e = \rho C$$



$\psi < 0$:

$$b_e = \rho\, b_c = \rho C / (1 - \psi)$$

CHAPTER 3

TENSION MEMBERS

This chapter shall apply to all prismatic members subject to axial tension due to static forces acting through the centroidal axis. For members subject to combined axial tension and flexure, refer to Chapter 7. For members subject to fatigue, refer also to Chapter 11. For anchor bolts and tie rods refer to Section 8.10.6. For the design tensile strength of connecting elements (splices, gusset plates...) refer to Section 3.1.3. For single angles in tension refer to Section 3.2. For limiting slenderness ratios in tension members refer to Section 2.2.1.2.

3.1 DESIGN STRENGTH

3.1.1 Design Tensile Strength

The design strength of tension members ($\phi_t P_n$) shall be the lower value obtained according to the limit states of yielding in the gross section and fracture in the net section.

a- For yielding in the gross section

ϕ_t = resistance factor for yielding = *0.85*

$$P_n = F_y A_{gt} \qquad (3.1)$$

b- For fracture in the net section

ϕ_t = resistance factor for fracture = *0.70*

$$P_n = F_u A_e \qquad (3.2)$$

Where:

A_e = effective net area, cm^2

A_{gt} = gross area of member, cm^2

F_y = specified minimum yield stress, t/cm^2

F_u = specified minimum tensile strength, t/cm^2

P_n = nominal axial strength, t

Where members without holes are fully connected by welds, the effective net section used in Equation 3.2 shall be as defined in Section 2.3. When holes are present in a member with welded-end connections, or at the welded connection in the case of plug or slot welds, the net section through the holes shall be used in Equation 3.4.

3.1.2 Block Shear Rupture Strength

Ends of tension members shall be checked against block shear rupture according to Section 8.9.

3.1.3 Design Strength of Connecting Elements in Tension

The design strength $\phi_t\, P_n$ of welded, bolted, and connecting elements loaded in tension (e.g., splices and gusset plates) shall be the lower value obtained according to limit states of yielding, rupture of the connecting element, and block shear rupture.

a- Tension yielding of the connecting element

$\phi_t = 0.85$

$$P_n = A_{gt}\, F_y \quad \text{.......... 3.3}$$

b- Tension rupture of the connecting element

$\phi_t = 0.70$

$$P_n = A_{nt}\, F_u \quad \text{.......... 3.4}$$

Where A_{nt} = net area of the connecting element

$A_{nt} \leq 0.85\, A_{gt}$

c- Failure due to block shear rupture
Refer to Section 8.9.

3.2 SINGLE-ANGLES IN TENSION

The tensile design strength $\phi_t\, R_n$ of hot-rolled single-angle members with equal or unequal legs shall be the lower value obtained according to the limit states of

yielding,	$\phi_t = 0.85,$	$P_n = F_y\, A_{gt}$
and fracture,	$\phi_t = 0.70,$	$P_n = F_u A_e$

a- For members connected by bolting, the net area and effective net area shall be determined from Section 2.1.3.

b- When the load is transmitted by longitudinal welds only or a combination of longitudinal and transverse welds through just one leg of the angle, the effective net area A_e shall be

$$A_e = A_{gt}\, U \quad \text{.......... 3.5}$$

Where:

A_{gt} = gross area of member subjected to tension, cm^2

U = reduction factor = $1-(\bar{x}/\ell) \leq 0.9$

$\bar{x}$ = connection eccentricity, cm

ℓ = length of connection in the direction of loading, cm

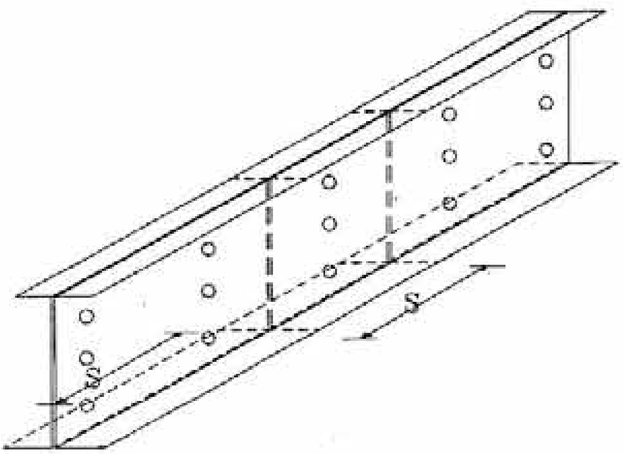
c- When a load is transmitted by transverse weld only through just one leg of the angle, A_e is the area of the connected leg and $U = 1$.

Members made of single angles shall have connections proportioned such that U > 0.6. Alternatively, a lesser value of U is permitted if these tension members are designed for the effect of eccentricity.

3.3 BUILT- UP TENSION MEMBERS

The longitudinal spacing of connectors between elements in continuous contact consisting of a plate and two rolled sections or a rolled section, is limited to the dimensions given in Section 8.4.7.

Furthermore the longitudinal spacing of connectors between member components should limit the slenderness ratio in any component between the connectors to 300 (refer to Fig. 3.1).



$S / r_{component} \leq 300$

Figure 3.1 Built-up Tension Members

Perforated cover plates or tie plates without lacing can be used on the open sides of built-up tension members. Tie plates shall have a length (*b*) not less than 2/3 the distance between the lines of welds or fasteners connecting them to the components of the member (*a*). The thickness of such tie plates (*t*) shall not be less than 1/50 of the distance between these lines. The spacing of tie plates (*L*) shall be such that the slenderness ratio of any component in the length between tie plates does not exceed 300 (refer to Fig. 4.2).

3.4 PIN-CONNECTED MEMBERS AND EYEBARS

The pin diameter shall not be less than 7/8 times the eyebar body width. The pin-hole diameter shall not be more than 1.0 mm greater than the pin diameter (refer to Fig. 3.2).

For steels having a yield stress greater than 3.6 t/cm^2, the hole diameter shall not exceed five times the plate thickness and the width of the eyebar body shall be reduced accordingly.

In pin-connected members, the pin hole shall be located midway between the edges of the member in the direction normal to the applied force. The width of the plate beyond the pin hole (*b*) shall be not less than the effective width on either side of the pin hole.

In pin-connected plates other than eyebars, the minimum net area beyond the bearing end of the pin hole, parallel to the axis of the member, shall not be less than 2/3 of the net area required for strength across the pin hole.

Eye-bars shall be of uniform thickness (*t*), without reinforcement at the pin holes, and have circular heads whose periphery is concentric with the pin hole.

The radius of transition between the circular head and the eye-bar body (*R*) shall be not less than the head diameter (*H*).

The width of the body of the eye-bars (*W*) should not exceed eight times its thickness (*t*).

The thickness (*t*) of less than 13 mm is permissible only if external nuts are provided to tighten pin plates and filler plates into snug contact. The width b from the hole edge to the plate edge perpendicular to the direction of applied load shall be greater than 2/3 and, for the purpose of calculation, not more than 3/4 times the eye-bar body width.

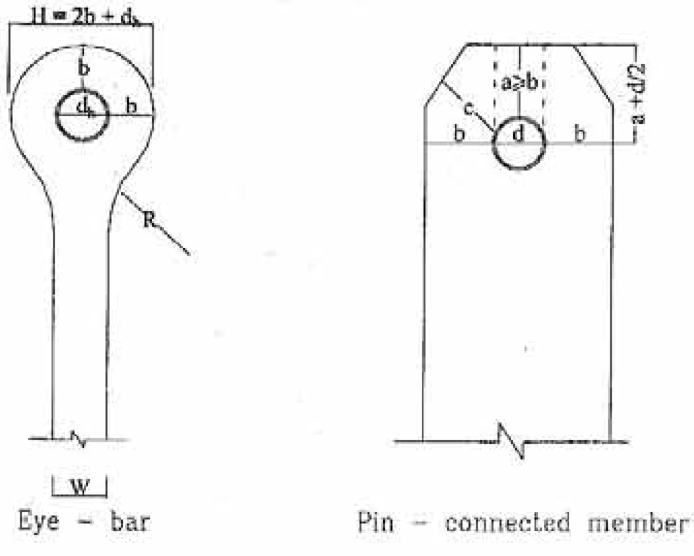


Figure 3.2 Pin Connected Members and Eyebars

The design strength of a pin-connected member ϕP_n shall be the lowest value of the following limit states:

a- Tension on the net effective area

$$\phi \;= \phi_t = 0.70$$

$$P_n = 2\, t\, b_{eff}\, F_u \qquad 3.6$$

b- Shear on the effective area

$$\phi \;= 0.70$$

$$P_n = 0.6\, A_{sf}\, F_u \qquad 3.7$$

c- For bearing on the projected area of the pin, refer to Section 8.5.3.

d- For yielding in the gross section, use Equation 3.7.

Where:

a	=	shortest distance from edge of the pin hole to the edge of the member measured parallel to the direction of the force, mm
A_{sf}	=	$2\,t\,(a + d\,/\,2)$, mm^2
b_{eff}	=	$2\,t + 16$, but not more than the actual distance from the edge of the hole to the edge of the part measured in the direction normal to the applied force, mm
d	=	pin diameter, mm
t	=	thickness of plate, mm

The corners beyond the pin hole are permitted to be cut at 45° to the axis of the member, provided the net area beyond the pin hole, on a plane perpendicular to the cut, is not less than that required beyond the pin hole parallel to the axis of the member.

The design strength of eye-bars shall be determined in accordance with Section 3.1 with A_{gt} taken as the cross-sectional area of the body.

CHAPTER 4

COMPRESSION MEMBERS

This chapter applies to compact, non-compact and slender prismatic as well as tapered members subject to axial compression through the centroidal axis. For members subject to combined axial compression and flexure, refer to Chapter 7. For cold-formed sections refer to Chapter 13.

4.1 EFFECTIVE LENGTH AND SLENDERNESS LIMITATIONS

4.1.1 Effective Length

The effective buckling length factor, K, shall be determined in accordance with Section 2.2.1.3.

4.1.2 Design by Plastic Analysis

Design by plastic analysis, as outlined in Section 2.3.2, is permitted if the column slenderness parameter λ_c does not exceed 1.1.

4.2 DESIGN COMPRESSIVE STRENGTH FOR FLEXURAL BUCKLING

4.2.1 Compact and Non-compact Doubly Symmetrical Sections

The design strength for flexural buckling of compression members whose elements have width-thickness ratio less than λ_r that is determined from Section 2.3.1.2 is $\phi_c P_n$

Where $\phi_c = 0.80$

$$P_n = A_g F_{cr} \qquad (4.1)$$

For $\lambda_c \leq 1.1$

$$F_{cr} = F_y(1-0.384\lambda_c^2) \qquad (4.2)$$

For $\lambda_c > 1.1$

$$F_{cr} = 0.648\, F_y / \lambda_c^2 \qquad (4.3)$$

The slenderness parameter λ_c is defined as

$$\lambda_c = \sqrt{\frac{F_y}{F_e}} \qquad (4.4)$$

Where:

$$F_e = \pi^2 E/(KL/r)^2 \qquad (4.5)$$

= Euler flexural buckling stress, t/cm^2

A_g = gross area of member, cm^2

F_{cr} = critical buckling stress, t/cm^2 (reflecting an initial geometric imperfection of L/750)

F_y = specified yield stress of material, t/cm^2

E = Young's modulus of elasticity, t/cm^2

L = laterally unbraced length of member, cm
K = effective buckling length factor
r = radius of gyration about the axis of buckling, cm

4.2.2 Slender Sections

This section applies to the design of axially loaded doubly symmetric members and members with plate elements having width-to-thickness ratio in excess of λ_r stipulated in section 2.3.1.2. The critical stress, F_{cr} shall de determined as follows:

For $\lambda_c\sqrt{Q} \leq 1.1$

$$F_{cr} = F_y\, Q\, (1\text{-}0.384\, Q\, \lambda_c^2) \qquad \text{.......... 4.6}$$

For $\lambda_c\sqrt{Q} > 1.1$

$$F_{cr} = 0.648\, F_y / \lambda_c^2 \qquad \text{.......... 4.7}$$

Where: Q = reduction factor for slender sections
$= A_e/A_g$
$P_n = A_g\, F_{cr}$

The gross cross-sectional area, A_g, and the radius of gyration, r, shall be computed on the basis of actual gross cross section. The effective area, A_e, used in computing the reduction factor, Q, will be determined on the basis of effective width of slender plate elements in accordance with sections 4.2.2.1 and 4.2.2.2.

4.2.2.1 Effective Width of Uniformly Compressed Unstiffened Elements

When the flat width-thickness ratio of uniformly compressed unstiffened elements (symmetric or non-symmetric) exceeds the limit λ_r stipulated in Section 2.3.1.2, a reduced effective width b_e shall be computed as follows:

For flanges, angles and plates projecting from built-up sections

$$b_e = 0.78\, t \sqrt{\frac{E}{F_y}}\, [1 - \frac{0.13}{b/t}\sqrt{\frac{E}{F_y}}] \qquad \text{.......... 4.8}$$

Where:

b = appropriate flat width (refer to Table 2.12c and d) as follows:
= C for outstanding flanges
= b for equal leg angles
= b or $(b+h)/2$ for unequal leg angles
= b for stem of T-sections
t = relevant thickness

4.2.2.2 Effective Width of Uniformly Compressed Stiffened Elements

When the flat width-thickness ratio of uniformly compressed stiffened elements (symmetric or non-symmetric) exceeds the limit λ_r stipulated in Section 2.3.1.2, a reduced effective width b_e shall be computed as follows:

a- For flanges of rectangular sections and webs of I-shaped and rectangular sections of uniform thickness

$$b_e = 1.92\, t \sqrt{\frac{E}{F_y}}\, [1 - 0.385 \sqrt{\frac{E}{F_y}} / \frac{b}{t}] \quad \text{.......... 4.9}$$

Where:

b = appropriate flat width (refer to Table 2.13) as follows:
= d_w for webs
= b for internal flange elements except for rectangular hollow sections
= b-$3t$ for rectangular hollow sections

t = relevant thickness

b- For axially loaded circular sections with diameter-to-thickness ratio, D/t, greater than 0.1 E/F_y but less than 0.45 E/F_y

$$Q = A_e/A_g$$
$$= 0.033\, E/(F_y(D/t)) + 2/3 \quad \text{.......... 4.10}$$

4.3 DESIGN COMPRESSIVE STRENGTH FOR FLEXURAL-TORSIONAL BUCKLING

4.3.1 Double Angle and Tee-Section Members

The design strength for double-angle and tee shaped compression members whose elements have width-thickness ratio less than λ_r as per Section 2.3.1.2 is $\phi_c P_n$,

Where ϕ_c = *resistance factor for compression = 0.80*

$$P_n = A_g\, F_{crn} \quad \text{.......... 4.11}$$

and

$$F_{crn} = [\frac{F_{cry} + F_{crz}}{2H}][1 - \sqrt{1 - \frac{4 F_{cry} F_{crz} H}{(F_{cry} + F_{crz})^2}}] \quad \text{.......... 4.12}$$

Where:

F_{cry} = critical flexural buckling stress about the y-axis of symmetry as per Sec. 4.2.1 for $\lambda_c = \frac{KL}{\pi r_y}\sqrt{\frac{F_y}{E}}$

$F_{crz} = GJ/Ar_o^2$ 4.13

r_o = polar radius of gyration about the shear center, cm

$$= \sqrt{y_o^2 + \frac{(I_x + I_y)}{A}}$$

$H = 1 - \{y_o^2/r_o^2\}$ 4.14

Where:

G	=	shear modulus, t/cm^2
J	=	torsional constant, cm^4
A	=	cross sectional area of the member, cm^2
I_x, I_y	=	moment of inertia about principal axes, cm^4
y_o	=	y-coordinate of the shear center with respect to the centroid, cm

T-sections conforming to the limits of Table 4.1 need not be checked for flexural-torsional buckling.

Table 4.1 Limiting Properties for T-Sections

Shape	Ratio of full flange width to profile depth	Ratio of Flange thickness to web or stem thickness
Built-up T-sections	> 0.5	> 1.25
Rolled T-sections	> 0.5	> 1.10

For double-angle and tee-shaped members whose elements have width-thickness ratio greater than λ_r refer to Sec. 4.3.3 to determine F_{cry} to be used in Equation 4.12.

4.3.2 Single Angle Members

The design strength for non-compact single angle members shall be governed by Equations 4.1, 4.2 & 4.3. The provisions of Sec. 4.3.3 may, however, be conservatively used to directly consider the flexural-torsional buckling strength for single angle members.

The design strength for slender single angle members shall be governed by Equations 4.6, 4.7, 4.8 & 4.9. The strength limit states of flexural-torsional buckling and local buckling are approximated by the reduction factor, Q, based on the effective area concept.

In case of single angles connected to gusset plates, the provisions of beam-columns of Chapter 7 shall apply to consider bending moment arising from load eccentricity. Alternatively, the computed design strength may be reduced by 40%.

4.3.3 Other Sections

The design strength for doubly symmetric, singly symmetric (except for Sec. 4.2.1 and 4.3.1) and unsymmetric compression members for the limit states of torsional and flexural-torsional buckling is $\phi_c P_n$,

Where: $\phi_c = 0.80$

$P_n = A_g F_{cr}$

The nominal critical stress F_{cr} is determined as follows:

For $\lambda_e\sqrt{Q} \leq 1.1$ $\quad F_{cr} = F_y Q (1\text{-}0.384\, Q \lambda_e^2)$ 4.15

For $\lambda_e\sqrt{Q} > 1.1$ $\quad F_{cr} = 0.648\, F_y / \lambda_e^2$ 4.16

The equivalent slenderness parameter λ_e is defined as:

$$\lambda_e = (F_y/F_e)^{1/2} \qquad 4.17$$

The critical torsional or flexural-torsional elastic buckling stress F_e is determined as follows:

a- For doubly symmetric sections:

$$F_e = [\, \pi^2 E\, C_w/(K_z L)^2 + GJ]\{1/(I_x + I_y)\} \qquad 4.18$$

b- For singly symmetric sections where y is the axis of symmetry:

$$F_e = \{ \frac{F_{ey} + F_{ez}}{2H} \}\{1 - \sqrt{1 - \frac{4 F_{ey} F_{ez} H}{(F_{ey} + F_{ez})^2}}\, \} \qquad 4.19$$

c- For un-symmetric sections, the critical flexural-torsional elastic buckling stress F_e is the lowest root of the cubic equation:

$$(F_e - F_{ex})(F_e - F_{ey})(F_e - F_{ez}) - F_e^2 (F_e - F_{ey})(x_o/r_o)^2 - F_e^2 (F_e - F_{ex})(y_o/r_o)^2 = 0 \qquad 4.20$$

and

$$F_{ex} = \pi^2 E/(K_x L/r_x)^2 \qquad 4.21$$

$$F_{ey} = \pi^2 E/(K_y L/r_y)^2 \qquad 4.22$$

$$F_{ez} = [\, \pi^2 E\, C_w/(K_z L)^2 + GJ]\{\frac{1}{A r_o^2}\} \qquad 4.23$$

Where:

Q	=	1.0 for sections with plate elements whose width-thickness ratio less than λ_r
	=	A_e/A_g for sections with plate elements whose width-thickness ratio greater than λ_r (refer to Secs.4.2.2.1 and 4.2.2.2)
K_z	=	effective length factor for torsional buckling
C_w	=	warping constant, cm^6
x_o, y_o	=	coordinates of the shear center with respect to the centroid, cm
A	=	cross-sectional area of the member, cm^2
K_x, K_y	=	effective length factors in x and y directions
r_x, r_y	=	radii of gyration about the principal axes, cm
r_o	=	polar radius of gyration about the shear center, cm
	=	$\sqrt{x_o^2 + y_o^2 + (I_x + I_y)/A}$
H	=	$1 - \{(x_o^2+y_o^2)/r_o^2\}$ 4.24

4.4 BUILT-UP MEMBERS

At the ends of built-up sections bearing on base plates or milled surfaces, all components in contact with one another shall be connected by welding having a length not less than the maximum width of the member or by bolts spaced longitudinally not more than four diameters apart for a distance equal to 1.5 times the maximum width of the member.

Along the length of built-up compression members between the end connections required above, longitudinal spacing of intermittent welds, bolts shall be adequate to provide for the transfer of the required forces. For limitations on the longitudinal spacing of connectors between elements in continuous contact consisting of a plate and a shape or two plates, refer to Sec. 8.4. Where a component of a built-up compression member consists of an outside plate, the maximum spacing shall not exceed the thickness of the thinner outside plate times $33/(F_y)^{1/2}$ nor 30 cm when intermittent welds are provided along the edges of the components or when fasteners are provided on all gauge lines at each section. When fasteners are staggered, the maximum spacing on each gauge line shall not exceed the thickness of the thinner outside plate times $50/(F_y)^{1/2}$ nor 45 cm.

4.4.1 Opened Built-Up Members

4.4.1.1 Lacing of Compression Members

As far as practicable, the lacing system shall not be varied throughout the length of the compression member. Lacing bars shall be inclined at an angle of 50° to 70° to the axis of the member where a single intersection system is used and at an angle of 40° to 50° where a double intersection system is used. Lacing bars shall be connected such that there will be no appreciable interruption of the triangulation system.

The maximum unsupported length of the compression member between lacing bars whether connected by welding or bolting shall follow the following requirements: individual components of compression members composed of two or more shapes shall be connected to one another at intervals, L_z, such that the effective slenderness ratio $K\ell_z/r_i$ of each of the component shapes, between the connectors, does not exceed 60 or two-thirds times the governing slenderness ratio of the built-up member which ever is smaller. The least radius of gyration, r_i, shall be used in computing the slenderness ratio of each component part.

The design strength of lacing bars shall be determined using the provisions of tension members (Chapter 3) and compression members (Sections 4.2 and 4.3).

The slenderness ratio KL/r_i of the single lacing shall not exceed 140. For double lacing this ratio shall not exceed 200. Double lacing bars shall be joined at the intersections. When the distance between the lines of welds or fasteners in the flanges is more than 400 mm, the lacing shall preferably be double or made of angles.

The effective length KL of lacing shall be taken as follows:

a- In bolted connections the length between the inner end bolts of the lacing bar in single intersection lacing and 0.7 of this length for double intersection lacing effectively connected at the intersection.
b- In welded connections the distance between the inner ends of effective lengths welds connecting the bars to the components in single lacing, and 0.7 of the length for double intersection lacing effectively connected at the intersection.

Laced compression members shall be provided with batten plates at the ends of the lacing system, at the points where the lacing system is interrupted, and where the member is connected to another member.

The length of end batten plate measured between end fastenings along the longitudinal axis of the member shall be not less than the perpendicular distance between the centroids of the main components, refer to Fig. 4.1.

The thickness of the lacing bars and batten plates shall not be less than 1/50 of the distance between the innermost lines of welds or bolts.

Batten plates and their fasteners shall be capable of carrying the forces for which the lacing system is designed, (considered as the actual shear plus 2% of the axial compressive force in the member under design).

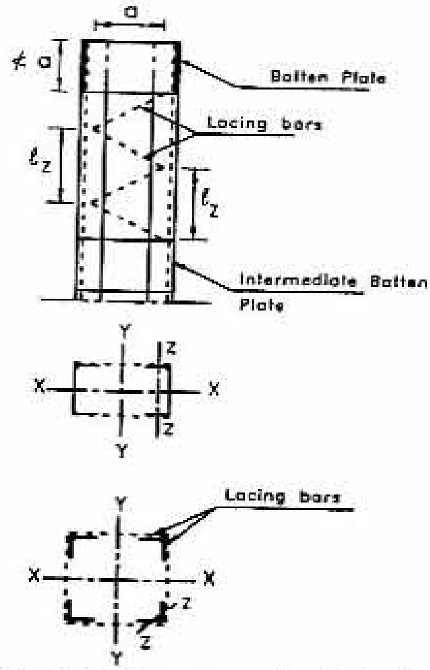


Figure 4.1 Laced Compression Members

4.4.1.2 Battening of Compression Members

The battens shall, as far as practicable, be spaced and proportioned uniformly throughout. The number of battens shall be such that the member is divided into not less than three bays within its actual center to center connections. Battens may be plates, channels or other sections.

In battened compression members, the slenderness ratio $K\ell_z/r_i$ of the main component shall not be greater than 60 or 2/3 times the maximum slenderness ratio of the member as a whole, whichever is smaller.

The member as a whole can be treated as a varendeel girder, or intermediate hinges may be assumed at mid distances to change the system into a statically determinate system. Battens and their connections shall be designed to resist simultaneously a longitudinal shear force = ($Q.d/n.a$) and a moment = ($Q.d/2n$) as shown in Fig. 4.2.

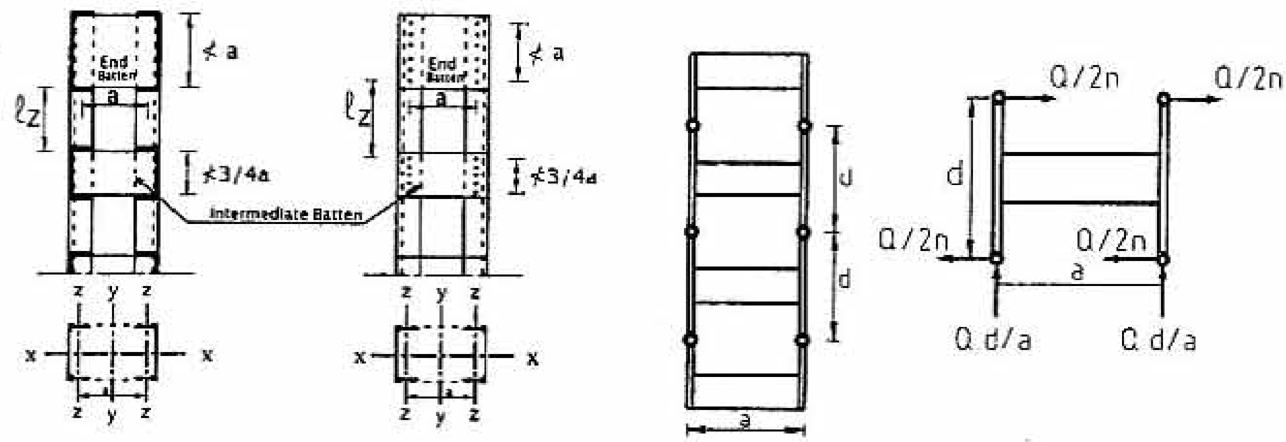
Where:

- Q = transverse shear force (considered as the actual shear plus 2% of the axial compressive force in the member under design)
- d = longitudinal distance center to center of battens
- a = minimum transverse distance between the centroids of welding or bolt groups
- n = number of parallel planes of battens

The effective length of each component of the main member between two consecutive battens parallel to the axis of the member shall be taken as the longitudinal distance between the end fasteners, L_z. End battens shall have an effective length of not less than the perpendicular distance between the centroids of the main components, and intermediate battens shall have an effective length not be less than 3/4 of this distance, refer to Fig. 4.2.

The thickness of batten plates shall be not less than 1/50 of the minimum distance between the innermost lines of connecting welds or bolts.

Battened compression members not complying with these requirements, or those subjected to bending moments in the plane of the battens, shall be designed according to the theory of elastic stability.



Welded **Bolted**

Figure 4.2 Battened Compression Members

4.4.1.3 Design Strength of Latticed and Battened Compression Members

The design strength of opened built-up members shall be determined in accordance with sections 4.2 and 4.3 except for the following modifications. If the buckling mode involves relative deformations that produce shear forces in the connectors between individual shapes, (KL/r) is replaced by $(KL/r)_m$ determined as follows:

a- For members with lacing bars and batten plates at their ends:

$$(KL/r)_m = \sqrt{(KL/r)^2 + (\ell_z/r_i)^2} \quad \text{.......... 4.25}$$

b- For members with batten plates only:

$$(KL/r)_m = \sqrt{(KL/r)^2 + (1.25\,\ell_z/r_i)^2} \quad \text{.......... 4.26}$$

Where:

ℓ_z = unsupported length of each separate part

r_i = the least radius of gyration for one part

4.5 TAPERED MEMBERS

In order to qualify under this section, the following conditions should be met in a tapered member

a- It shall possess at least one axis of symmetry.
b- The flanges shall be identical and prismatic.
c- The depth of the section shall vary linearly according to the relation:

$$d = d_o \{ 1 + \gamma_c z/L \} \qquad 4.27$$

Where:

d_o = depth at the smaller end of the member, cm

d_L = depth at the larger end of the member, cm

γ_c = $(d_L-d_o)/d_o \leq$ the smaller of $0.268(L/d_o)$ or 6.0

z = distance from the smaller end of the member, cm

L = unbraced length of the member measured between the center of gravity of the bracing members, cm

The design strength of tapered compression members shall be determined in accordance with Section 4.2, using an effective slenderness parameter, λ_{eff}, computed as follows:

$$\lambda_{eff} = \frac{S}{\pi}\sqrt{\frac{F_y}{E}} \qquad 4.28$$

Where:

S = KL/r_{oy} for weak axis buckling and $K_\gamma L/r_{ox}$ for strong axis buckling

K = effective buckling length factor for a prismatic member

K_γ = effective buckling length factor for a tapered member as determined by a rational analysis

r_{ox} = strong axis radius of gyration at the smaller end of a tapered member, cm

r_{oy} = weak axis radius of gyration at the smaller end of a tapered member, cm

The smallest area of the tapered member shall be used for A_g in Equation 4.1

4.6 PINNED CONNECTED COMPRESSION MEMBERS

Pin connections of pin-connected compression members shall conform to the requirements of Chapter 3 considering only bearing on the projected area of the pin and gross section yielding limit states.

CHAPTER 5

FLEXURAL MEMBERS

Flexural members are the members subjected to simple bending or biaxial bending about the section principal axes and associated with shearing forces. This chapter applies to flexural compact, non-compact, and slender prismatic members having $(h/t_w) \leq \lambda_r$ subjected to simple bending about one principal axis. For flexural members subjected to biaxial bending, refer to Chapter 7. For flexural members with slender web elements (plate girders), refer to chapter 6. For members subjected to fatigue, refer also to Chapter 11.

5.1 DESIGN FOR FLEXURE

For flexural members, the nominal flexural strength M_n is the lowest value obtained according to the limit states of:

a- yielding,
b- lateral-torsional buckling,
c- flange local buckling, and
d- web local buckling.

The lateral-torsional buckling limit state is not applicable to members subject to bending about the minor axis or to square or circular shapes.

This section applies to homogeneous and hybrid sections with at least one axis of symmetry. For simple bending, the beam is loaded in a plane parallel to principal axis that passes through the shear center or the beam is restrained against twisting at load points and supports. The lateral-torsional buckling provisions are limited to doubly symmetric shapes, channels, double angles, and tees, otherwise lateral torsional buckling analysis shall be performed.

5.1.1 Bending Coefficient

Bending coefficient C_b is a modification factor for beams subjected to non-uniform moment, where both ends of the beam segment are braced:

$$C_b = 1.75 + 1.05\left(\frac{M_1}{M_2}\right) + 0.3\left(\frac{M_1}{M_2}\right)^2 \leq 2.3 \quad \text{.................................. 5.1}$$

Where (M_1/M_2) are the algebraic ratio of the smaller moment to the larger moment at the braced ends of the beam segments considering positive sign for reverse curvature. Equation 5.1 is valid for straight line moment diagrams within the unbraced length. When bending moment at any point within the unbraced length is larger than the values at both ends of the beam segment, the value of C_b shall be conservatively taken as unity or more accurately defined by the following equation:

$$C_b = \frac{12.5M_2}{2.5M_2 + 3M_a + 4M_b + 3M_c} \quad \text{.......... 5.2}$$

Where:

M_2 = absolute value of maximum moment in the unbraced segment, cm.t
M_a = absolute value of moment at quarter point of the unbraced segment, cm.t
M_b = absolute value of moment at centerline of the unbraced beam segment, cm.t
M_c = absolute value of moment at three-quarter point of the unbraced beam segment, cm.t

For cantilevers or overhanging where the free ends is unbraced, C_b=1.0

5.1.2 Flexural Resistance Factor

The flexural design strength of beams shall be taken as the product of the flexural resistance factor (ϕ_b) and the nominal flexural strength (M_n); i.e. ($\phi_b\ M_n$). The flexural resistance factor shall be taken for all limit states as follows:

$$\phi_b = 0.85$$

5.1.3 Nominal Flexural Strength for Members

5.1.3.1 Compact Sections with ($\lambda \leq \lambda_p$)

The nominal flexural strength M_n shall depend on the lateral unbraced length of the member (L_b) as follows:

a- $L_b \leq L_p$

$$M_n = M_p \quad \text{.......... 5.3}$$

Where:

M_p = plastic moment (= $F_y Z \leq 1.5\ M_y$ for homogeneous I-sections), cm.t
M_y = yield moment = moment corresponding to onset of yielding at the extreme fiber from an elastic stress distribution (= $F_y S$ for homogeneous section and $F_{yf} S$ for hybrid sections), cm.t
Z = plastic section modulus about relevant principal axis, cm^3
S = elastic section modulus about relevant principal axis, cm^3
L_b = distance between points braced against lateral displacement of the compression flange, or between points braced to prevent twist of the cross sections, cm
L_p = limiting laterally unbraced length for full plastic bending capacity as defined below, cm
F_y = yield stress, t/cm^2
F_{yf} = yield stress of flange, t/cm^2

λ = controlling slenderness parameters, refer to Table 2.13, i.e. maximum of: flange width-thickness ratio for flange local buckling, or web depth- thickness ratio for web local buckling

λ_p = limiting width-thickness ratio for compact flange, or limiting depth-thickness ratio for compact web as defined in Chapter 2

i- For I–shaped members including hybrid sections and channels about major axis:

$$L_p = \frac{80 r_y}{\sqrt{F_{yf}}} \quad \text{.......... 5.4}$$

ii- For solid rectangular bars and box sections :

$$L_p = \frac{273 r_y}{M_p}\sqrt{JA} \quad \text{.......... 5.5}$$

Where:

r_y = radius of gyration about minor principal axis, cm

A = cross sectional area, cm ^{2}

J = torsional constant, cm ^{4}

b- $L_p < L_b \leq L_r$

$$M_n = \left[M_p - (M_p - M_r)\left\{\frac{L_b - L_p}{L_r - L_p}\right\}\right] C_b \leq M_p \quad \text{.......... 5.6}$$

Where:

M_r = limiting buckling moment, as defined below, t.cm

L_r = limiting laterally unbraced length for inelastic lateral torsional buckling, as defined below, cm

i- For doubly symmetric I-shaped members and channels about major axis:

M_r = $F_L S_x$ For bending about major axis, and

$$L_r = \frac{1380 A_f}{dF_L}\sqrt{\frac{I}{2}\left(1 + \sqrt{1 + (2XF_L)^2}\right)} \quad \text{.......... 5.7}$$

$$X = \left(\frac{0.104 r_T d}{A_f}\right)^2 \quad \text{.......... 5.8}$$

Where:

S_x = elastic section modulus about major axis, cm^3

F_L = $0.75 F_y$ for rolled sections, t/cm^2

= smaller of $0.60(F_{yf}$ or $F_{yw})$ for built-up sections, t/cm^2

F_{yw} = yield stress of web, t/cm^2

r_T = radius of gyration about the minor axis of a section comprising the compression flange plus one sixth of the web area, cm

A_f = area of compression flange, cm^2

d = total depth of the beam, cm

ii- For solid rectangular bars and box sections:

$$L_r = \frac{4200 r_y \sqrt{JA}}{M_r} \quad \text{.......... 5.9}$$

$$M_r = F_{yf} S_x \quad \text{.......... 5.10}$$

c- $L_b > L_r$

The nominal flexural strength is:

$$M_n = C_b\ M_{cr} \leq M_p \quad \text{.......... 5.11}$$

Where M_{cr} is the critical elastic moment (cm.t), determined as follows:

i- For doubly symmetric I-shaped members and channels about major axis:

$$M_{cr} = S_x \sqrt{\left(\frac{1380 A_f}{d L_b}\right)^2 + \left(\frac{20700}{(L_b/r_T)^2}\right)^2} \leq M_p \quad \text{.......... 5.12}$$

ii- For solid rectangular bars and symmetric box sections:

$$M_{cr} = \frac{4200\sqrt{JA}}{L_b/r_y} \quad \text{.......... 5.13}$$

iii- Tees and Double Angles

For tees and double – angle beams loaded in the plane of symmetry:

$$M_{cr} = \frac{4100\sqrt{I_y J}}{L_b}\left[B + \sqrt{1 + B^2}\right] \leq M_y \quad \text{.......... 5.14}$$

Where:

$M_{cr} \leq 1.5\ M_y$ for stems in tension

$$B = \pm\ 2.3\left(d/L_b\right)\sqrt{I_y / J}$$

$M_{cr} \leq M_y$ for stems in compression

I_y = Moment of Inertia about minor axis, cm^4

The plus sign for B applies when the stem is in tension and the minus sign applies when the stem is in compression. If the tip of the stem is in compression anywhere along the unbraced length, use the negative value of B.

5.1.3.2 Non Compact Sections with $(\lambda_p<\lambda\leq\lambda_r)$

The nominal flexural strength M_n shall depend on the lateral unbraced length of the member (L_b) as follows:

a- $L_b \leq L'_p$

$$M_n = M_p \qquad \text{5.15}$$

$$M_n = \left[M_p - (M_p - M_r)\left\{\frac{\lambda-\lambda_p}{\lambda_r-\lambda_p}\right\}\right] \leq M_p \qquad \text{5.16}$$

$$L'_p = \left[L_p + (L_r - L_p)\left\{\frac{M_p - M_n}{M_p - M_r}\right\}\right] \qquad \text{5.17}$$

Where:

- M_n = nominal flexural strength, cm.t
- M_p = plastic moment as defined in equation (5.3), cm.t
- M_r = limiting buckling moment as defined in equation (5.7 and 5.10), cm.t
- λ = controlling slenderness parameters as defined in Chapter 2, the maximum of:
  flange width-thickness ratio for flange local buckling, or
  web depth-thickness ratio for web local buckling
- λ_p = limiting width-thickness ratio for compact flange or limiting depth-thickness ratio for compact web as defined in Chapter 2
- λ_r = limiting width-thickness ratio for non-compact flange or limiting depth-thickness ratio for non-compact web as defined in Chapter 2

M_n shall be computed for both limiting width-thickness ratio for flange and for depth-thickness ratio for web whichever gives smaller value for M_n.

b- $L'_p < L_b \leq L_r$

M_n shall be computed as the smaller of equation 5.6 and 5.15

c- $L_b > L_r$

M_n shall be computed using equation 5.11

5.1.3.3 Slender Sections with $(\lambda > \lambda_r)$

The nominal flexural strength M_n, for channels and doubly and singly symmetric I-shaped beams (including hybrid beams) bent about major and minor axes, shall be governed by either the web or flange local buckling or lateral torsional buckling whichever is smaller. For flange local buckling, where $\lambda \leq 30$, M_n shall be taken as the minimum of the following:

a- $$M_n = C' \frac{S_x}{\lambda^2} \quad \text{......} 5.18$$

OR,

b- M_n = as computed from equation 5.11 considering the gross sectional properties and C_b=1.

Where S_x is the gross elastic section modulus, and

$$C' = \begin{cases} 817 & For \quad rolled \quad Sections, \ t/cm^2 \\ 470 & For \quad welded \quad Sections, \ t/cm^2 \end{cases}$$

Alternatively and for other sections, provided that flange slenderness ratio $\lambda \leq 30$, the nominal flexural strength M_n, for slender sections shall be the smaller of Equation 5.12 considering the gross sectional properties and $C_b=1$ and the following:

$$M_n = F_L \, S_{eff} \quad \text{......} 5.19$$

Where S_{eff} is the effective section modulus (S_x or S_y). as stated in Chapter 2. For web local buckling, refer to Chapter 6. For flange local buckling with $\lambda > 30$, refer to Chapter 13.

5.1.4 Unbraced Length for Design by Plastic Analysis

Design by plastic analysis, as limited in Section 1.5.2, and 2.3.2 is permitted for a compact section member bent about the major axis when the laterally unbraced length L_b (cm) of the compression flange adjacent to plastic hinge locations associated with the failure mechanism does not exceed L_{pd} (cm). determined as follows:

a- For doubly symmetric and singly symmetric I-shaped members with the compression flange equal to or larger than the tension flange (including hybrid members) loaded in the plane of the web:

$$L_{pd} = \frac{[240 + 150(M_1/M_2)]}{F_y} r_y \quad \text{......} 5.20$$

Where:

F_y = specified minimum yield stress of the compression flange, t/cm^2

M_1 = smaller moment at end of unbraced length of beam, cm.t
M_2 = larger moment at end of unbraced length of beam, cm.t
r_y = radius of gyration about minor axis, cm

(M_1/M_2) is positive when moments cause reverse curvature and negative for single curvature

b- For solid rectangular bars and symmetric box beams:

$$L_{pd} = \frac{[340 + 200(M_1/M_2)]}{F_y} r_y \geq 200\, r_y \;/\; F_y \qquad 5.21$$

There is no limit on L_b for members with circular or square cross sections or for any beam bent about its minor axis.

In the region of the last hinge to form, and in regions not adjacent to a plastic hinge, the flexural design strength shall be determined in accordance with Section 5.1.3.

5.2 DESIGN FOR SHEAR

This section applies to unstiffened webs of singly or doubly symmetric beams, including hybrid beams, and channels subject to shear in the plane of the web. For the design shear strength of webs with stiffeners, see Chapter 6. For shear in the weak direction of the shapes above, pipes, and unsymmetric sections, see Chapter 7. For web panels subject to high shear, see Chapter 10. For shear strength at connections, see Chapters 8 and 9.

5.2.1 Web Area Determination

The web area A_w shall be taken as the overall depth d times the web thickness t_w.

5.2.2 Design Shear Strength

The Design shear strength of unstiffened webs, with $h/t_w \leq 260$, is $\phi_v V_n$ where:

$\phi_v = 0.85$
V_n = nominal shear strength defined as follows, tons

For $h/t_w \leq 112/\sqrt{F_{yw}}$

$$V_n = 0.6 F_{yw} A_w \qquad 5.22$$

For $112/\sqrt{F_{yw}} < h/t_w \leq 139/\sqrt{F_{yw}}$

$$V_n = 0.6 F_{yw} A_w \left(\frac{112 / \sqrt{F_{yw}}}{h/t_w} \right) \quad \text{......... 5.23}$$

For $139/\sqrt{F_{yw}} < h/t_w \leq 260$

$$V_n = A_w \left(\frac{9500}{(h/t_w)^2} \right) \quad \text{......... 5.24}$$

The general design shear strength of webs with or without stiffeners is given in Chapter 6.

The effect of all web openings on the design strength of steel and composite beams shall be determined. Adequate reinforcement shall be provided when the required strength exceeds the net strength of the member at the opening.

Where:

d = overall depth of the section, cm

h = clear distance between flanges less the fillet or corner radius for rolled shapes; and for built-up sections, the distance between adjacent lines of fasteners or clear distance between flanges when welds are used, cm, as shown in Fig. 5.1

t_w = web thickness, cm

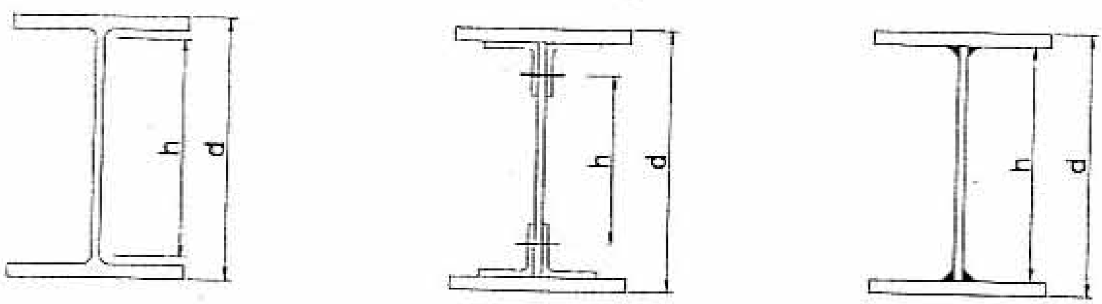


Figure 5.1 Definition of Beam Depth d and Clear Distance h

CHAPTER 6

PLATE GIRDERS

I-shaped plate girders shall be distinguished from I-shaped beams on the basis of the web slenderness ratio h/t_w, where for plate girders this ratio is greater than λ_r given by Eq. 6.1 or Eq. 6.2 below. For design flexural strength, the provisions of Section 5.1 shall apply for $h/t_w \leq \lambda_r$, while those of Section 6.2 shall apply for $h/t_w > \lambda_r$.

The design shear strength shall be based on Section 5.2 for girders without transverse stiffeners, and shall be based on Sections 6.3 and 6.4 for girders with transverse stiffeners, with or without tension-field action.

This chapter applies also to I-shaped plate girders with equal or unequal flanges, and with slender webs having $h/t_w > \lambda_r$, where:

For equal flanges:

$$\lambda_r = \frac{222}{\sqrt{F_{yw}}} \qquad \text{......6.1}$$

For unequal flanges: $$\lambda_r = \frac{58}{\sqrt{F_{yw}}}\left[1 + 2.83\left(\frac{h}{h_c}\right)\right] \qquad \text{......6.2}$$

For the application range of h/h_c given by $\frac{3}{4} \leq \frac{h}{h_c} \leq \frac{3}{2}$

Where:

F_{yw} = specified minimum yield stress of web, t/cm^2

h = distance between the two innermost lines of fasteners at the flanges, or clear distance between flanges when welds are used, cm

h_c = twice the distance from the centroid to the innermost line of fasteners at the compression flange or to the inside face of the compression flange when welds are used, cm

6.1 LIMITATIONS

Doubly and singly symmetric single-web non-hybrid and hybrid plate girders loaded in the plane of the web shall be proportioned according to the provisions of this Chapter (Section 6.2 for flexure and Section 6.3 for shear) or Section 5.2 for shear, provided that the following limits are satisfied:

- For $\frac{a}{h} \leq 1.5$: $$\frac{h}{t_w} \leq \frac{530}{\sqrt{F_{yf}}} \qquad \text{......6.3}$$

- For $\frac{a}{h} > 1.5$: $$\frac{h}{t_w} \leq \frac{986}{\sqrt{F_{yf}\left(F_{yf} + 1.16\right)}} \qquad \text{......6.4}$$

Where:

a = clear distance between transverse stiffeners, cm
t_w = web thickness, cm
F_{yf} = specified minimum yield stress of the flange, t/cm^2

In unstiffened girders, h/t_w shall not exceed 260.

6.2 DESIGN FLEXURAL STRENGTH

The design flexural strength for plate girders with slender webs shall be $\phi_b M_n$ where $\phi_b = 0.85$ and M_n is the lower value obtained according to the limit states of tension-flange yield and compression-flange buckling.

- For tension-flange yield: $M_n = S_{xt} R_e F_{yt}$ 6.5

- For compression-flange buckling: $M_n = S_{xc} R_{PG} R_e F_{cr}$ 6.6

Where:

$$R_e = \frac{12 + a_r\left(3m - m^3\right)}{12 + 2a_r} \le 1.0 \quad \text{.......... 6.7}$$

and

$$R_{PG} = 1 - \frac{a_r}{1200 + 300a_r}\left(\frac{h_c}{t_w} - \frac{222}{\sqrt{F_{cr}}}\right) \le 1.0 \quad \text{.......... 6.8}$$

Where:

F_{yt} = yield stress of tension flange, t/cm^2
F_{cr} = critical compression flange stress, t/cm^2
S_{xt} = elastic section modulus referred to tension flange, cm^3
S_{xc} = elastic section modulus referred to compression flange, cm^3
R_e = hybrid girder factor, taken =1.0 for non-hybrid girders
a_r = ratio of web area to compression flange area (≤10)
m = ratio of F_{yw} to F_{yt} (for use in Eq. 6.5) or to F_{cr} (for use in Eq. 6.6)
R_{PG} = plate girder bending strength reduction factor

The critical stress F_{cr} to be used is dependent upon the slenderness parameters λ, λ_P, λ_r, and C_{PG} t/cm^2 as follows:

For $\lambda \le \lambda_P$:

$$F_{cr} = F_{yf} \quad \text{.......... 6.9}$$

For $\lambda_P < \lambda \le \lambda_r$:

$$F_{cr} = C_b F_{yf}\left[1 - 0.4\left(\frac{\lambda - \lambda_p}{\lambda_r - \lambda_p}\right)\right] \le F_{yf} \quad \text{.......... 6.10}$$

For $\lambda > \lambda_r$:

$$F_{cr} = \frac{C_{PG}}{\lambda^2} \quad \text{.......... 6.11}$$

In the foregoing, the slenderness parameter shall be determined for both the limit state of lateral-torsional buckling and the limit state of flange local buckling; the slenderness parameter which results in the lowest value of F_{cr} governs.

a- For the limit state of lateral-torsional buckling:

$$\lambda = \frac{L_b}{r_T} \quad \text{.......... 6.12}$$

$$\lambda_p = \frac{80}{\sqrt{F_{yf}}} \quad \text{.......... 6.13}$$

$$\lambda_r = \frac{200}{\sqrt{F_{yf}}} \quad \text{.......... 6.14}$$

$$C_{PG} = 24000 C_b \quad \text{.......... 6.15}$$

Where:

L_b = as given in Section 5.1.3

C_b = as given in Section 5.1.1

r_T = radius of gyration about the y-axis of the compression flange plus one-sixth of the web, cm

b- For the limit state of flange local buckling:

$$\lambda = \frac{C}{t_f} \quad \text{.......... 6.16}$$

$$\lambda_p = \frac{15.3}{\sqrt{F_{yf}}} \quad \text{.......... 6.17}$$

$$\lambda_r = \frac{28}{\sqrt{F_{yf}}} \quad \text{.......... 6.18}$$

and for simplicity: $C_{PG} = 470$ 6.19a

or more accurately: $C_{PG} = 0.6 F_{yf} \dfrac{S_{xce}}{S_{xc}} \lambda^2$ 6.19b

where C is defined in Table 2.12c, S_{xce} is the effective elastic section modulus referred to compression flange, cm^3, and

$$C_b = 1.0$$

The limit state of flexural web local buckling is not applicable.

6.3 DESIGN SHEAR STRENGTH

The design shear strength of stiffened plate girders, where the transverse stiffeners satisfy the requirements of Section 6.4, shall be $\phi_v V_n$, where $\phi_v = 0.85$ and V_n is determined from either Section 6.3.1 or Section 6.3.2.

6.3.1 Without Tension Field Action

When tension field action is not considered in developing the web design shear strength, the transverse stiffeners shall satisfy the requirements of Section 6.4. only. V_n is determined as follows:

$$V_n = 0.6 A_w F_{yw} C_v \quad \text{......} 6.20$$

where A_w is the web area = $h\ t_w$, and C_v is the ratio of "critical" web stress, according to linear buckling theory, to the shear yield stress of web material, and C_v is determined as follows:

a- For $\dfrac{h}{t_w} \le 50\sqrt{\dfrac{k_v}{F_{yw}}}$:

$$C_v = 1.0 \quad \text{......} 6.21$$

For $50\sqrt{\dfrac{k_v}{F_{yw}}} < \dfrac{h}{t_w} \le 62\sqrt{\dfrac{k_v}{F_{yw}}}$:

$$C_v = \frac{50\sqrt{k_v / F_{yw}}}{(h/t_w)} \quad \text{......} 6.22$$

b- For $\dfrac{h}{t_w} > 62\sqrt{\dfrac{k_v}{F_{yw}}}$:

$$C_v = \frac{3100 k_v}{(h/t_w)^2 F_{yw}} \quad \text{......} 6.23$$

The web plate buckling coefficient k_v is given as:

$$k_v = 5 + \frac{5}{(a/h)^2} \quad \text{......} 6.24$$

except that k_v shall be taken as 5.0 if a/h exceeds 3.0 or $[260/(h/t_w)]^2$.

6.3.2 With Tension Field Action

When tension field action is considered in developing the web design shear strength, the transverse stiffeners must satisfy the requirements of Section 6.4.1 and Section 6.4.2. V_n is determined as follows:

6.3.2.1 Simplified Method

a- For $\dfrac{h}{t_w} \le 50\sqrt{\dfrac{k_v}{F_{yw}}}$:

$$V_n = 0.6 A_w F_{yw} \qquad \text{......} 6.25$$

b- For $\dfrac{h}{t_w} > 50\sqrt{\dfrac{k_v}{F_{yw}}}$:

$$V_n = 0.6 A_w F_{yw} \left(\frac{50\sqrt{k_v / F_{yw}}}{(h / t_w)} \right) \qquad \text{......} 6.26$$

6.3.2.2 Accurate Method

$$V_n = 0.6 A_w F_{yw} \left(C_v + \frac{1 - C_v}{1.15\sqrt{1 + (a/h)^2}} \right) \qquad \text{......} 6.27$$

Where C_v is determined from either Eq. 6.21, 6.22 or 6.23, depending on the value of h/t_w.

For end-panels in non-hybrid plate girders, all panels in hybrid and web-tapered plate girders, and when *a/h* exceeds 3.0 or $[260/(h/t_w)]^2$, tension field action is not permitted and V_n is determined according to Section 6.3.1.

6.4 TRANSVERSE STIFFENERS

Transverse stiffeners are not required in plate girders where $h/t_w \le 112/\sqrt{F_{yw}}$, or where the required shear V_u, as determined by structural analysis for the factored loads, is less than or equal to $\phi_v V_n$, where $\phi_v = 0.85$ and V_n is determined from Section 5.2.2. Stiffeners may be required in certain portions of a plate girder to develop the required shear or to satisfy the limitations given in Section 6.1. Transverse stiffeners shall satisfy the following requirements:

6.4.1 Shear Strength without or with Tension Field Action

Transverse stiffeners used to develop the web design shear strength as provided in Section 6.3.1 or Section 6.3.2, shall have a moment of inertia about an axis in the web center for stiffener pairs or about the face in contact with the web plate for single stiffeners, which shall not be less than $at_w^3 J$, Where:

$$J = [2.5/(a/h)^2] - 2 \geq 0.5 \qquad \text{6.28}$$

Intermediate stiffeners are permitted to be stopped short of the tension flange, provided bearing is not needed to transmit a concentrated load or reaction. The weld by which intermediate stiffeners are attached to the web shall be terminated not less than four times nor more than six times the web thickness from the near toe of the web-to-flange weld. When single stiffeners are used, they shall be attached to the compression flange, if it consists of a rectangular plate, to resist any uplift tendency due to torsion in the flange. When lateral bracing is attached to a stiffener, or a pair of stiffeners, these, in turn, shall be connected to the compression flange to transmit one percent of the total flange stress, unless the flange is composed only of angles.

Bolts connecting stiffeners to the girder web shall be spaced not more than 30cm on center. If intermittent fillet welds are used, the clear distance between welds shall not be more than 16 times the web thickness nor more than 25cm.

6.4.2 Shear Strength with Tension Field Action

When designing for tension field action according to Section 6.3.2, the stiffener area A_{st} shall not be less than:

$$\frac{F_{yw}}{F_{yst}}\left[0.15Dht_w(1-C_v)\frac{V_u}{\phi_v V_n} - 18t_w^2\right] \geq 0 \qquad \text{6.29}$$

Where:

F_{yst} = specified yield stress of the stiffener material, t/cm^2

D = 1.0 for stiffeners in pairs

= 1.8 for single angle stiffeners

= 2.4 for single plate stiffeners

C_v and V_n are defined in Section 6.3, and V_u is the required shear at the location of the stiffener.

6.5 FLEXURE-SHEAR INTERACTION

For $0.6\phi_v V_n \leq V_u \leq \phi_v V_n$ $(\phi_v = 0.85)$ and $0.75\phi_b M_n \leq M_u \leq \phi_b M_n$ $(\phi_b = 0.85)$, plate girders with webs designed for tension field action shall satisfy the additional flexure-shear interaction criteria:

$$\frac{M_u}{\phi_b M_n} + 0.625\frac{V_u}{\phi_v V_n} \leq 1.375 \qquad \text{6.30}$$

Where M_n is the nominal flexural strength of plate girders from Section 6.2, and V_n is the nominal shear strength from Section 6.3.

CHAPTER 7

MEMBERS UNDER COMBINED BENDING AND AXIAL FORCES

7.1 DOUBLY SYMMETRIC MEMBERS IN BENDING AND COMPRESSION

The interaction of bending and compression in doubly symmetric prismatic sections subjected to biaxial bending and compression shall be limited by Equations 7.1a and 7.1b

a- For $\dfrac{P_u}{\phi P_n} \geq 0.20$

$$\frac{P_u}{\phi P_n} + \frac{8}{9}\left[\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right] \leq 1.0 \quad \text{.......... 7.1a}$$

b- For $\dfrac{P_u}{\phi P_n} < 0.20$

$$\frac{P_u}{2\phi P_n} + \left[\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right] \leq 1.0 \quad \text{.......... 7.1b}$$

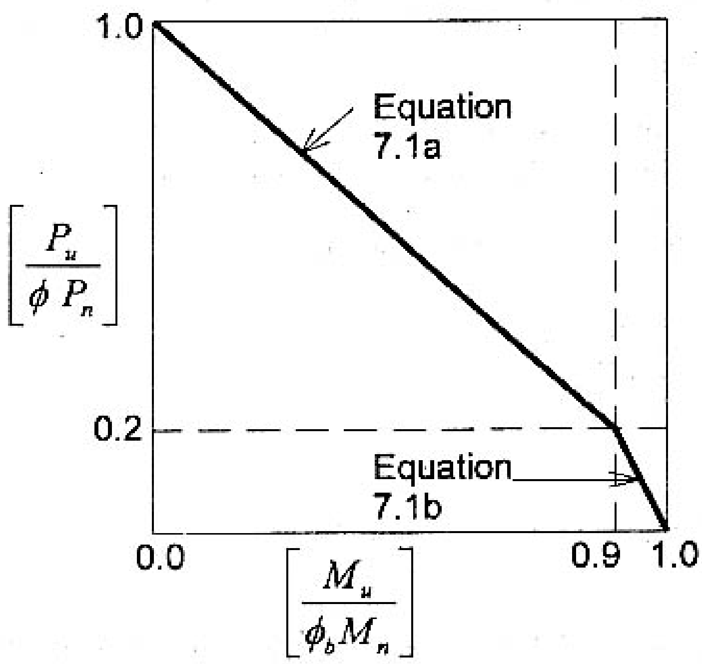


Figure 7.1 Beam Column Interaction Equations

Where:

P_u = required compressive strength, tons

P_n = nominal compressive strength determined according to Section 4.2, tons

M_u = required flexural strength including second-order (P-δ and P-Δ) effects that shall be calculated from a second-order elastic analysis or from the approximate procedure described in Section 2.2.2 cm.ton

M_n = nominal flexural strength determined according to Section 5.1, cm.ton

x = subscript relating symbol to strong axis bending

y = subscript relating symbol to weak axis bending

ϕ = ϕ_c = resistance factor for compression = 0.80

ϕ_b = resistance factor for bending = 0.85

A more advanced second-order referenced inelastic analysis of the interaction of bending and compression is permitted in lieu of Equations 7.1a and 7.1b if serviceability limits at working loads are satisfied and no plastic-hinges in any member appear before the full working loads are achieved.

7.1.1 Doubly Symmetric Members Subjected to Single Axis Bending and Compression

For doubly symmetric members in bending and compression with moments primarily in one plane, it is permissible to consider the two independent limit states, in-plane instability and out-of-plane buckling or flexural-torsional buckling, separately in lieu of the combined approach provided in section 7.1.

(a) For the limit state of in-plane instability, Equations 7.1 shall be used with ϕP_n, M_{ux}, and ϕM_{nx} determined in plane of bending.

(b) For the limit state of out-of-plane buckling

$$\frac{P_u}{\phi P_{ny}} + \left(\frac{M_{ux}}{\phi M_{nx}}\right)^2 \leq 1.0 \qquad \text{.......... 7.1c}$$

Where:

ϕP_{ny} = available compressive strength out of plane of bending, ton

ϕM_{nx} = available flexural-torsional strength for strong axis flexure.

7.2 SINGLY SYMMETRIC MEMBERS IN BENDING AND COMPRESSION

Equations 7.1a and 7.1b can still be applied for singly symmetric prismatic sections, if the area of the bigger flange is not more than twice the area of the smaller flange.

7.3 DOUBLY AND SINGLY SYMMETRIC MEMBERS IN BENDING AND TENSION

The interaction of bending and tension in doubly and singly symmetric prismatic sections shall be limited by the same Equations 7.1a and 7.1b where:

P_u = required tensile strength, tons

P_n = nominal tensile strength determined in accordance with Section 3.1, tons

ϕ = ϕ_t = resistance factor for tension = 0.85

ϕ_b = resistance factor for bending = 0.85

7.4 CALCULATION OF THE REQUIRED FLEXURAL STRENGTH

The required flexural strength M_u must account for the elastic second-order (P-δ and P-Δ) effects. M_u can be calculated directly from a second-order elastic analysis or approximately from the superposition of two first-order elastic analyses (Fig. 2.13) according to the procedure described in Section 2.2.2.

CHAPTER 8

BOLTED CONNECTIONS

8.1 MATERIAL PROPERTIES

8.1.1 Non- Pretensioned Carbon and Alloy Steel Bolts

Ordinary or high strength bolts from grades 4.6 up to and including 10.9 can be used. The nominal values of the yield stress F_{yb} and the ultimate tensile strength F_{ub} for bolts are as given in Table 8.1:

Table 8.1 Nominal Values of Yield Stress F_{yb} and Ultimate Tensile Strength F_{ub} for Bolts

	Ordinary Bolts					High Strength Bolts	
Bolt grade	4.6	4.8	5.6	5.8	6.8	8.8	10.9
F_{yb} (t/cm^2)	2.4	3.2	3.0	4.0	4.8	6.4	9.0
F_{ub} (t/cm^2)	4.0	4.0	5.0	5.0	6.0	8.0	10.0

These bolt grades are used in conjunction with structural components of steel up to St 52. Bolts of grades lower than 4.6 or higher than 10.9, shall not be utilized. Bolts of grades 4.6 up to 6.8 are made from low or mild carbon steel, and are the least expensive type of bolts for light structures. Grade 8.8 is of heat – treated high strength steel and Grade 10.9 is of heat – treated alloy steel. When subject to shear, tension or combined shear and tension the non-pretension bolts must be in accordance with section 8.5.

8.1.2 Pretensioned High Strength Bolts

High strength bolts of grade 8.8 and 10.9 are mainly used as pretensioned bolts with controlled tightening, where the forces acting transverse to the shank are transmitted either by friction (slip) or by bearing and must conform with requirements of Section 8.5. When subject to shear, tension or combined shear and tension the pretensioned high strength bolts must be in accordance with either Section 8.6 or Section 8.7. The material properties of high strength bolts of grade 8.8 and 10.9 are as previously mentioned in Table 8.1.

8.1.3 Rivets

The rivet steel is a mild carbon steel and is available in two grades namely grade 1 and grade 2 where the corresponding ultimate tensile strength (F_{ur}) is 5.0 t/cm^2 and 6.0 t/cm^2, respectively.

Structural riveting has essentially been replaced by welding and bolting. Reference to rivets in this chapter is for assessment and modifications of riveted connections in existing buildings.

8.2 CATEGORIES OF BOLTED CONNECTIONS

The design of a bolted connection shall conform to one of the following categories:

8.2.1 Category (A): Bearing Type Connections with Non-Pretensioned Bolts at Ultimate Limit State

8.2.1.1 Design Basis

In this category ordinary bolts or high strength bolts from grade 4.6 up to and including 10.9 may be used. Neither pretensioning nor special provisions for contact surfaces are required, the snug tight position is sufficient.

When subject to shear the design ultimate factored shear load shall not exceed the design shear resistance nor the design bearing resistance in accordance with Sections 8.5.2 and 8.5.3.

When subject to tension only or combined shear and tension the design resistance must be in accordance with Sections 8.5.4 and 8.5.5.

8.2.1.2 Applications and Limitations

When subject to shear the non-pretensioned bolts (snug tight position) of Grades 4.6 up to 10.9 are permitted except that outlined in section 8.2.2.2 where the use of a slip-critical connection is a must.

The non-pretensioned bolts of grade 10.9 is not permitted. Grades 4.6 up to 8.8 are permitted to be non-pretensioned except in the following connections:

- Column splices in all tall buildings 60 ms or more in height.
- Column splices in tall buildings 30 to 60 ms in height, if the least horizontal dimension is less than 40% of the height.
- Column splices in tall buildings less than 30 ms in height if the horizontal dimension is less than 25% of the height.
- Connections of all beams and girders to columns and of any other beams and girders on which bracing of columns is dependent, in structures over 36 ms height.
- In all structures, carrying cranes over five-tons capacity: roof-truss, splices and connections of trusses to columns, column splices, column bracing, knee braces and crane supports.
- Connections for supports of running machinery or of other live loads which produce impact or reversal of stress.

8.2.2 Category (B): Slip-Critical Connections with High Strength Pretensioned Bolts at Serviceability Limit State

8.2.2.1 Design Basis

In this category pretensioned high strength bolts from grades 8.8 and 10.9, with controlled tightening and surface preparation in accordance with Section 8.6.8 shall be used.

When subject to shear, slip shall not occur at the serviceability limit state (i.e. service loads) in accordance with Section 8.6.3.2

When subject to shear and tension, slip shall not occur at the serviceability limit state (i.e. service loads) in accordance with Section 8.6.5

In addition the design ultimate shear load (factored loads) shall not exceed the design shear resistance nor the design bearing resistance in accordance with Sections 8.5.2 and 8.5.3.

The design ultimate tension load (factored loads) plus the induced prying force shall not exceed the design tension resistance in accordance with Section 8.6.4.

8.2.2.2 Applications

Slip critical shear connections are required when slip would be detrimental to the serviceability of the structure. Fully tensioned slip-critical connections must be used for the following connections:

- Connections subjected to fatigue.
- Structural connections carrying cranes.
- Connections for supports of running machinery.
- Building connections when wind and earthquake are governing to warrant consideration for fatigue.

When subjected to tension only or to shear and tension the high strength bolts must be pretensioned and may be used for the following and similar connections:

- Hanger connections.
- Extended-end plate fully rigid connections.

However, as slip criteria is not detrimental the use of category (C) (section 8.2.3) is more economical for the hanger and extended end plate connections (and similar).

8.2.3 Category (C): Bearing-Type Connections with High Strength Pretensioned Bolts at Ultimate Limit State

8.2.3.1 Design Basis

Pretensioned bolts of Grades (8.8) and (10.9) may be used in bearing type connections. Fully tensioned bolts in bearing – type connections must not be confused with fully tensioned bolts in slip-critical connections. Fully tensioned bolts in bearing-type connections have no requirements regarding the slip resistance of the contact surface. Fully tensioned bolts in bearing-type connections must follow the tightening requirements of section 8.6.8.

Depending on the applied ultimate factored straining actions the corresponding design resistance must be in accordance with section 8.7.2.

8.2.3.2 Applications and Limitations

Connections previously outlined in section 8.2.2.2, where the slip-critical connection (category B) is a must, shall not be used for the bearing-type connections with pretensioned bolts.

When subjected to tension only or combined shear and tension and where the slip criteria is not detrimental, the use of the bearing type with pretensioned bolts is the economical choice. The main applications are as follows:

- Hanger connections.
- Extended fully rigid end plate connections.
- Building connections where the occurrence of full design wind or earthquake loads is not governing to warrant consideration for fatigue.

8.3 HOLES, CLEARANCES, WASHERS AND NUTS REQUIREMENTS

8.3.1 Holes

a- Holes for bolts may be drilled or punched unless specified.

b- Where drilled holes are required, they may be sub- punched and reamed.

c- Slotted holes shall either be punched in one operation, or else formed by punching or drilling, two round holes that are completed by high quality flame cutting, and dressing to ensure that the bolt can freely travel.

8.3.2 Clearances in Holes for Fasteners

a- Except for fitted bolts or where low-clearance or oversize holes are specified, the nominal clearance in standard holes shall be:

- 1 mm for M12 and M14 bolts
- 2 mm for M16 up to M24 bolts
- 3 mm for M27 and larger bolts

b- Holes with 2 mm nominal clearance may also be specified for M12 and M14 bolts provided that the design meets the requirements specified in Sections 8.4.1 and 8.4.2.

c- Unless special clearances are specified, the clearance of fitted bolts shall not exceed 0.3 mm.

d- The nominal sizes of short slotted holes shall be not greater than:

- (d + 1) mms by (d + 4) mms for M 12 and M 14 bolts
- (d + 2) mms by (d + 6) mms for M 16 to M 22 bolts
- (d + 2) mms by (d + 8) mms for M 24 bolts
- (d + 3) mms by (d + 10) mms for M 27 and larger bolts

Where d is the nominal bolt diameter.

e- The nominal sizes of long slotted holes shall be not greater than:
  - (d + 1) mms by (2.5 d) for M 12 and M 14 bolts
  - (d + 2) mms by (2.5 d) for M 16 to M 24 bolts
  - (d + 3) mms by (2.5 d) for M 27 and larger bolts

8.3.3 Nuts Constructional Precautions

a- For structures subject to vibrations, precautions shall be taken to avoid any loosening of the nuts.

b- If non- pretensioned bolts are used in structures subject to vibrations, the nuts should be secured by locking devices or other mechanical means.

c- The nuts of pretensioned bolts may be assumed to be sufficiently secured by the normal tightening procedure.

8.3.4 Washers Utilities

a- Washers may not be required for non-pretension bolts except as follows:

- A taper washer shall be used where the surface is inclined at more than 3 ° to a plane perpendicular to the bolt axis.
- Washers shall be used whenever necessary due to a requirement to use longer bolt in order to keep the bolt threads out of a shear plane or out of a fitted hole.

b- Hardened washers shall be used for pretensioned bolts under the bolt head as well as under the nut, whichever is to be rotated.

8.3.5 Tightening of Bolts

a- Non-pretension bolts shall be tightened sufficiently to ensure that sufficient contact is achieved between the connected parts.

b- It is not necessary to tighten non-pretensioned bolts to the maximum tightening value given in Section 8.6.8. However as an indication, the tightening required should be:

- Achieved by one man using a normal prodger spanner

or

- Up to the point where an impact wrench first starts to impact.

c- Pretensioned bolts shall be tightened in conformity with Section 8.6.8.

8.4 POSITIONING OF HOLES FOR BOLTS AND RIVETS

8.4.1 Basis

a- The positioning of the holes for bolts and rivets shall be done such as to prevent corrosion and local buckling, and to facilitate the installation of the bolts and rivets.

b- The positioning of the holes shall be also in conformity with the limits of validity of the rules used to determine the design bearing strength of the bolts as given in Section 8.5.2.

8.4.2 Minimum End Distance

a- The end distance (e_1) from the center of a fastener to the adjacent end of any steel element, measured in the direction of load transfer (Fig. 8.1) should not be less than 1.5*d*, where d is the nominal bolt diameter.

b- The end distance should be increased if necessary to provide adequate bearing resistance (Section 8.5.3).

8.4.3 Minimum Edge Distance

The edge distance (e_2) from the center of a fastener to the adjacent edge of any steel element, measured at right angles to the direction of load transfer (Fig. 8.1) should not be less than 1.5*d*.

8.4.4 Maximum End or Edge Distance

The maximum end or edge distance shall be 12 times the thickness (*t*) of the thinnest connected part under consideration.

8.4.5 Minimum Spacing

a- The spacing (*S*) between centers of fasteners in the direction of load transfer (Fig. 8.1) should not be less than 3*d*.

b- The spacing (*g*) between rows of fasteners, measured perpendicular to the direction of load transfer (Fig. 8.1) should normally be not less than 3*d*.

8.4.6 Maximum Spacing in Compression Members

The spacing (*S*) of the fasteners in each row and the spacing (*g*) between rows of fasteners should not exceed the lesser of 14*t* or 200 mm. Adjacent rows of fasteners may be symmetrically staggered (Fig. 8.2).

8.4.7 Maximum Spacing in Tension Members

In tension members the center – to – center spacing $S_{1,i}$ of fasteners in inner rows may be twice that given in Section 8.4.6 for compression members, provided that the spacing $S_{1,o}$ in the outer row along each edge does not exceed that given in Section 8.4.6 (Fig. 8.3).

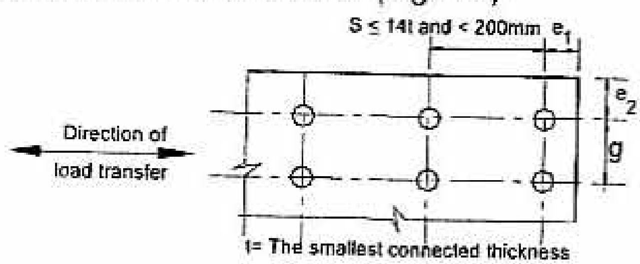


Figure 8.1 Spacing in Tension or Compression

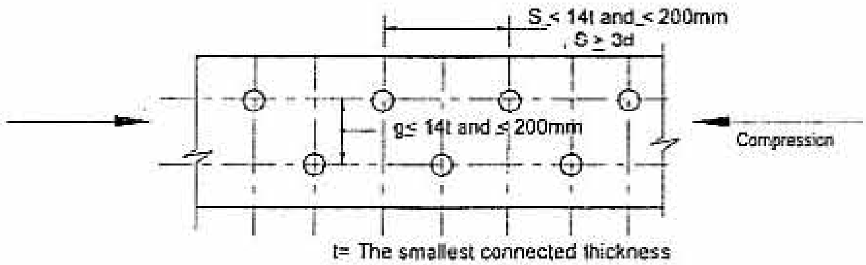


Figure 8.2 Staggered Spacing – Compression

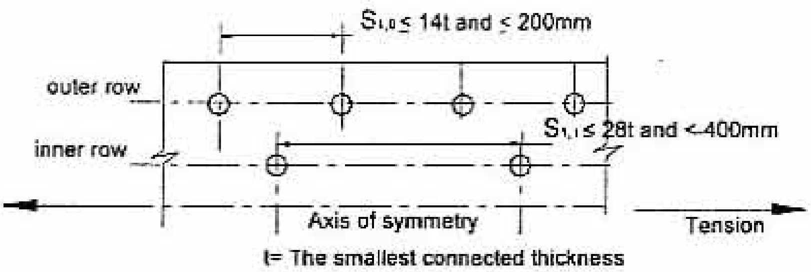


Figure 8.3 Maximum Spacing in Tension Members

8.4.8 Slotted Holes

a- The minimum distance (e_3) from the axis of a slotted hole to the adjacent end or edge of any steel element should not be less than 1.5*d* (Fig. 8.4).

b- The minimum distance (e_4) from the center of the end radius of a slotted hole to the adjacent end or edge of any steel element should not be less than 1.5*d* (Fig. 8.4).

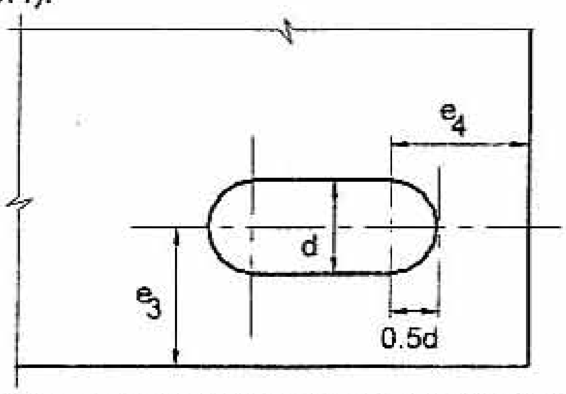


Figure 8.4 End and Edge Distances for Slotted Holes

8.5 NON-PRETENSIONED BOLTED CONNECTIONS OF THE BEARING-TYPE
CATEGORY (A) "ULTIMATE LIMIT STATE"
8.5.1 Design Basis

The load and resistance factor design concept which was previously outlined in section 1.4 and 1.5 giving the structural safety requirements for elements is also applicable for fasteners as follows:

$$\phi R_n \geq \gamma_i Q_i \qquad \text{8.1}$$

Where:

ϕ = shear, bearing or tension resistance factor

R_n = shear, bearing or tension nominal strength, tons

γ_i = load factor

Q_i = loads (such as dead load, live load, wind load, earthquake load), or straining actions. (such as bending moment, shearing force, axial force and torsional moment resulting from various loads), tons.

In this category ordinary bolts or high strength bolts from grades 4.6 up to and including grade 10.9 can be used in accordance with the limitations of Section 8.2.1.2. Neither pretensioning nor special provisions for contact surfaces are required; the snug-tight situation is sufficient for installation.

The design load shall not exceed the nominal shear strength, the nominal bearing strength and/or the nominal tension strength in accordance with Sections 8.5.2, 8.5.3, 8.5.4 and 8.5.5.

8.5.2 Design Shear Strength

a- The design shear strength ϕR_{nv} based on shear strength for bolt grades 4.6, 5.6 and 8.8 where the shear plane passes through the threaded portion of the bolt is:

$$\phi_v R_{nv} = \phi_v \ (0.6\, F_{ub})\, A_s.n \qquad \text{8.2}$$

Where:

ϕ_v = 0.6

F_{ub} = tensile strength of the bolt material as given in Table 8.1, t/cm^2

A_s = the tensile stress area of bolt, cm^2

n = number of shear planes.

b- For bolt grades 4.8, 5.8, 6.8 and 10.9 , the nominal shear strength is to be decreased according to the design shear strength given by the following relation:

$$\phi_v R_{nv} = \phi_v \ (0.5\, F_{ub})\, A_s.n \qquad \text{8.3}$$

c- For bolts where the threads are excluded from the shear planes, the gross cross sectional area of bolt (A) is to be utilized.

d- The values for the design shear strength given in Equations 8.2 and 8.3 are to be applied only where the bolt holes are with nominal clearances not exceeding those for standard holes as specified in Section 8.3.2.

e- M12 and M14 bolts may be used in 2 mms clearance holes provided that for bolts of strength grade 4.8, 5.8, 6.8 or 10.9 the design shear strength is to be decreased by 15%.

f- The shear strength using Equations 8.2 and 8.3 is to be applied also for the following cases:
   -Short slotted holes if the length of slot is normal to the direction of force.
   - Long slotted holes if the length of slot is normal to the direction of force.

g- Oversized holes shall not be used for the bearing-type connections.

h- Short and long-slotted holes where the length of slot is parallel to the direction of force are not allowed in the bearing-type connections.

8.5.3 Design Bearing Strength

a- The design bearing strength ($\phi_{br} R_{br}$) of a single bolt shall be the effective bearing area times the nominal bearing stress (αF_u) according to the following equation:

$$\phi_{br} R_{br} = \phi_{br} . d . (min \sum t . (\alpha F_u)) \qquad 8.4$$

Where:

ϕ_{br}	=	0.70
d	=	bolt shank diameter, cm
$Min \sum t$	=	smallest sum of plate thicknesses in the same direction of the force, cm
F_u	=	tensile strength of steel connected plates, t/cm^2
α	=	factor depending on the end distance = $0.8e_1/d \leq 2.40$

b- For standard holes and short-slotted holes normal to the direction of force when the end distance is not less than 1.5d, bolt spacing center-to-center not less than 3d and with two or more bolts in the line of force, the α values of Equation 8.4 are as given in Table 8.2

Table 8.2 Values of α for Different Values of End Distance (e_1) in Direction of Force

	End distance (e_1) in direction of force			
	$e_1 \geq 3d$	$2.5d \geq e_1 \geq 2.0d$	$2.0d \geq e_1 \geq 1.5d$	$e_1 = 1.5d$
α	2.4	2.0	1.6	1.2

c- For long-slotted holes perpendicular to the direction of load transfer, end distance not less than 1.5d, bolt center-to-center spacing not less than 3d, and with two or more bolts in the line of force, the α values of Table 8.2 are to be reduced by 20%.

8.5.4 Design Tension Strength

a- The design tension strength ($\phi_t R_{nt}$) based on the nominal tension strength for bolt grades 4.6, 4.8, 5.6, 5.8 and 6.8 is given by the following equation:

$$\phi_t R_{nt} = \phi_t (0.66 F_{ub}).A_s \qquad \text{.......... 8.5}$$

Where:

$$\phi_t = 0.70$$

b- For bolt grades 8.8 and 10.9, when subjected to tension, their use is not allowed in the bearing-type connections without pretensioning in accordance with section 8.2.1.2.

8.5.5 Combined Shear and Tension

When bolts are subjected to combined shear and tension, the following circular strength interaction equation is to be satisfied:

$$\left[\frac{R_{ut}}{\phi_t R_{nt}}\right]^2 + \left[\frac{R_{uv}}{\phi_v R_{nv}}\right]^2 \leq 1.0 \qquad \text{.......... 8.6}$$

Where:

R_{ut} = factored tension load on bolt, tons

R_{uv} = factored shear on bolt, tons

$\phi_t R_{nt}$ = design strength of bolt in tension alone (Equation 8.5), tons

$\phi_v R_{nv}$ = design strength of bolt in shear alone (Equations 8.2 or 8.3), tons

8.6 HIGH STRENGTH PRETENSIONED BOLTED CONNECTIONS OF THE FRICTION TYPE "SLIP-CRITICAL CONNECTIONS" CATEGORY (B) "SERVICABILITY LIMIT STATE"

8.6.1 Design Basis

In this category of connections high strength bolts of grade 8.8 and 10.9 are only to be utilized.

The bolts are inserted in clearance holes in the steel component and then pretensioned by tightening the head or the nut in accordance with section 8.6.8 where a determined torque is applied. The contact surface will be firmly clamped together around the bolt holes.

Any applied force across the shank of the bolt is transmitted by friction between the contact surfaces of the connected components. The joint here is referred to as a slip-critical joint.

Since resistance to slip in "Slip critical connections" is a limit state to be satisfied at service load, the limiting frictional load (P_s) to be used in concept is the same for load and resistance factor design as were used for the Egyptian Code of Steel Construction and Bridges (2001) - Allowable Stress Design.

The service load that must be transferred by friction without slip must not exceed the frictional resistance according to Section 8.6.3.2.

In addition, the design of slip-critical joints requires the full consideration of the ultimate strength limit states. The strength of the fasteners in shear and bearing must be provided in accordance with sections 8.5.2 and 8.5.3. based on factored loads while the strength of the fasteners, in slip-critical connection, when subject to tension the ultimate tension strength must be in accordance with section 8.6.4 based on factored loads. When subject to combined shear and tension the ultimate resistance at the ultimate serviceability state must be in accordance with section 8.6.5.

8.6.2 Design Principles of High Strength Pretensioned Bolts

8.6.2.1 The Pretension Force

The axial pretension force T produced in the bolt shank by tightening the nut or the bolt head is given by:

$$T = (0.7)\, F_{yb} A_s \quad \text{......} \quad 8.7$$

Where:

F_{yb} = yield (proof) stress of the bolt material, (Table 8.1), t/cm^2

A_s = the bolt stress area, cm^2

8.6.2.2 The Friction Coefficient of the Slip Factor

a- The friction coefficient between surfaces in contact is the dimensionless value by which the pretension force in the bolt shank is to be multiplied in order to obtain the frictional resistance P_S in the direction of the applied force.

b- The design value of the friction coefficient depends on the condition and the preparation of the surfaces to be in contact. Surface treatments are classified into three classes, where the coefficient of friction μ should be taken as follows:

μ = 0.5 for class A surfaces
μ = 0.4 for class B surfaces
μ = 0.3 for class C surfaces.

c- The friction coefficient μ of the different classes is based on the following treatments:

In class A:

- Surfaces are blasted with shot or grit with any loose rust removed, no painting.
- Surfaces are blasted with shot or grit and spray metallized with Aluminum.
- Surfaces are blasted with shot or grit and spray metallized with a Zinc based coating.

In class B:

- Surfaces are blasted with shot or grit and painted with an alkali-zinc silicate painting to produce a coating thickness of 50-80 micro-millimeter.

In class C:

- Surfaces are cleaned by wire brushing, or flame cleaning, with any loose rust removed.

d- If the coatings other than specified are utilized, tests are required to determine the friction coefficient. The tests must ensure that the creep deformation of the coating due to both the clamping force of the bolt and the service load joint shear are such that the coating will provide satisfactory performance under sustained loading.

8.6.3 Shear Strength Resistance of High Strength Pretensioned Bolts in Slip-Critical Connections

8.6.3.1 General

a- Oversized holes are allowed in any or all plies of slip-critical connections. Hardened washers shall be installed with oversized holes in an outer ply.

b- Short slotted holes are allowed in any one or all plies of slip-critical connections without regard to direction of loading. Hardened washers shall be installed over short-slotted holes in an outer ply.

c- Long slotted holes are allowed in only one of the connected parts at an individual faying surface without regard to the direction of loading. Where long-slotted holes are used in an outer ply, plate washers or a continuous bar with standard holes having a size sufficient to completely cover the slot after installation shall be provided. Such plate washers or continuous bars shall be not less than 8 mms thick and shall be of structural grade material but need not be hardened. In addition hardened washers shall be placed over the outer surface of the plate washer or bar.

8.6.3.2 Shear Resistance of Slip-Critical Connections at Service Load

As slip of any magnitude must be prevented, the slip resistance (shear resistance) is the design frictional strength at service load for a single high strength bolt of either grade 8.8 or 10.9 with a single friction plane. The slip resistance (P_s) is derived according to the following relation:

$$P_s = \phi . \mu \ T / (F.S) \qquad \text{.......... 8.8}$$

Where:

ϕ = 1.0 for standard and slotted holes when the slot length is perpendicular to the line of force, ϕ = 0.85 when the slot length is parallel to the line of force

T = axial pretensioning force in the bolt, tons

μ = friction coefficient

$F.S$ = Safety factor with regard to slip = 1.25 and 1.05 for cases of loading I and II respectively for ordinary steel work.
= 1.6 and 1.35 for cases of loading I and II respectively for cranes and crane girders which are subjected mainly to dynamic loads.

Table 8.3 gives the pretension force (T) and the slip resistance per one friction surface (P_s) for bolts with standard holes of grade 10.9.

8.6.3.3 Shear Resistance of Slip-Critical Connections at Factored Loads

a- In addition to Section 8.6.3.1, the high strength bolts in a slip-critical connection are to be checked in accordance with Section 8.5.2 for shear resistance based on factored loads.

b- Bearing Resistance based on factored loads is also to be checked in accordance with Section 8.5.3.

8.6.4 Tension Resistance in Slip-Critical Connections

The design of slip-critical connections for bolt grades 8.8 and 10.9 when subject to a factored tension force $(\gamma_i\, T_i)_{ext,b}$ per bolt: the total applied load per bolt shall be the sum of the factored tension force $(\gamma_i\, T_i)_{ext,b}$ and the prying force (P). The prying force (P) produced by deformation of the connected parts (refer to Fig. 8.5) should be determined according to section 8.8, and hence the following relation is to be satisfied:

$$(\gamma_i\, T_i)_{ext,b} + P \leq \phi_t R_{nt} \quad \text{........ 8.9}$$

$$\text{and } R_{nt} = (0.8\ F_{ub})\, A_s \quad \text{........ 8.10}$$

Where:

ϕ_t = 0.70

$(\gamma_i\, T_i)_{ext,b}$ = the applied external factored tension force per bolt, tons

R_{nt} = the ultimate tension resistance per bolt, tons

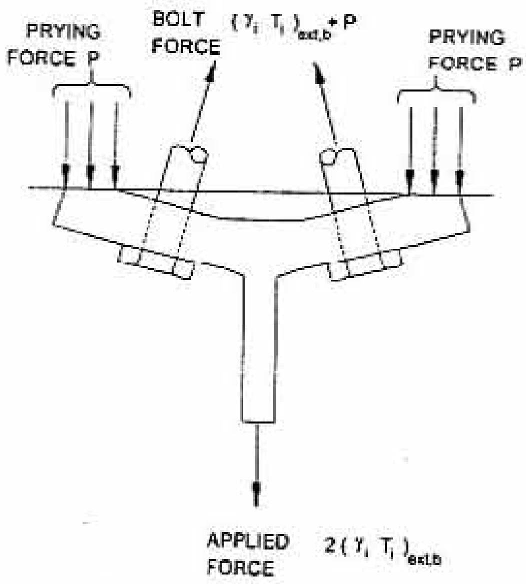


Figure 8.5 Prying Force

Table 8.3 Properties and Strength of High Strength Bolts (Grade 10.9*)

Bolt Diameter (d) mm	Bolt Area (A) cm^2	Stress Area (A_s) cm^2	Pretension Force (T) tons	Required Torque (M_a) kg.m	Permissible Friction Load of One Bolt Per One Friction Surface (P_s) tons							
					Ordinary Steel Work				Cranes			
					St. 37&44 (μ=0.4)		St. 52 (μ=0.5)		St. 37&44 (μ=0.4)		St. 52 (μ=0.5)	
					Cases of Loading**				Cases of Loading**			
					I	II	I	II	I	II	I	II
M12	1.13	0.84	5.29	12	1.69	2.01	2.11	2.52	1.32	1.56	1.65	1.95
M16	2.01	1.57	9.89	31	3.16	3.37	3.95	4.71	2.47	2.92	3.09	3.66
M20	3.14	2.45	15.43	62	4.93	5.90	8.17	7.36	3.85	4.56	4.82	5.71
M22	3.80	3.03	19.08	84	8.10	7.27	7.63	9.10	4.77	5.65	5.96	7.06
M24	4.52	3.53	22.23	107	7.11	8.45	8.89	10.60	5.55	8.58	8.94	8.22
M27	5.73	4.59	28.91	157	9.25	11.03	11.56	13.78	7.22	8.55	9.03	10.70
M30	7.06	5.61	35.34	213	11.30	13.48	14.13	18.86	8.83	10.46	11.04	13.07
M36	10.18	8.17	51.47	372	18.47	19.64	20.58	24.55	12.86	15.24	18.08	19.05

$T = (0.7)\ F_{yb}.A_s \quad M_a = 0.2\ d.T \qquad P_s = \phi \mu\ T / (F.S)$

* For HSB grade 8.8, the above values shall be reduced by 30%

** **Case I:** Primary stresses due to dead load + live loads or superimposed loads + dynamic effects + centrifugal forces.

Case II: Primary and additional stresses due to Case I + wind loads or earthquake loads, braking forces, lateral shock effect, change of temperature, frictional resistance of bearings, settlement of supports in addition to the effect of shrinkage and creep of concrete.

8.6.5 Combined Tension and Shear in Slip-Critical Connections

The design of slip-critical connections subjected to tension and shear must satisfy the following two relations:

a- Ultimate Tension Strength

$$(\gamma_i\, T_i)_{ext,b} + P \leq (\phi_t R_{nt} = \phi_t\,(0.8\ F_{ub})\, A_s) \quad \text{.......... 8.11}$$

Where:

$\phi_t = 0.70$

b- Serviceability slip constraint

$$Q_b \leq P_s\,(1 - T_{ext,b}/T) \quad \text{.......... 8.12}$$

Where:

P_s = the frictional resistance as given by Equation 8.8, tons
T = the pretension force per bolt as given by Equation 8.7, tons
$T_{ext,b}$ = applied external tension force per bolt based on service load, tons
Q_b = design slip resistance per bolt, tons

8.6.6 Design Strength of Slip-Critical Connections Subjected to Shear and Bending Moment

a- In moment connections of the type shown in Fig. 8.6 the loss of clamping forces in region "A" is always coupled with a corresponding increase in contact pressure in region "B". The clamping force remains unchanged and there is no decrease of the frictional resistance. The slip resistance (Q_b) must be checked based on service load according to the following relation:

$$Q_b \leq (P_s = \mu\ \ T\ /\ F.S.) \quad \text{.......... 8.13}$$

b- In addition the induced maximum factored tensile force $(\gamma_i\, T_i)_{ext,b,M}$ due to the applied factored moment $(\gamma_i\, M_i)$ and the prying force (P) that may occur must satisfy the following relation.

$$(\gamma_i\, T_i)_{ext,b,M} + P \leq (\phi_t R_{nt} = \phi_t\,(0.8\ F_{ub})\, A_s) \quad \text{.......... 8.14}$$

Where:

$(\gamma_i\, T_i)_{ext,b,M}$ = the maximum induced factored force due to the factored applied moment
ϕ_t = 0.70

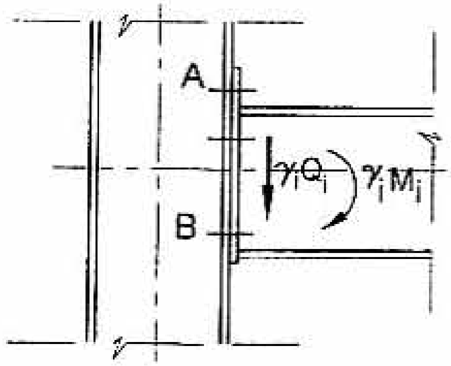


Figure 8.6 Connections Subjected to Combined Shear and Bending Moment

8.6.7 Design Strength of Slip-Critical Connections Subjected to Combined Shear, Tension and Bending Moment

When the connection is subjected to shearing force (Q), a tension force (T_{ext}) and a bending moment (M) due to different cases of loading, the design strength per bolt must satisfy the following:

a- Serviceability based on service loads

$$Q_b \leq (\mu\ T\ /\ (F.S))\ (1- T_{ext,b}\ /\ T) \qquad 8.15$$

b- Strength resistance based on factored loads

$$(\gamma_i\ T_i)_{ext,b} + (\gamma_i\ T_i)_{ext,b,M} + P \leq ((\phi_t R_{nt}) = \phi_t\ (0.8\ \ F_{ub})\ A_s) \qquad 8.16$$

Where:

$\phi_t = 0.70$

8.6.8 Bolting Procedure and Execution

a- Bolts may be tightened by calibrated wrenches, which can indicate either the applied torque or the angle of rotation of the nut.

For the first method, torque wrenches which have a cut–out-device to limit the required amount of the applied torque must be employed. Wrenches may be of the manual, pneumatic, or electric-type. The torque "M_a" (Table 8.3) required to induce the pretensioning force "T" shall be calculated as follows:

$$M_a = 0.2d.T \qquad 8.17$$

Where:

M_a = applied torque, t.cm

d = diameter of bolt, cm

T = bolt pretension force, tons

b- The second method of tightening is based on a predetermined rotation of the nut. The tightening can be achieved in different ways as follows:

i- The parts to be joined are first brought into contact by making the bolts snug tight by a few impacts of an impact wrench. Following this initial step each nut is tightened one half turn.

ii- The bolt is first tightened using a wrench until the several plies of the joint achieve a " snug fit" after which the nut is further turned by the amount:

$$\alpha = 90^{o} + t + d \qquad 8.18$$

Where:

α = rotation in degrees

t = total thickness of connected parts, mm

d = bolt diameter, mm

c- Preparation of Contact Surfaces

The contact surfaces must be free from dust, oil, paint, etc. Spots of oil cannot be removed by flame cleaning without leaving harmful residues, and must be removed by chemical means. It is sufficient to remove any film of rust or other loose material by brushing with a soft steel brush.

d- Protection Against Corrosion

Parts to be joined with high strength bolts of the friction type must be protected against corrosion, by suitable protection against entry of humidity between the contact surfaces as well as the bolt holes.

For structural components, where the contact surfaces have been prepared for a prestressing process, and are stored for long periods, there is a risk of rusting. An inspection regarding the coefficient of friction is essential.

e- Inspection

i- Tensioning Force

One of the following two procedures may be adopted to check that the specified torque "M_a" has been applied:

– The bolt is turned a further 10° for which at least the specified torque has to be applied.

- The position of the nut on the bolt which is to be checked is marked. The bolt is then held firmly and the nut is unscrewed by 1/6 of a complete turn. To turn the nut back to its original position, it is necessary to apply the specified torque.

ii- Friction Coefficient Check

It is desirable to make random checks of the friction coefficient achieved by surface preparation.

8.7 HIGH STRENGTH PRETENSIONED BOLTED CONNECTIONS OF THE BEARING TYPE CATEGORY (C) – "ULTIMATE LIMIT STATE"

8.7.1 Design Basis

In this category of connections, high strength bolts of grade 8.8 and 10.9 are only to be utilized. The bolts are pretensioned by tightening in accordance with section 8.6.8.

Fully pretensioned bolts in this category of connections have no requirements regarding the contact surface.

8.7.2 Design Shear Strength, Bearing Strength, Tension Strength and Combined Shear and Tension of the Pretensioned Bolted Connections of the Bearing Type

The ultimate design strength must follow the following relations previously outlined in Sections 8.5.2, 8.5.3, 8.5.4, and 8.5.5 as follows:

Shear:

$$\phi_v R_{nv} = \phi_v \,(0.6\, F_{ub})\, A_s\, n \qquad 8.19$$

$$\phi_v R_{nv} = \phi_v \,(0.5\, F_{ub})\, A_s\, n \qquad 8.20$$

Bearing:

$$\phi_{br} R_{br} = \phi_{br}\,.\, d.\, (\min \sum t\,(\alpha F_u)) \qquad 8.21$$

Tension:

$$(\gamma_l T_i)_{ext,b} + P \leq \phi_t R_{nt} \qquad 8.22$$

$$R_{nt} = (0.8\, F_{ub})\, A_s \qquad 8.23$$

Combined tension and shear:

$$\left[\frac{R_{ut}}{\phi_t R_{nt}}\right]^2 + \left[\frac{R_{uv}}{\phi_v R_{nv}}\right]^2 \leq 1.0 \qquad 8.24$$

Where:

$$R_{ut} = (\gamma_i\, T_i)_{ext,b}$$

For the case of bending moment and tension and shear Equation 8.15 and 8.16 previously outlined in Section 8.6.7 is to be utilized as follows:

$$(\gamma_i\, T_i)_{ext,b} + (\gamma T_i)_{ext,b,M} + P \leq (\phi_t R_{nt} = \phi_t\,(0.8\ F_{ub})\, A_s) \ldots\ldots \quad 8.25$$

8.8 PRYING FORCE

8.8.1 General

The prying force is a tension force produced by differential deformation of the connected parts. The prying force (P) shall be determined according to section 8.8.3.

8.8.2 Configuration

Figure 8.7 a, b and c illustrates the most common type of connections, where the outer overhangings may press on their corresponding supports causing the prying force (P). The prying action depends on the flexibility of the tee stub flange and the end plate which is denoted in Fig. 8.7 a, b and c by the thickness (t_p).

8.8.3 Determination of the Prying Force

In order to determine the prying force P, the connection is to be transformed to an equivalent tee stub connection as shown in Fig. 8.8. The prying force (P) can be determined using the following:

$$P = \left[\frac{5\,b}{8\,a} - \frac{t_p^3}{100}\right] \cdot \left((\gamma_i T_i)_{ext,b} \text{ or } ((\gamma_i T_i)_{ext,b,M})\right) \ldots\ldots\ldots\ldots 8.26$$

Alternatively, a more refined value of the prying force can be obtained using the following:

$$P = \left[\frac{\dfrac{1}{2} - \dfrac{w t_p^{\,4}}{30ab^2 A_s}}{\left(\dfrac{3a}{4b}\right)\left(\dfrac{a}{4b}+1\right) + \dfrac{w t_p^{\,4}}{30ab^2 A_s}}\right] \cdot \left((\gamma_i T_i)_{ext,b} \text{ or } ((\gamma_i T_i)_{ext,b,M})\right) \ldots\ldots\ldots 8.27$$

Where:

w = flange tee stub breadth tributary to one bolt, cm

a,b = bolt outer overhanging and inner bolt dimensions w.r.t the stem tee stub; respectively, cm

$(\gamma_i\ T_i)_{ext,b}$ = factored external tension force per bolt, tons

$(\gamma_i\ T_i)_{ext,b,M}$ = factored external tension force per bolt due to the factored applied moment $(\gamma_i\ M_i)$, tons

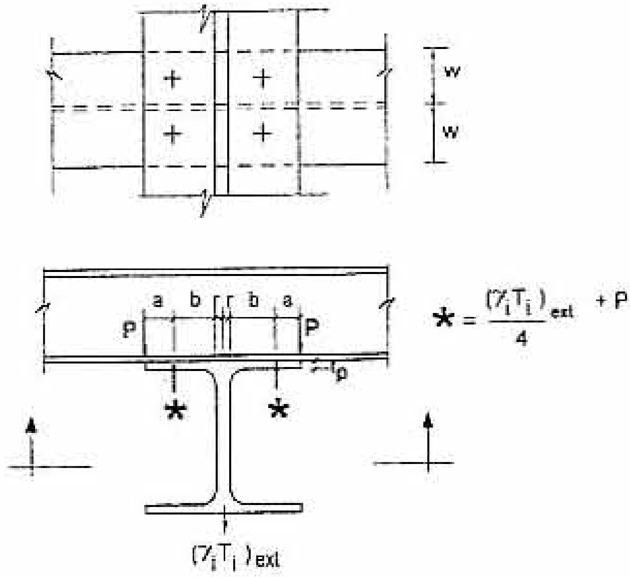
$(\gamma_i\ T_i)_{ext,b,M}$ (Fig. 8.7.b) can be obtained by replacing the factored moment $(\gamma_i\ M_i)$ by two equal and opposite external forces.

$(\gamma_i\ T_i)_M = (\gamma_i\ C_i)_M = (\gamma_i\ M_i)\ /d_b$ (Fig. 8.7.c) or by applying an exact analysis of an end plate moment connection.

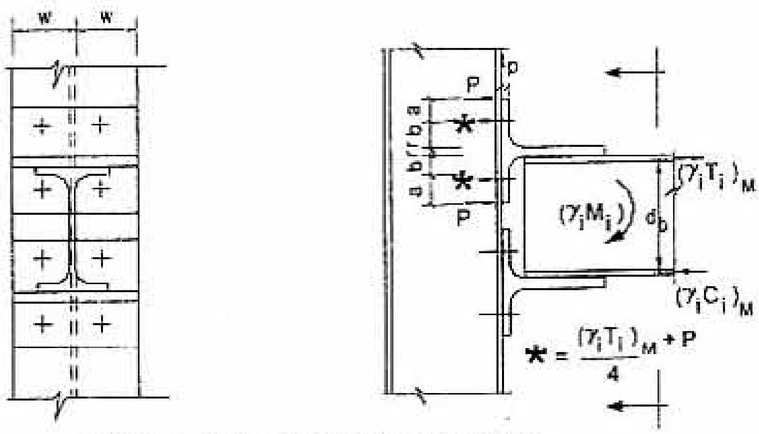
Where:

$(\gamma_i\ T_i)_{ext.b} = (\gamma_i\ T_i)_{ext}\ /\ 4$ (Fig 8.7.a)

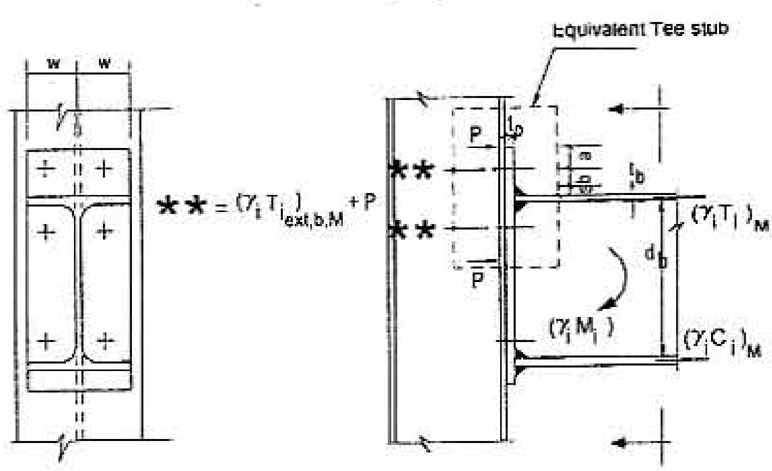
$(\gamma_i\ T_i)_{ext,b,M} = (\gamma_i\ T_i)_M\ /\ 4$ (Fig. 8.7.b)



(a) Beam to beam connection
in orthogonal direction



(b) Tee stub moment connection



(c) End plate moment connection

Figure 8.7 Common Types of Connections Producing Prying Forces

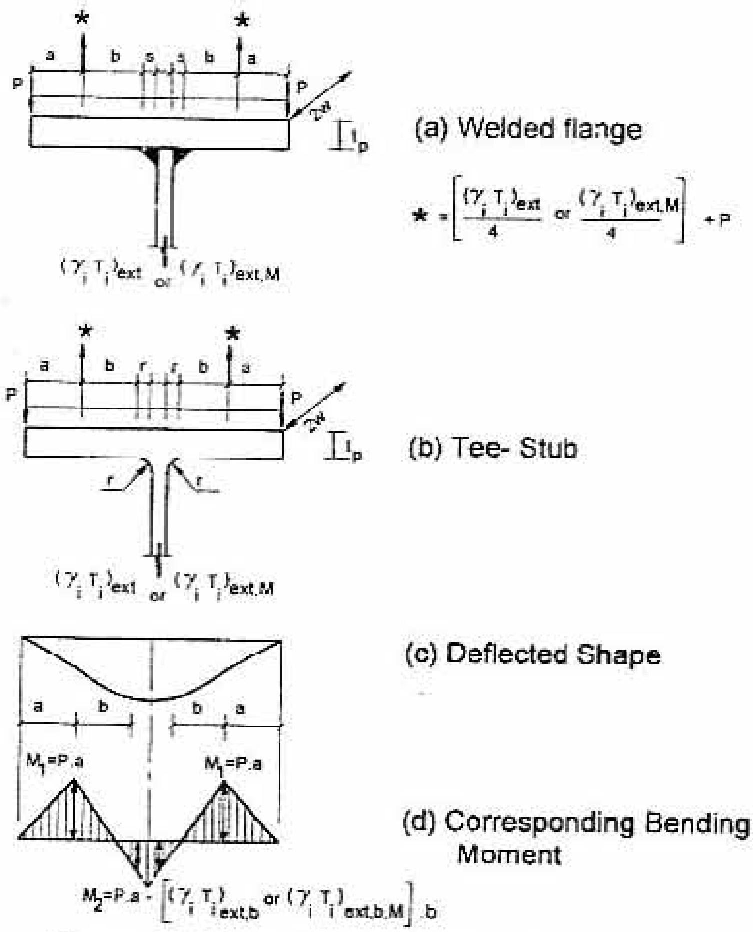


Figure 8.8 Equivalent Tee Stub Connection

8.8.4 Determination of the Tee Stub or the End Plate Thickness

a- The ideal situation shown in Fig. 8.9 is to place the rows of bolts at (A-A) and (B-B) as close as possible to the tension flange (two bolts or four bolts per row).

b- A row of bolts near the beam compression flange at C-C is to be utilized in order to prevent this part from springing.

c- Compute an approximate end plate thickness using the model shown in Fig. 8.9.b or using the following relations:

- case of four bolts around tension flange (i.e. two bolts per row with flange breadth = $2w$):

$$t_p = 0.50\sqrt{\frac{\gamma_i\ M_i(2b+2s+t_b)}{d_b\ w\ F_y}} \qquad \text{.. 8.28}$$

- case of eight bolts around tension flange (i.e. four bolts per row with flange breadth = $4w$):

$$t_p = 0.353\sqrt{\frac{\gamma_i\ M_i(2b+2s+t_b)}{d_b\ w\ F_y}} \quad \text{.......... 8.29}$$

Where:

$\gamma_i\ M_i$	=	factored beam moment, cm.t
b	=	internal distance w.r.t the Tee-Stub web of the beam flange,, cm
s	=	fillet weld size, cm
$t_b,\ d_b$	=	flange beam thickness and depth ($d_b = h - t_b$), cm
w	=	flange tee stub or end plate breadth w.r.t one column of bolts (i.e. tributary to one bolt), cm

d- Compute the induced factored prying force P using Equation 8.26 where the end plate thickness corresponds to step (c) of Equation (8.28) or (8.29).

e- Compute the exact factored induced bending moment in the end plate as follows (Fig. 8.9.c)

$$M_1 = P.a$$

$$M_2 = P.a - (\gamma_i\ T_i)_{ext,b,M}\ .b \quad \text{.......... 8.30}$$

f- Hence compute the exact required end plate thickness and the safety of bolts using the following two equations:

$$t_p = \sqrt{\frac{4(\text{greater of } M_1 \text{ or } M_2)}{w\ F_y}} \quad \text{.......... 8.31}$$

$$(\gamma_i\ T_i)_{ext,b,M} + P \leq (\phi_t\ R_{nt} = 0.70 x\ 0.8\ F_{ub}\ A_s) \quad \text{.......... 8.32}$$

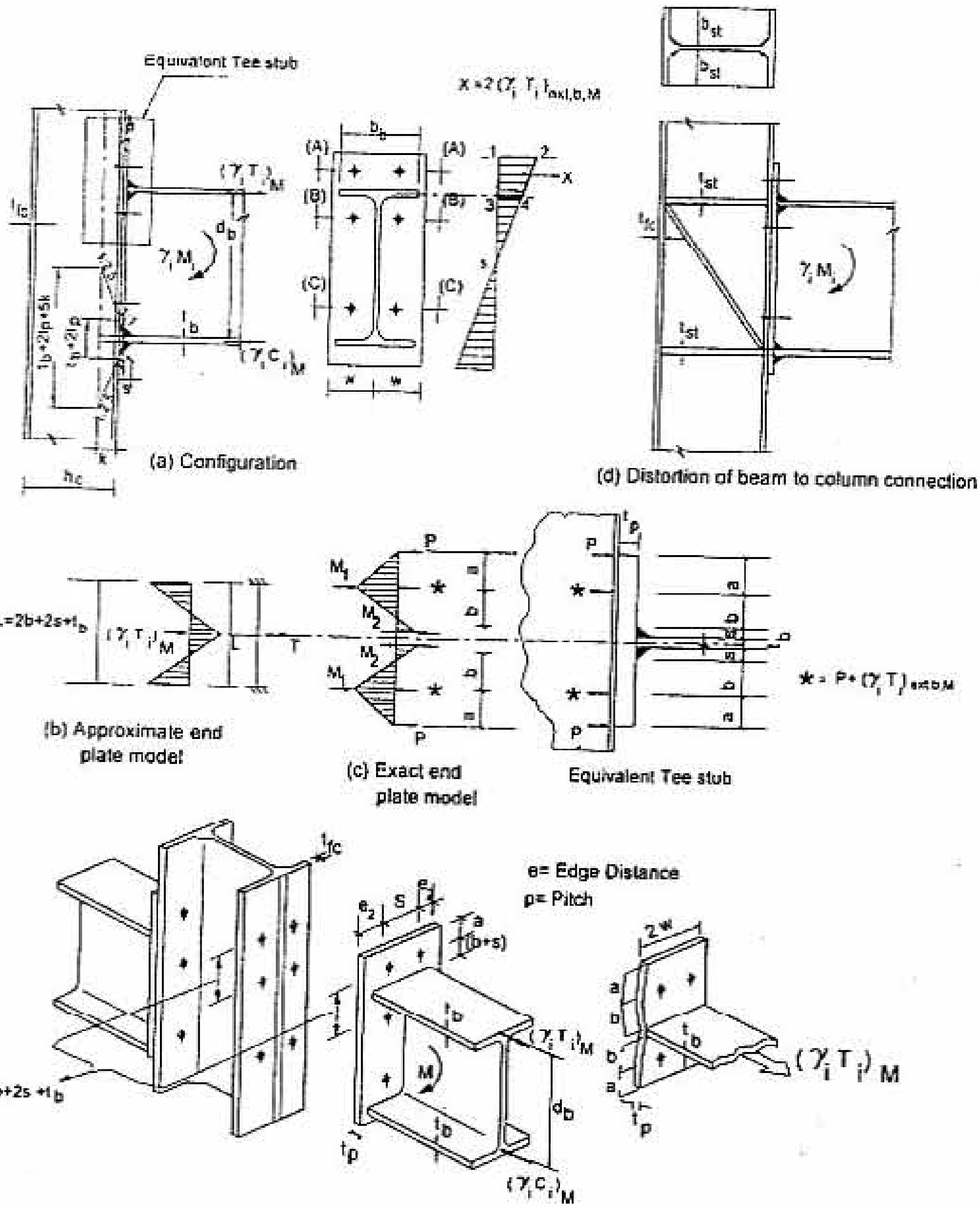


Figure 8.9 Determination of the Tee Stub or the End Plate Thickness "t_p"

8.9 DESIGN RUPTURE STRENGTH

8.9.1 Shear Rupture Strength

The design strength for the limit state of rupture along a shear failure path in the affected elements of connected members shall be taken as $\phi_v R_n$ as follows:

$$\phi_v R_n = \phi_v (0.6 F_u) A_{nv} \qquad 8.33$$

Where:

ϕ_v = resistance factor for shear = 0.70

F_u = ultimate tensile stress of connected element, t/cm^2

A_{nv} = net area subject to shear, cm^2

8.9.2 Tension Rupture Strength

The design strength for the limit state of rupture along a tension path in the affected elements of connected members shall be taken as $\phi_t\ R_n$ as follows:

$$\phi_t\ R_n = \phi_t\ (\ F_u\ )\ A_{nt} \qquad \text{.......... 8.34}$$

Where:

ϕ_t = resistance factor for rupture = 0.70

A_{nt} = net area subject to tension, cm^2

8.9.3 Block Shear Rupture Strength

At beam end connections, where the top flange is coped and for similar situations where a failure might occur by shear along a plane through the fasteners plus tension along any perpendicular plane, a block shear failure occurs as shown in Fig. 8.10.

Block shear is a limit state in which the resistance is determined by the sum of the shear strength on a failure path(s) and the tensile strength on a perpendicular segment. When ultimate rupture strength on the net section is used to determine the resistance on one segment, yielding on the gross section shall be used on the perpendicular segment. The block shear rupture design strength $\phi_v\ R_n$ shall be determined as follows:

a- When:

$$F_u\ .\ A_{nt} \geq 0.6\ F_u\ .\ A_{nv}$$

$$\phi_v\ R_n = \phi_v\ (0.6\ F_y\ .\ A_{gv} + F_u\ .\ A_{nt}) \qquad \text{.......... 8.35}$$

b- When:

$$0.6\ F_u\ .\ A_{nv} > F_u\ .\ A_{nt}$$

$$\phi_v\ R_n = \phi_v\ (0.6\ F_u\ .\ A_{nv} + F_y\ .\ A_{gt}) \qquad \text{.......... 8.36}$$

Where:

ϕ_v = resistance factor for shear = 0.70

A_{gv} = gross area subject to shear, cm^2

A_{gt} = gross area subject to tension, cm^2

A_{nv} = net area subject to shear, cm^2

A_{nt} = net area subject to tension, cm^2

R_n = nominal shear strength, tons

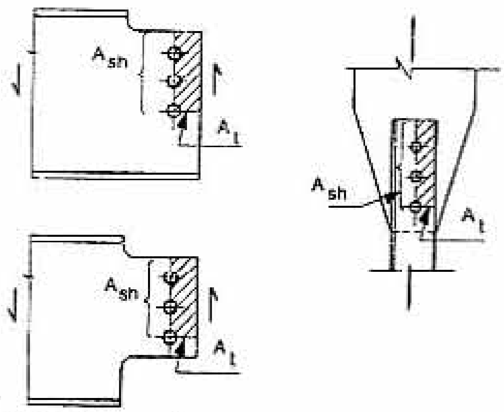


Figure 8.10 Failures by Tearing Out of Shaded Area

8.10 ADDITIONAL REMARKS

8.10.1 Long Grip

a- Bolts of grade 4.6 and 4.8, which carry calculated stresses fulfilling Section 8.5 with a grip exceeding 5 times the bolt diameter (*d*) shall have their number increased 1% for each 1 mm increase in the grip.

b- This provision for other grades of bolts shall be applicable only with grip exceeding 8*d*.

8.10.2 Long Joints

a- Where the distance L_j between the centers of end fasteners in a joint, measured in the direction of the transfer of force (Fig. 8.11) is more than 15*d*, the design shear and bearing strength of all the fasteners calculated as specified in Sections 8.5.2 and 8.5.3 shall be reduced by a reduction factor B_L given by the following:

$$B_L = 1 - \frac{L_j - 15d}{200d} \qquad \text{........ 8.37}$$

Where:

$$0.75 \le B_L \le 1.0 \quad and \quad L_j \le 65\,d$$

b- This provision is not to be applied, when there is a uniform distribution of force transfer over the length of the joint such as the transfer from the web of I section to the column flange.

8.10.3 Single Lap Joints With One Bolt

a- In single lap joints with only one bolt, (Fig. 8.12) the bolt shall be provided with washers under both the head and the nut to avoid pull-out failure.

b- The bearing strength determined in accordance with Section 8.5.3 shall be limited to :

$$R_{nv} \le \phi_t\,(1.5\,F_u).\,d.t \qquad \text{........ 8.38}$$

c- In case of high strength bolts of grades 8.8 and 10.9 hardened washers should be used for single lap joints with only one bolt.

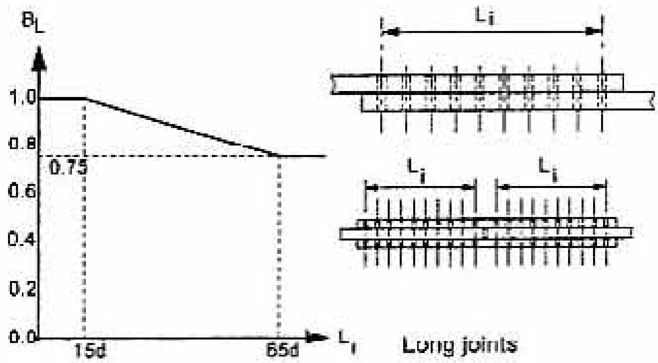


Figure 8.11 Long joints

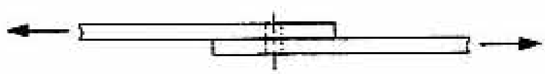


Figure 8.12 Single Lap Joint with One Bolt

8.10.4 Fasteners through Packing

a. Where bolts transmitting load in shear and bearing pass through packings of total thickness t_p greater than one-third of the bolt diameter d, the design shear strength calculated as specified in Section 8.5.2 shall be reduced using a reduction factor B_b as follows:

$$B_b = \frac{9d}{8d + 3t_p} \quad \text{.......} \quad 8.39$$

Where: $B_b \leq 1$

b. For double shear connections with packing on both sides of a splice, t_p should be taken as the thickness of the thicker packing.

8.10.5 Hybrid Connections

a. When different forms of fasteners are used to carry a shear load or when welding, and fasteners are used in combination, then one form of connector shall normally be designed to carry the total load.

b- This section is valid for ordinary bolts and high strength bolts of the bearing type.

c- As an exception to this provision, pretensioned high-strength bolts of Grades 8.8 and 10.9 in connections designed as friction type (slip-critical connections) may be designed as sharing with welds, provided that the final tightening of the bolts is carried out after the welding is completed.

d- In making welded alterations to structures, existing rivets and high-strength bolts tightened to the requirements for slip-critical connections are permitted to be utilized for carrying loads present at the time of alteration and the welding need only provide the additional design strength required.

8.10.6 Anchor Bolts and Tie Rods

The design shear and tensile strength through the threaded portion as prescribed in Sections 8.5.2 and 8.5.4 are restricted to bolts of different grades.

For other threaded parts as anchor bolts or threaded tie rods fabricated from round steel bars, the shear and tensile strength given by Equations 8.2 and 8.5 shall be decreased by 15%.

8.11 COLUMN BASES AND BEARING ON CONCRETE

Proper provisions shall be made to transfer the column loads and moments to the footings and foundation. The design requirements are as given below:

a- The design bearing strength ($\phi_c\ P_p$) must at least equal the factored column load P_u

$$\phi_c P_p \geq P_u \qquad \text{8.40}$$

Where:

$\phi_c = 0.6$ for bearing on concrete

b- There are two categories of nominal strength P_p in bearing as follows:

i- Bearing on the full area A_1 of concrete support:

$$P_p = 0.85\ F_c'\ A_1 \qquad \text{8.41}$$

ii- Bearing on area A_1 which is less than the full area A_2 of a concrete support:

$$P_p = 0.85\ F_c'\ A_1 \sqrt{A_2 / A_1} \leq 0.85\ F_c'\ (2A_1) \qquad \text{8.42}$$

Where:

A_1 = area of steel plate concentrically bearing on concrete support, cm^2
A_2 = maximum area of the position of the supporting surface that is geometrically similar to and concentric with loaded area, cm^2

$F_c^{'}$ = specified 28 day compressive strength of concrete, t/cm^2

8.12 BEARING STRENGTH SURFACES

The strength of surfaces in bearing is ϕR_n where $\phi = 0.70$. R_n is defined below for the various types of bearing.

a- For milled surfaces, pins in reamed, drilled, and ends of fitted bearing stiffeners:

$$R_n = 1.8 \, F_y \, A_{pb} \qquad \text{.......... 8.43}$$

Where:

F_y = specified minimum yield stress, t/cm^2

A_{pb} = projected bearing area, cm^2.

b- For expansion rollers:

If $d \leq 60$ cm:

$$R_n = 1.2 \, (F_y - 0.90) \, \ell d \, /20 \qquad \text{.......... 8.44}$$

CHAPTER 9

STRUCTURAL WELDING

The following sections regarding the welded connections are applicable to structures loaded with predominantly static loads, while for fatigue loadings refer to Chapter 11.

9.1 WELDABILITY AND STEEL PROPERTIES

"Weldability "is the capacity of a metal to be welded under the fabrication conditions imposed, into a specific, suitably designed structure, and to perform satisfactorily in the intended service.

Weldability is enhanced by low carbon, fine grain size and restricted (low) thickness. Conversely, it is reduced by high carbon, coarse grain, and heavy thickness. Table 9.1 abstracts the requirements covering weldability related variables.

9.2 STRUCTURAL WELDING PROCESS, WELDING POSITIONS AND ELECTRODES REQUIREMENTS

9.2.1 Welding Positions

The different welding positions are shown in Fig. 9.1 where:

a- In the flat position weld metal can be deposited faster because gravity is working with the welder, so large electrodes and high currents can be used.

b- In the vertical and overhead positions, electrodes diameters below 4 mm (or at most 5 mm) are to be utilized otherwise weld metal runs down.

c- For arc welding the weld metal is deposited by the electro-magnetic field, the welder is not limited to the flat or horizontal position.

d- The designer should avoid whenever possible the overhead position, since it is the most difficult one.

e- Welds in the shop are usually in the flat position, where manipulating devices can be used to rotate the work in a flat position.

f- Field welds that may require any welding position depending on the orientation of the connection have to meet welding inspection requirements of Section 9.9.

Table 9.1 Requirements for Properties Affecting Weldability of Steel Sections, Plates, and Bars

Grade of Steel	Nominal Values of Yield Stress F_y and Ultimate Strength F_u				Maximum Thickness of Statically Loaded Structural Elements (mm)	Charpy V-notch Test Temperature T_{cv}		
	Thickness t					T_{cv} Test Temperature (^oc)	Minimum Energy J (joules) for	
	t ≤ 40 mm		40 mm < t ≤100 mm					
	F_y (t/cm^2)	F_u (t/cm^2)	F_y (t/cm^2)	F_u (t/cm^2)			> 10mm ≤150 mm	>150mm ≤250 mm
St 37	2.40	3.70	2.15	3.40	250	-20°	27	23
St 44	2.80	4.40	2.55	4.10	150	-20°	27	23
St 52	3.60	5.20	3.35	4.90	130			

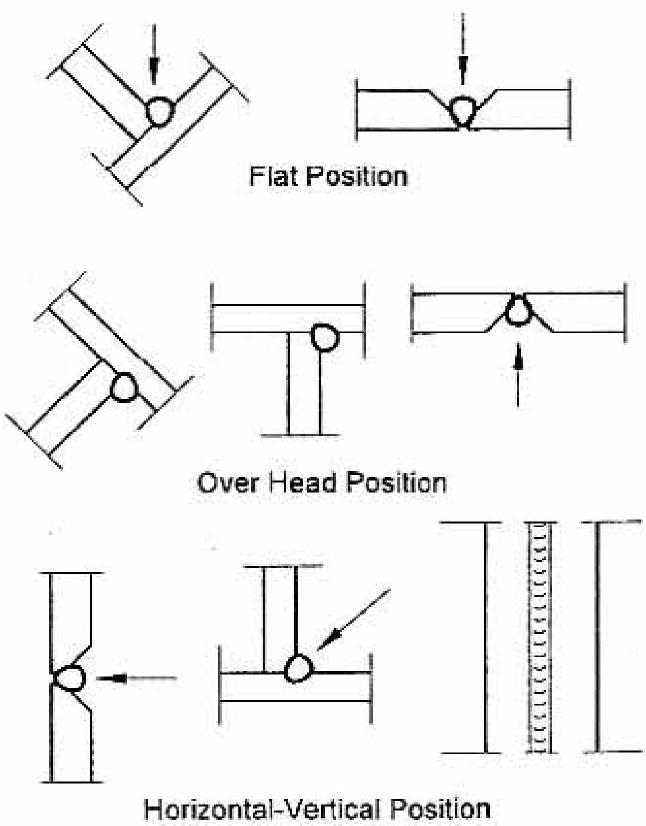


Figure 9.1 Welding Positions

9.2.2 Welding Processes

Weldable structural steels meeting the requirements of Table 9.1 are welded by one of the following welding processes:

- Shielded Metal Arc Welding (S.M.A.W.)
- Submerged Arc Welding (S.A.W.)
- Gas Metal Arc Welding (G.M.A.W.)
- Flux Cored Arc Welding (F.C.A.W.)
- Gas Tungsten Arc Welding (G.T.A.W.)

9.2.3 Electrode Requirements

a- The common sizes of electrodes for hand welding are 4 and 5 mm diameter. For the flat welding position 6 mm can be used.

b- 8 mm fillet weld size is the maximum size that can be made in one run with 5 mm coated electrodes.

c- For large sizes several runs of electrode in arc welding are to be made, while for gas processes any size can be made in one run.

The appropriate electrode type regarding the weld process as well as their yield and maximum tensile strength are given in Table 9.2.

The following flow chart designates the electrode and flux symbols given in Table (9.2).

9.3 THERMAL CUTTING

Two cutting systems are available:

a- Oxyfuel gas, which can cut almost any plate thickness used commercially.

b- Plasma arc which will cut almost up to about 40 mm thickness and is much faster than Oxyfuel.

9.4 DISTORTION

Non- uniform rate of cooling after welding causes shrinkage which may cause distortion in the welded steel elements. In order to minimize distortion the following recommendations are to be taken into consideration:

a- Use the minimum weld metal no larger than is necessary to achieve the design strength.

b- Use symmetrical simultaneous welds.

c- Use minimum preheat. The rate of preheating must be slow and uniform, it is desirable to maintain the preheat temperature during the whole welding process.

d- For welds requiring more than one pass of welding, the interpass temperature is to be maintained to the temperature of the deposited welds when the next pass is begun.

e- Use intermittent staggered welds.

f- Use clamps, jigs, etc., this forces weld metal to stretch as it cools.

Flowchart electrode and flux symbols designation

Shielded Metal Arc Welding (SMAW)

E	A	B	C	D

Electrode	Minimum tensile strength of deposit weld metal in imperical units = (60 ksi = 4.20 t/cm^2 or 70 ksi = 4.90 t/cm^2)	Electrode welding position = 1 for all positions including vertical up = 2 for only flat and horizontal positions = 4 for all positions excluding vertical up	= 5,6 or 8 for low hydrogen electrodes (moisture content should be less than 0.2% of the volume) = 2,3,or 4 rutile electrodes = 1 or 0 for organic electrodes

Gas Metal Arc Welding (GMAW)

E	R	A	B	S	C

Electrode if used for GMAW	Rod if used for GTAW	Minimum tensile strength of deposit weld metal in imperical units = (60 ksi = 4.20 t/cm^2 or 70 ksi = 4.90 t/cm^2	Solid electrode	Express the operating characteristics of electrode such as penetration, polarity, current type, etc.......

Submerged Arc Welding (SAW)

F	A	H	I

Flux	Minimum tensile strength of deposit weld metal in imperical units = (60 ksi = 4.20 t/cm^2 or 70 ksi = 4.90 t/cm^2)	= A for as welded = P post weld heat treatment	The temprature that gives impact strength of 27 joules = 3 for -30 C°, = 4 for -40 C°, etc.....

Flux Chord Arc Welding (FCAW)

E	A	C	T

Electrode	Minimum tensile strength of deposit weld metal in imperica units = (60 ksi = 4.20 t/cm^2 or 70 ksi = 4.90 t/cm^2)	Electrode welding position = 1 for all positions = 2 for only flat and horizontal positions	Usability Code = 2 for single pass CO_2 shielded only

Table 9.2 Metal / Filler Metal Combination for Mechanical Chemical Matching

Group	Material Code	Min. Tensile Strength (t/cm^2) *	Min. Yield Strength (t/cm^2) *	Welding Process	Electrode Classification	Min. Yield Strength (t/cm^2)	Min. Tensile Strength (t/cm^2)
I	ST 37	3.40-3.70	2.15-2.40	SMAW	E60xx	3.3	4.15
					ER70S	4.0	4.80
				GMAW	F6xx	4.00	4.80
				SAW	F7xx	3.30	4.15
					E6xT	4.00	4.80
				FCAW	E7xT	3.30	4.15
					E60xx	4.00	4.80
	ST 44	4.10-4.40	2.55-2.80	SMAW	E70xx	4.0	4.80
II	ST 52	4.90-5.20	3.35-3.60	GMAW	ER70S	4.00	4.80
				SMAW	F7xx	4.00	4.80
				FCAW	E7xT	4.00	4.80

* The minimum values are shown in Section 1.3.3.

9.5 DESIGN, STRENGTH AND LIMITATIONS OF BUTT (GROOVE) WELDED CONNECTIONS

9.5.1 Nomenclature of the Common Terms

Figure 9.2 shows the nomenclature of the common terms for groove welds.

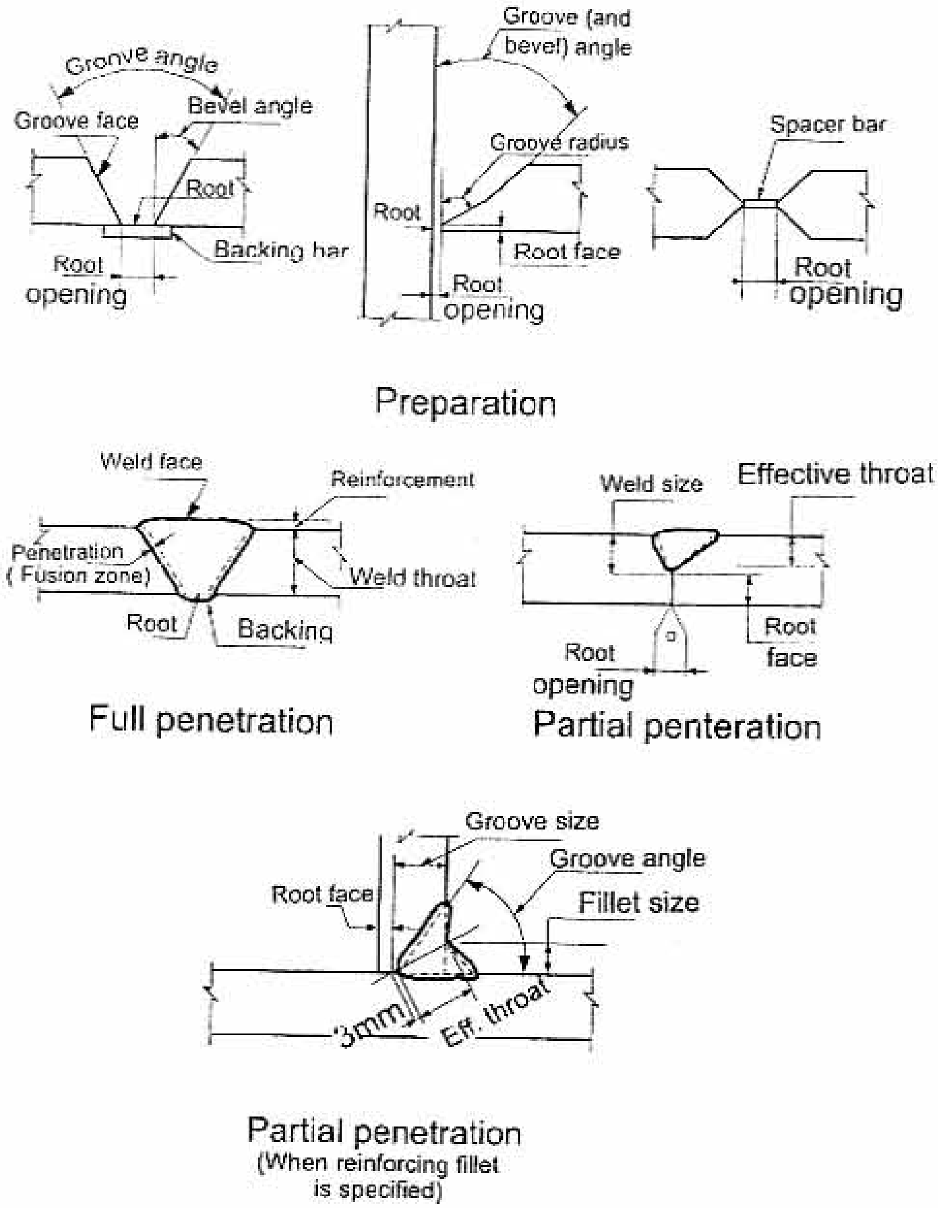
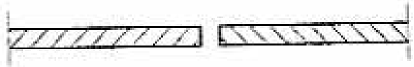
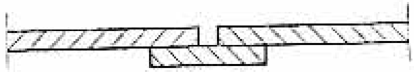
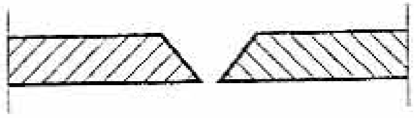
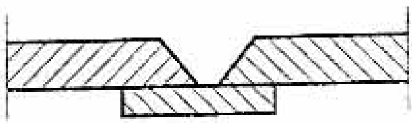
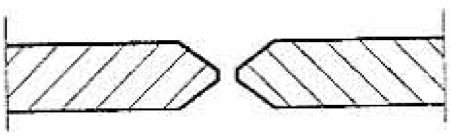
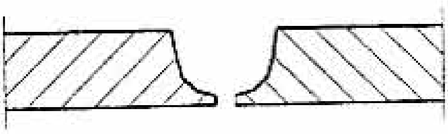
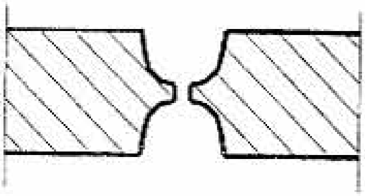


Figure 9.2 Butt (Groove) Weld Nomenclature

Table 9.3 Types of Groove Butt Welding

1.  Square - Butt • Welded from one side: up to 1.5 mm thick-no gap. • Welded from both sides using Normal Electrodes : up to 3 mm. thick-no gap. up to 5 mm. thick-take 1.5 mm gap. • Welded from both sides using deep penetration electrodes: up to 16 mm. -no gap.	2.  Square Butt with Backing Bar • Normal Electrodes : up to 5 mm. thick. 5mm gap. up to 13mm. thick. 8mm gap. • Deep Penetration Electrodes : up to 13mm. thick. 6mm gap.
3.  Single "V" Butt Weld • Included Angle: 60^0 for flat position . 70^0 for vertical position . 80^0 for over head position . • Root Thickness: 0-3 mm . • Thickness up to 25 mm . • Gap 1.5 mm - 3 mm .	4.  Single "V" Butt Weld With Backing Bar • Included angles as(3). • Gap 3 mm. - 5 mm . • Thickness up to 25 mm .
5.  Double "V" Butt Weld • Included angle gap and root thickness as(3) • Thickness 16 mm. - 50 mm.	6.  Single "U" Butt Weld • Inculded angle 20^0- 40^0 • Gap 3 mm. - 5 mm . • Root thickness 3mm.-5mm. • Root radius 3mm.-10mm. • Thickness 25mm.-50mm.

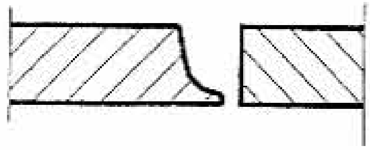
7.



Double "U" Butt Weld

- Dimensions as(6).
- Thickness 38 mm. upwards.

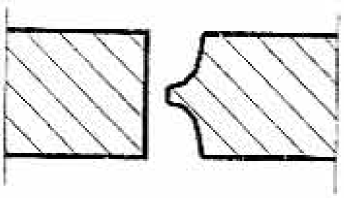
8.



Single "J" Butt Weld

- Included angle 20° - 30°.
- Gap 3mm. - 5mm.
- Root thickness 3mm.-5mm.
- Root radius 5mm.-10mm.
- Thickness 25mm - 50mm.

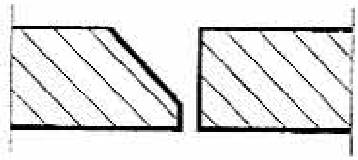
9.



Double "J" Butt Weld :

- Dimensions as(8).
- Thickness 38 mm upwards .

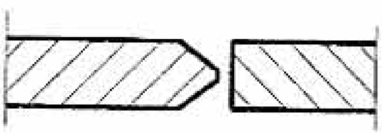
10.



Single Bevel Butt Weld

- Included angle 45° - 50°.
- Gap 3 mm. - 6 mm.
- Root thickness 1.5mm-3mm.
- Thickness up to 25 mm.

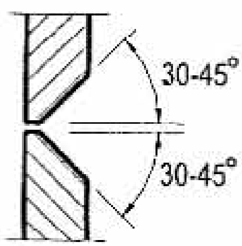
11.



Double Bevel Butt Weld

- Dimensions as 10 .
- Thickness 25 mm. upwards .

12.



Double Bevel Butt Weld

- Angles as shown .
- Gap 1.5 mm - 3 mm .
- Root thickness 1.5mm-3mm.
- Thickness up to 38 mm.

Regarding the advantages, the economy, and the defects of each type, the following remarks are to be considered:

a- Double bevel, double V, double J and double U groove welds are more economical than single welds of the same type because of less contained volume.
b- Bevel or V grooves can be flame cut and therefore are less expensive than J and U grooves which require planning or arc- air gouging.
c- Single V welding is achieved from one side, it is difficult to prevent distortion, and this type is usually economical up to 25 mm thickness.
d- Single U welding is achieved from one side, the distortion is less than the single V and is not economical under 19 mm thickness.
e- Double V is a balanced welding with reduced distortion requires reversals.
f- Double U is a balanced welding with reduced distortion requires reversals and is not recommended below 38mm thickness.
g- Groove welds joining plates of different thicknesses shall preferably be made with a gradual thickness change not exceeding 1:4 as shown in Fig. 9.3a for tension members. In compression members there is no need for a gradual thickness transition. The difference in thickness may be balanced by a slope in the weld metal rather than machining the parent metal as shown in Fig. 9.3b.
h- T-Groove welds are accepted even if they are not completely welded achieving a partial penetration groove weld if the total weld thickness is greater than the parent metal thickness, see Fig.9.3c.

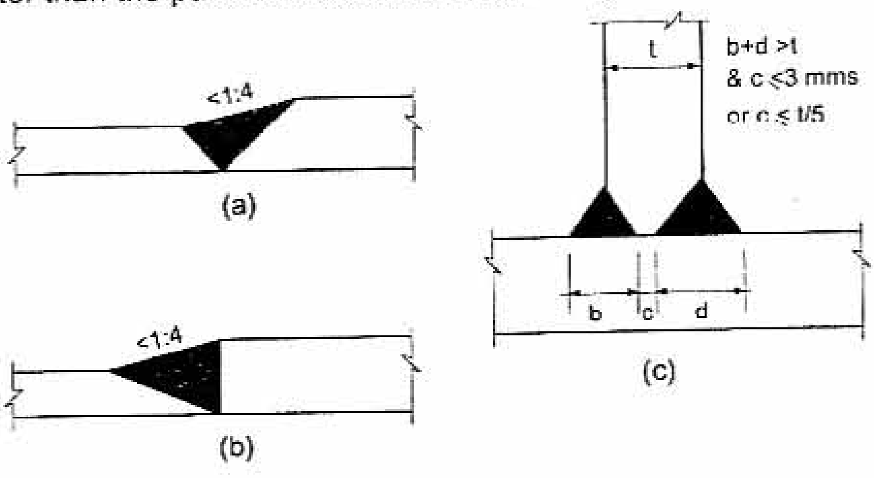


Figure 9.3 Groove Welds for Plates of Different Thicknesses

If the requirements of Fig. 9.3.c are not fulfilled the Tee-Groove welds are to be analyzed as being fillet welds according to the provisions of Section 9.6.

9.5.2 The Groove Weld Effective Area and Thickness Dimensions

a- The effective area of groove welds shall be considered as the effective length of the weld, times the effective thickness dimension.
b- The effective length of a groove weld shall be the width of the part joined.
c- The effective thickness dimension of a full penetration groove weld is the thickness of the thinner part joined as shown in Fig. 9.4.a.

d- For incomplete (Partial) penetration groove welds and unsealed groove welds, the effective thickness, of weld is taken as the sum of the actual penetrated depths, as shown in Fig. 9.4 b, c and d.

e- For J or U partial joint penetration groove weld the effective thickness dimension shall be the depth of chamfer.

f- For bevel or V joint where the induced angle at the root of the groove is greater than 60° the thickness dimension shall be the depth of chamfer (D). While if the induced angle is less than 60° and greater than or equal to 45° the effective thickness dimension shall be the depth of the chamfer minus 3 mms (i.e. $t_g = D - 3\ mms$) as shown in Fig. 9.4 e and f.

g- To insure fusion and minimize distortion for partial joint penetration groove weld the minimum thickness dimension is determined by the thicker of the two parts joined as given in Table 9.4.

h- Weld size is determined by the thicker of the two parts joined except that the weld size need not exceed the thickness of the thinner part joined when a larger size is required by calculated strength. For this exception particular care shall be taken to provide sufficient soundness of the weld.

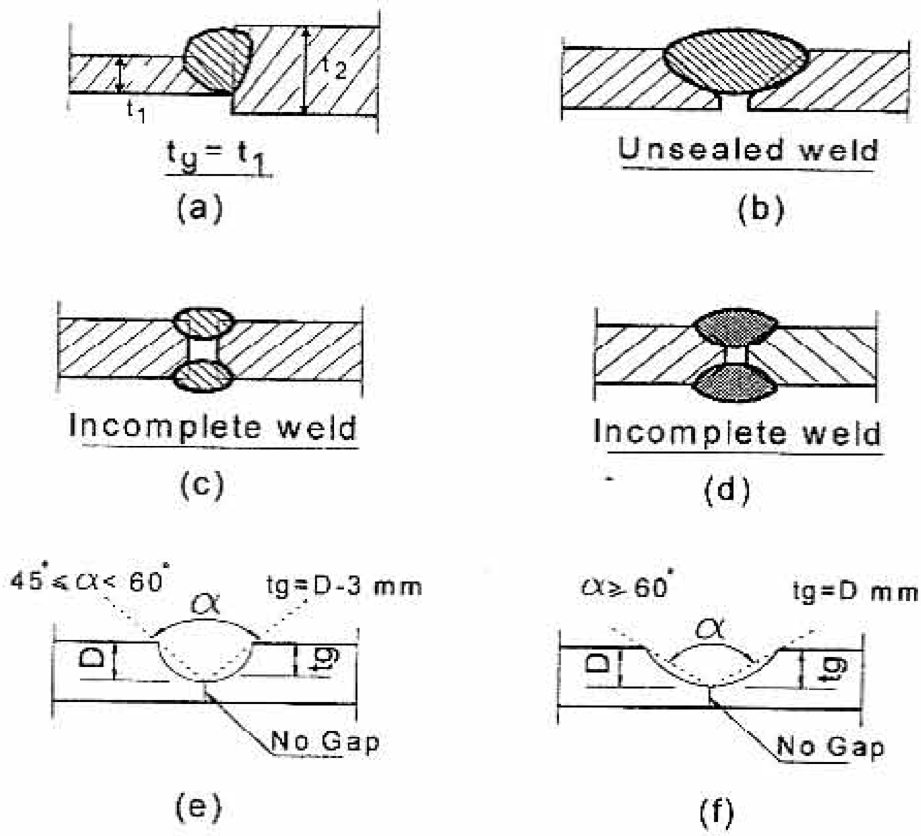


Figure 9.4 Details of Groove Weld

Table 9.4 Minimum Thickness Dimensions of Partial Joint Penetration Groove Weld

Material thickness of thicker part joined (greater of t_1 or t_2)	Up to 6 mm	Over 6 to 12	Over 12 to 18	Over 18 to 38	Over 38 to 56	Over 56 to 150	Over 150
Minimum effective thickness dimension (t_g) mm	3	5	6	8	9	12	16

9.5.3 Design Strength of Butt (Groove) Welds
9.5.3.1 Matching Filler Metal Requirements

The electrode material should have the properties of the base metal. When properties are comparable, the weld metal is referred to as "Matching" weld metal.

For complete and partial penetration groove welds, it is allowed to use weld metal with a strength level equal to "Matching" weld or one strength level stronger (see Table 9.2).

9.5.3.2 Design Strength of Butt (Groove) Welds

The Load and Resistance Factor Design concept which was previously given in Chapter (1), where Equation (1.7) gives the structural safety requirements for structural elements is also applicable to welds as follows:

$$\phi\ R_n\ \geq \gamma_i\ Q_i$$

Where:

ϕ = *shear*, tension or compression resistance weld reduction factor

R_n = *shear*, tension or compression nominal weld strength per unit length = R_{nw} for welds

γ_i = loads *factors*

Q_i = *service* loads such as bending moment, shearing force, axial force and torsional moment resulting from various loads

$\gamma_i\, Q_i$ = R_{uw} = factored load per unit length of weld which corresponds to the applied straining action

9.5.3.3 Tension, Compression and Shear Design Strength of Complete Penetration Groove Welds

a- Tension and compression normal to the effective area and tension and compression parallel to axis of weld (per unit length) must follow the relations below:

$$\phi \; R_{nw} = 0.85 \, t_b \, F_y \quad \text{t/cm} \qquad \text{9.1}$$

Where:

ϕ = 0.85

t_b = base plate metal thickness

F_y = the yield stress of the base metal, t/cm^2

for Hybrid sections F_y is the smaller yield stress for the connected elements t/cm^2

b- The shear strength on the effective area per unit length shall be as follows:

$$\phi \; R_{nw} = 0.85 \, t_b \, (0.6 \, F_y) \qquad \text{9.2}$$

9.5.3.4 Tension, Compression and Shear Design Strength of Partial Penetration Groove Welds

Partial penetration groove welds are similarly treated as complete penetration groove welds. Equation 9.I and 9.2 shall be applied as given below.

a- Tension and compression strength

$$\phi \; R_{nw} = 0.85 \, t_g \, F_y \qquad \text{9.3}$$

b- Shear strength

$$\phi \; R_{nw} = 0.85 \, t_g \, (0.6 \, F_y) \qquad \text{9.4}$$

Where:

t_g = the actual penetrated weld depth, cm

9.5.4 Constructional Restrictions and Remarks

a- Single V and U groove welds shall be sealed, whenever possible by depositing a sealing run of weld metal on the back of the joint. Where this is not done, the strength in the weld shall be not more than one half of the corresponding permissible design strength indicated in Section 9.5.3.

b- In the case of single and double V and U butt weld 18 mm , and over in size, in dynamically loaded structures, the back of the first run shall be cut out to a depth of at least 4 mm , prior to the application of subsequent runs. The grooves thus formed and the roots of single V and U groove welds shall be filled in and sealed.

c- When it is impossible to deposit a sealing run of weld metal on the back of the joint, then provided that backing material is in contact with the back of the joint, and provided also that the steel parts are bevelled to an edge with a gap not less than 3 mm and not more than 5 mm, to ensure fusion into the root of the V and the backing material at the back of the joint, strength may be taken as specified in Section 9.5.3.3.

d- Possible defects that may result in discontinuities within the weld are to be avoided. Some of the more common defects are: incomplete fusion, inadequate joint penetration, porosity, undercutting, inclusion of slag and cracks (refer to Section 9.8)

e- i- Butt welds shall be built up so that the thickness of the reinforcement at the center of the weld is not less than the following:

- Butt welds $\leq$ 30 mm in size reinforce by 10%
- Butt welds > 30 mm in size reinforce by 3mm

ii- Where flush surface is required, specially in dynamic loading, the butt weld shall be built up as given in (a) and then dressed flush.

9.6 DESIGN, STRENGTH AND LIMITATIONS OF FILLET WELDED CONNECTIONS

9.6.1 Nomenclature of the Common Terms

Figure 9.5 shows the nomenclature of the common terms for fillet welds.

9.6.2 Different Types of Fillet Welded Connections

Fillet welds are made between plates surfaces which are usually at right angles, but the angle between the plates may vary from 60^{O} to 120^{O}. Tee joints, corner welds and cruciform joints are all combinations of fillet welds and are as shown in Fig. 9.6.

The ideal fillet is normally of the mitre shape which is an isosceles triangle as shown in Fig. 9.7(h). The mitre and convex welds are stronger than a concave fillet weld of the same leg length when the weld is subject to static loadings, but the concave is stronger when subject to dynamic loadings.

Vertical welds made upwards in one run, are generally convex. Usually low currents produce the convex welds.

The penetration of the weld should reach the root where the contour of penetration is usually as shown in Fig. 9.7(g).

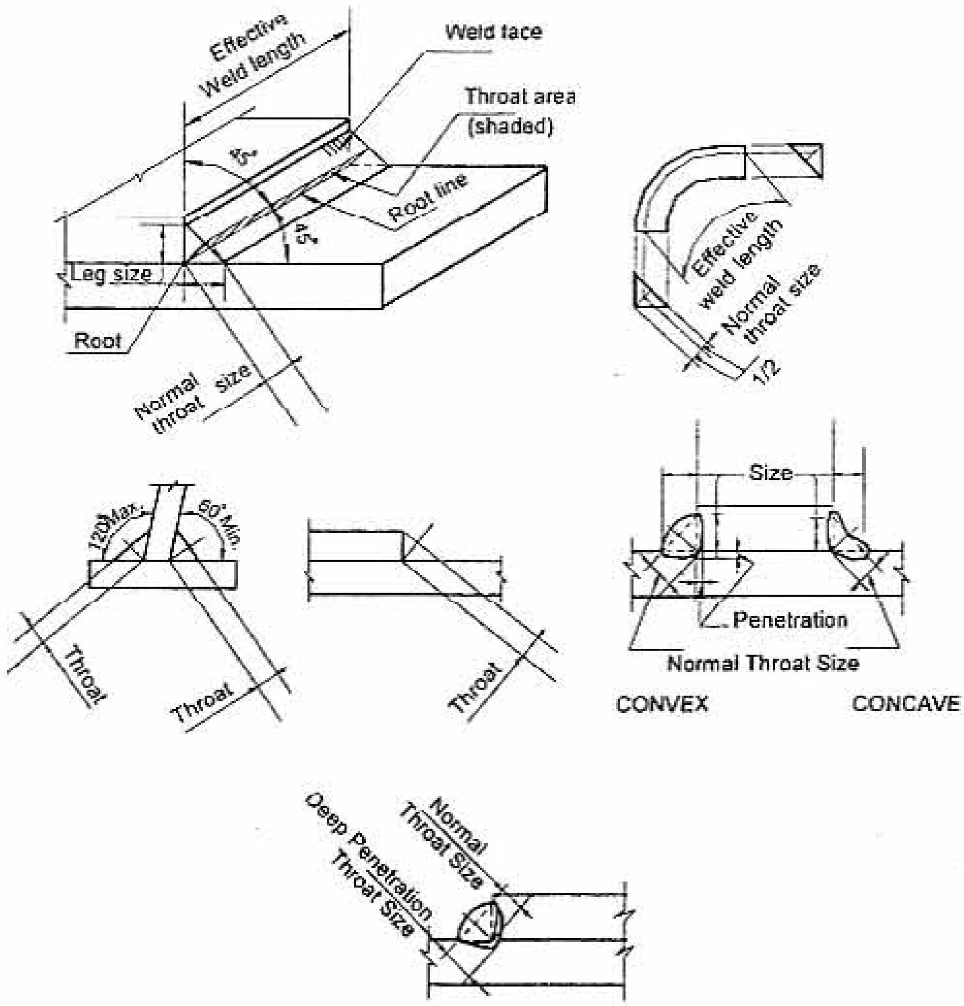


Figure 9.5 Fillet Weld Nomenclature

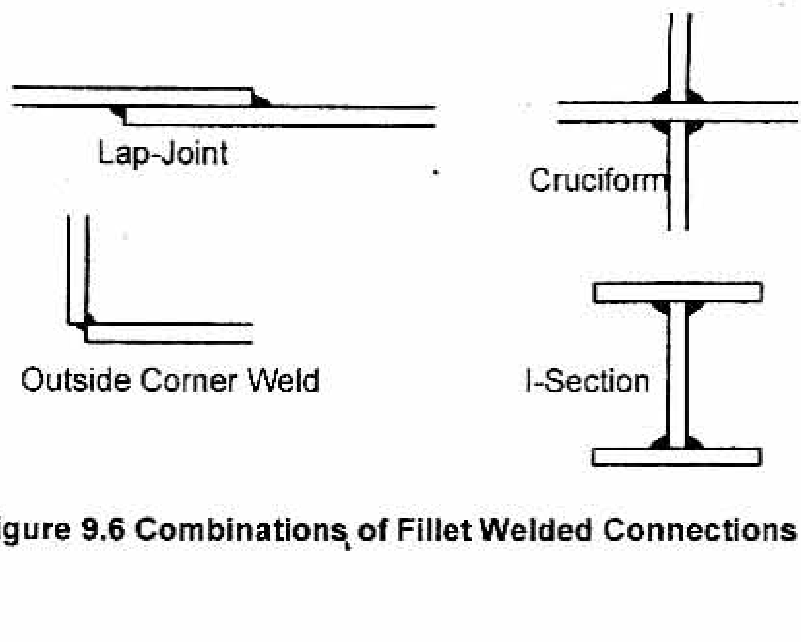


Figure 9.6 Combinations of Fillet Welded Connections

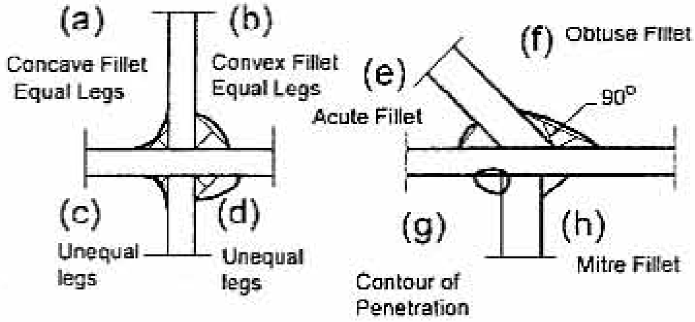


Figure 9.7 Fillet Weld Configurations

9.6.2.1 Effective Area of Fillet Welds

The effective weld section is equal to the largest triangle which can be inscribed between the fusion surfaces and the weld surface, provided there is a minimum root penetration, which is not taken into account. The effective throat (*a*) is then the distance from the root to the surface of the isosceles triangular weld along the line bisecting the root angle as shown in Fig. 9.8.

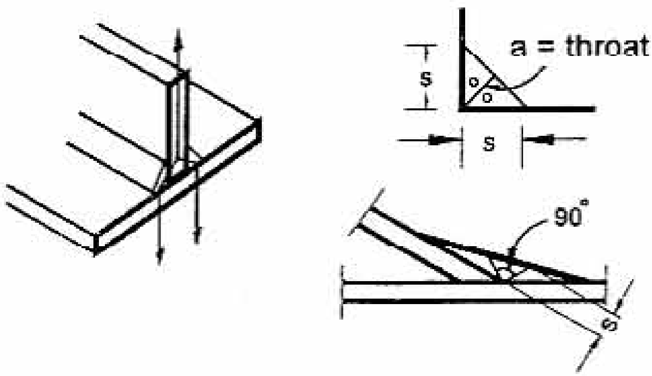


Figure 9.8 Dimensions of Size and Throat of Fillet Weld

Fillet welds are stressed across the throat of the weld, while their size (*s*) is specified by the leg length (*s*) where:

$$a = K.s \qquad 9.5$$

The value of "*K*" depends on the angle between the fusion faces and it may be taken as follows:

Table 9.5 Values of K Relating Size and Throat of Weld

Degree	60° – 90°	91° – 100°	101° – 106°	107°– 113°	114°– 120°
K	0.7	0.65	0.6	0.55	0.5

9.6.3 Limitations

a- Deposited fillet weld metal is related to the limiting angles between the fusion faces that shall not be greater than 120 ° nor less than 60 ° for flat and down hand welding, 70 ° for vertical welding and 80 ° for overhead welding. The throat of a fillet weld as deposited shall be not less than 6/10 and 9/10 of the minimum leg length in the case of concave and convex fillets respectively as shown in Fig. 9.9.

b- The minimum size of fillet welds shall be not less than the size required to transmit calculated forces nor the size as shown in Table 9.6 which is based upon experiences and provides some margin for uncalculated stresses encountered during fabrication, handling, transportation and erection. The limitations of Table 9.6 are based upon the quench effect of thick material on small welds. Very rapid cooling of weld metal may result in a loss of ductility. Further, the restraint to weld-metal shrinkage provided by thick material may result in weld cracking.

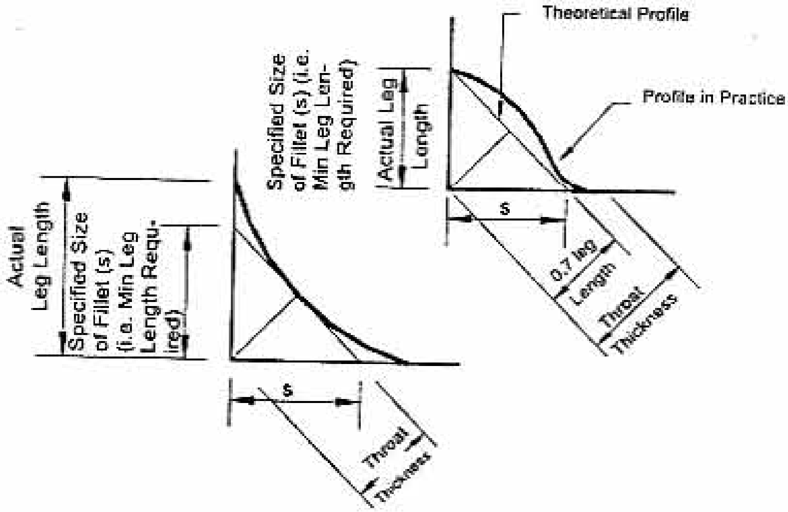


Figure 9.9 Deposited Fillet Weld Metal

Table 9.6 Minimum Size of Fillet Welds

Material thickness of thicker part joined (mms)	To 6 mm inclusive	Over 6 to 12	Over 12 to 18	Over 18
Minimum size of fillet weld (s) mms	4	5	6	8

- For flange-web welds and similar connections, the actual weld size need not be larger than that required to develop the web capacity and the requirements of Table 9.6 shall not apply.

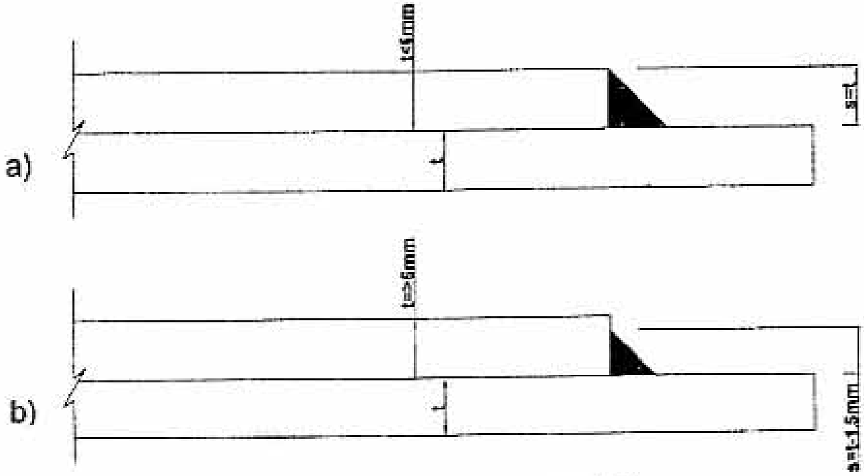


Figure 9.10 Maximum Weld Size

c- The Maximum size of fillet weld should be as follows:
  i- The maximum size of fillet weld(s) should not exceed the thinner thickness of the connected plates for longitudinal and edge fillet weld.
  ii- At long edges of material less than 6 mms thick the size (s) should not be greater than the thickness of the material (see Fig. 9.10 a).
  iii- At long edges of materiel 6 mms or more in thickness, the size(s) should not be greater than the thickness of the material minus 1.5 mms, unless the weld is especially designated on the drawings to be built out to obtain full-throat thickness, (see Fig. 9.10(b)).

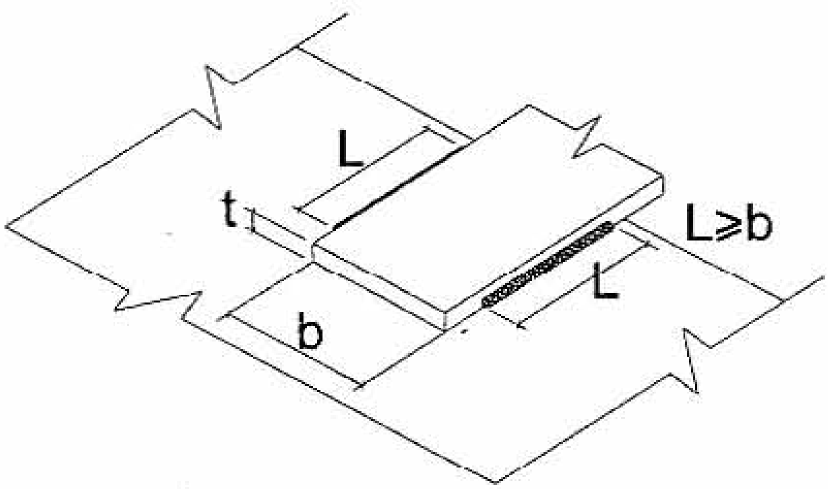


Figure 9.11 Longitudinal Fillet Weld

d- The minimum effective length of fillet welds designed on the basis of strength shall be not less than four times the weld size(s) or 5 cms whichever is largest.
e- When longitudinal fillet welds are used alone in a connection (see Fig. 9.11), the length of each weld should be at least equal to the width of the connecting material because of shear lag.
f- The maximum effective length of fillet weld loaded by forces parallel to the weld, such as lap splices and gusseted truss connections, shall not exceed

70 times the fillet weld size(s). A uniform stress distribution may be assumed throughout the maximum effective length. Generally in lap joints longer than 70s a reduction factor β-allowing for the effects of non-uniform distribution of stress along its length is to be utilized according to the following relation:

$$\beta = 1.2 - 0.2L / (70\ s) \quad \text{......} \quad 9.6$$

Where,

L = overall length of the fillet weld

g- There are No limitations for the length of fillet weld for beam to column connections as well as for the flange to web weld in welded built up plate girders (see Fig. 9.12 (a) and (b)).

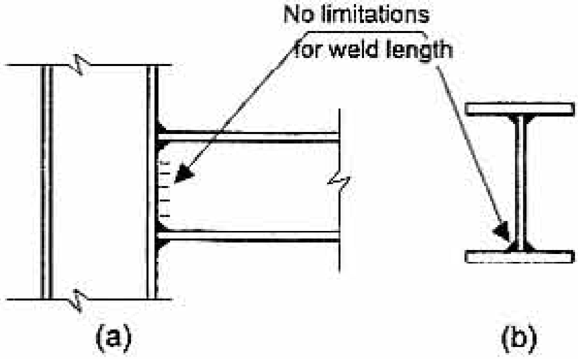


Figure 9.12 Different Locations with No Limitations

h- In lap joints, the minimum amount of lap shall be five times the thickness of the thinner part joined but not less than 5.0 cms, so that the resulting rotation when pulled will not be excessive as shown in Fig. 9.13. In addition lap joints joining plate, or bars subjected to axial stresses shall be fillet welded along the end of both lapped bars except where the deflection of the lapped parts is sufficiently restrained to prevent opening of the joint under maximum loading.

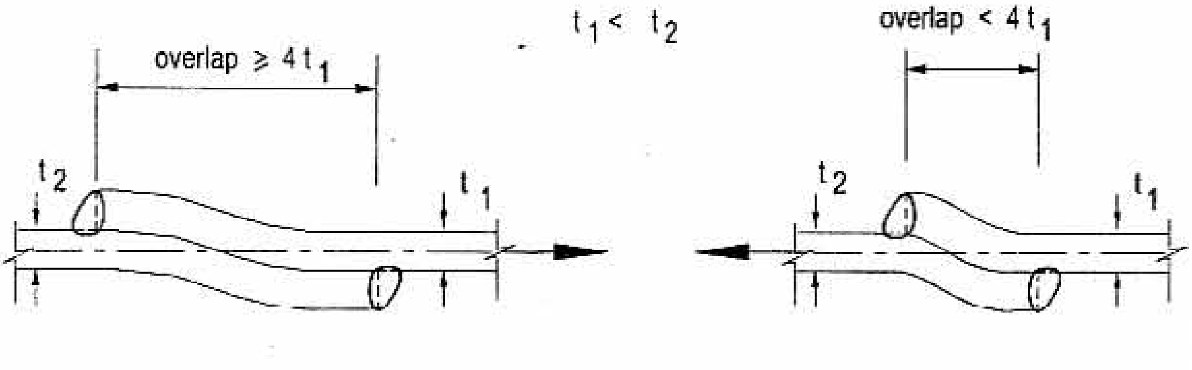


Figure 9.13 Minimum Lap to Minimize Excessive Rotation

i- Fillet weld terminations :

i- Fillet weld terminations shall not be at the extreme ends or sides of parts or members. They shall be:

- either returned continuously around the ends or sides respectively for a distance not less than two times the nominal weld size (see Fig. 9.14(a)).
- or shall terminate not less than two times the nominal weld size as shown in Fig. 9.14(b).

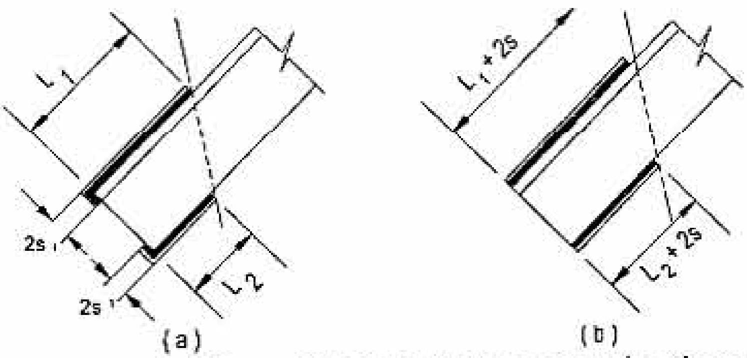


Figure 9.14 Fillet Weld Terminations

ii- For details and structural elements subject to cyclic (fatigue), out of plane forces and/or moments of frequency and magnitude that would tend to initiate a progressive failure of the weld, filler welds shall be returned around the side or end for a distance not less than two times the nominal weld size. The common connections where this limitation applies are: brackets, beam seats, framing angles, simple end plates and similar connections.

iii- For framing angles and simple end-plate connections which depend upon flexibility of the outstanding legs for connection flexibility, if end returns are used, their length shall not exceed four times the nominal size of the weld.

iv- Fillet welds which occur on opposite sides of a common plane shall be interrupted at the corner common to both welds.

v- End returns shall be indicated on the design and detail drawings.

j- Intermittent fillet welds may be used to transfer calculated stress across a joint of faying surfaces when the strength required is less than that developed by a continuous fillet weld of the smallest permitted size, and join components of built up members. The following limitations are to be satisfied:

i- Intermittent welds shall not be used in parts intended to transmit stresses in dynamically loaded structures.

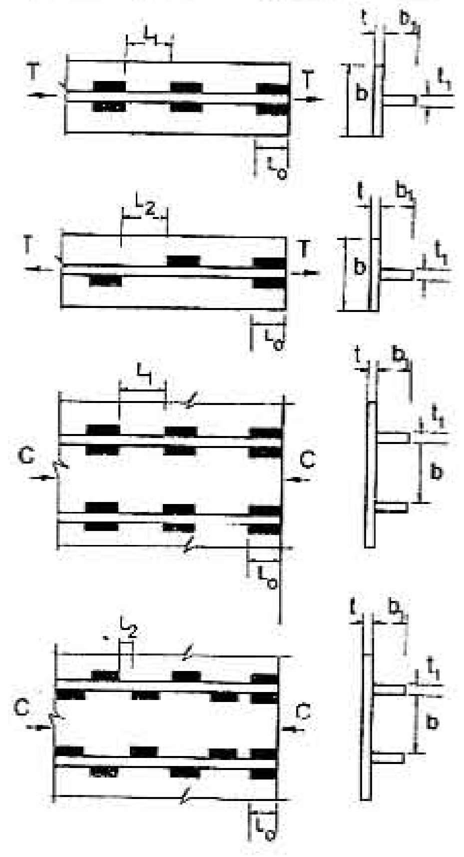
ii- The effective length of any segment of intermittent fillet welding shall not be less than four times the weld size with a minimum of 4.0 mms.

iii- The clear distance between effective lengths of consecutive intermittent fillets whether chained (L_1) or staggered (L_2) shall not exceed 12 times the thickness of the thinner part in compression or 16 times in tension and in no case shall it exceed 20 cms (see Fig. 9.15).

iv- In a line of intermittent fillet welds, the welding shall extend to the ends of the connected parts for staggered welds. This applies generally to both edges but need not apply to subsidiary fittings or components such as intermediate stiffeners.

v- For a member in which plates are connected by means of intermittent fillet welds, a continuous fillet weld shall be provided on each side of the plate for a length (L_0) at each end equal to at least three quarters of the width of the narrower plate connected (see Fig. 9.15).

vi- Stiffeners and girder connections are permitted to be directly welded with the compression flange. In the case of the tension flange, intermediate plates (not welded to the flange) shall be inserted between the flange and the stiffener in order to prevent weakening of the flange by transverse welds. Where intermittent welds are used the clear distance between consecutive welds, whether chained or staggered shall not exceed 16 times the thickness of the stiffener. The effective length of such weld shall not be less than 10 times the thickness of the stiffener in the case of staggered welds and 4 times in the case of chained welds, or one quarter the distance between stiffeners whichever is smallest.



- $L_O \geq 0.75\,b$ or $0.75\,b_1$ – whichever is smaller

- (L_1) or $(L_2) \leq 16\,t$ or $16\,t_1$ or 200 mm - whichever is smallest (Tension)

- (L_1)or $(L_2) \leq 12t$ or $12t_1$ or 200 mm-whichever is smallest (Compression)

Figure 9.15 Intermittent Fillet Welds

k- Single side fillet welds:

i- Single side fillet welds subjected to normal tensile stresses perpendicular to the longitudinal direction of the welds are not allowed.

ii- Single side fillet welds between the flanges and the web in I girder shall be made with a penetration of at least half the web thickness.

iii- For the single side fillet flange – to – web weld, thin fillet weld shall be completed on the other side of the web and made symmetrical at supports, and at the positions of concentrated loads where the web is not stiffened by vertical stiffeners.

9.6.4 Design Strength and Matching Weld Metal of Fillet Weld

9.6.4.1 Matching Filler Metal Requirements

For fillet welded connections the electrode material used must have at least the properties of the base metal, Table 9.2 is to be utilized for the choice of the "MATCHING" electrode. The use of weld metal with a strength less than "matching" weld metal is not allowed while the use of electrode one strength level greater than Matching may be used.

9.6.4.2 Design Strength of Fillet Weld

The factored load per unit length of weld ($R_u = \gamma_l \phi_i$) in a fillet weld loaded in an arbitrary direction can be resolved into the following components:

$(R_{uw})_{t\perp}$ = the normal factored load perpendicular to the axis of weld per unit length of weld.

$(R_{uw})_{q//}$ = the shear factored load parallel to the axis of weld per unit length of weld.

$(R_{uw})_{q\perp}$ = the shear factored load perpendicular to the axis of weld per unit length of weld.

These factored loads shall be related to the size(s) of the legs of the isosceles triangle inscribed in the weld seam if the angle between the two surfaces to be welded is between 60° and 90°. When this angle is greater than 90° the size of the leg of the inscribed right angle isosceles triangle shall be taken.

The design strength per unit length of fillet weld for all kinds of factored loads is as follows:

$$\phi\ R_{uw} = 0.7\, s\, (0.4\, F_u) \quad \text{........} \quad 9.7$$

Where:

ϕ = shear, tension or compression resistance weld reduction factor = 0.7

R_{uw} = shear, tension or compression nominal weld strength per unit length, t/cm

s = leg dimension of fillet weld (size), cm

F_u = ultimate tensile strength of the base metal, t/cm^2

for Hybrid sections F_u is the smaller ultimate strength for the connected elements

In case where welds are subjected simultaneously to normal and shear factored loads, they shall be checked for the corresponding principal stresses. For this combination of stresses, an effective factored load value $(R_{uw})_{eff}$ per unit length may be utilized and the corresponding design weld strength is to be increased by 10% as follows:

$$(R_{uw})_{eff} = \sqrt{(R_{uw})^2_{f\perp} + 3[(R_{uw})^2_{q//} + (R_{uw})^2_{q\perp}]} \leq 0.77\, s(0.4F_u) \quad \text{.....9.8}$$

The effective length of a fillet weld is usually taken as the overall length of the weld minus twice the weld size(s) as deduction for end craters.

9.7 PLUG AND SLOT WELDS

The stress transfer of plug and slot welds is limited to resisting shear loads in joints at plane parallel to the faying surface. The strength capacity is calculated as the product of the area of the hole or slot times the ultimate shear stress as follows:

$$\phi\ R_{nw} = 0.7\ (\text{Area of hole or slot}).(0.6\ F_u) \quad \text{..........9.9}$$

The proportions and spacing of holes and slots and the depth are illustrated in Fig. (9.16) and the corresponding limitations are as given in Table (9.7).

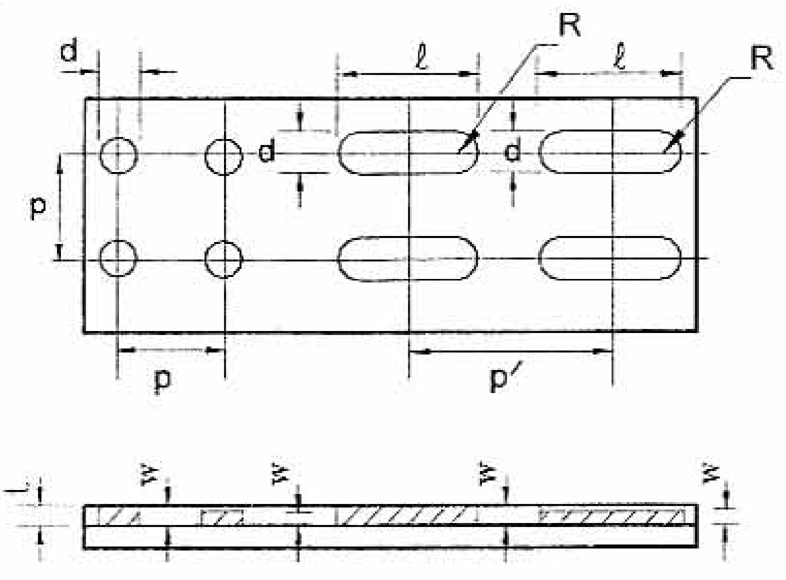


Figure 9.16 Definitions of Plug and Slot Welds

Table 9.7 Plug and Slot Welds Limitations

<table>
<tr><th>Plate Thicknes t (mm)</th><th>Min. Hole diameter or Slot width d_{min} (mm)</th><th colspan="2">Hole and Slot Proportions Spacing and Depth of Weld</th></tr>
<tr><td>5 & 6</td><td>14</td><td colspan="2">$d \geq (t+8)$mm preferably rounded to next higher even number; also $d \leq 2.25\ t$ or d_{min}+3mm whichever is greater</td></tr>
<tr><td>7&8</td><td>16</td><td rowspan="5">$p \geq 4d$
$p/ \geq 2\ell$
$\ell \leq 10\ w$
$R = d/2$
$R \geq t$</td><td rowspan="5"><u>Depth of filling of plug and slot welds (w):</u>
Where $t \leq 16$ mm, $w = t$
Where $t > 16$ mm, $w > t/2$
but not less than 16 mm.</td></tr>
<tr><td>9 &10</td><td>18</td></tr>
<tr><td>11 & 12</td><td>20</td></tr>
<tr><td>13&14</td><td>22</td></tr>
<tr><td>15 &16</td><td>24</td></tr>
</table>

N.B. There are no limitations for the edge distances.

9.8 GENERAL RESTRICTIONS TO AVOID UNFAVOURABLE WELD DETAILS

9.8.1 Lamellar Tearing

Lamellar tearing is a separation (or crack) in the base metal, caused by through – thickness weld shrinkage strains. The probability of this failure can be minimized by:

a- Using small weld size providing the shrinkage to be accommodated.

b- The welding procedure should also establish a welding sequence such that the component restraint and the internal restraints in the weldment are held to a minimum.

c- The use of a welding procedure with low hydrogen weld and an effective preheating minimize lamellar tear.

Some joints susceptible to lamellar tearing can be improved by careful detailing as shown in Fig. 9.17.

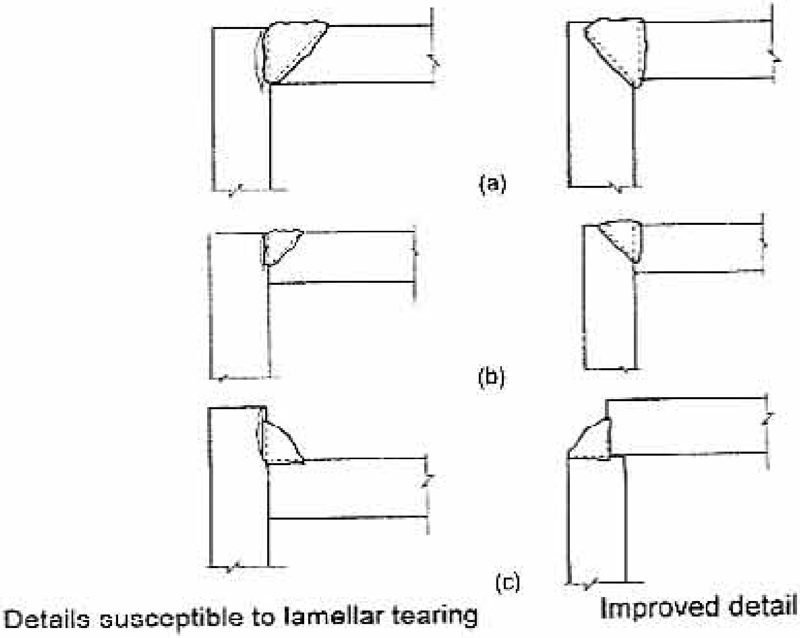


Figure 9.17 Improved Welded Connections to Reduce Lamellar Tear

9.8.2 Notches and Brittle Fracture

a- The one sided fillet welds can result in severe notches as shown in Fig. 9.18(a). The remedy is to use two fillets one on each side. A similar condition arises with partial penetration groove welds.

b- Backing bars can cause a fatigue weld notch if they are welded as shown in Fig. 9.18(b). A remedy would be to weld in the groove as in Fig. 9.18(c), where any undercut would be filled, or at least backed up by the final weld joint. The backing bars should also be continuous throughout its length.

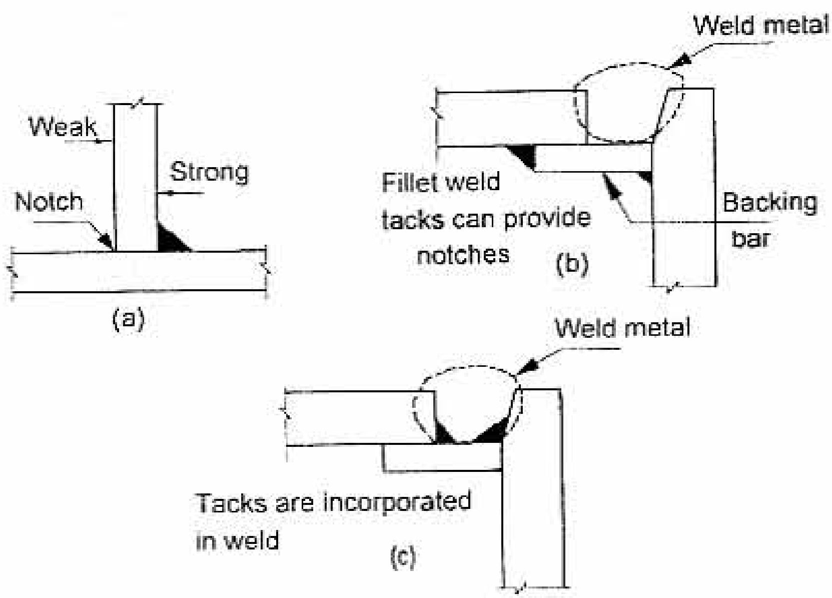


Figure 9.18 Notches and Brittle Fracture

9.9 WELD INSPECTION METHODS

The designer must specify in the contract document the type of weld inspection required as well as the extent and application of each type of inspection. Table 9.8 summarizes the characteristics and capabilities of the five most commonly used methods for welding inspection.

Table 9.8 Characteristics of Common Weld Inspection Methods

Inspection Method	Characteristics and Applications	Limitations
Visual (VT)	Most common, most economical. Particularly good for single pass.	Detects surface imperfections only.
Dye Penetrant (DPT)	Will detect tight cracks, open to surface.	Detects surface imperfections only. Deep weld ripples and scratches may give false indications.
Magnetic Particle (MT)	Will detect surface cracks and subsurface cracks to about 2 mm depth with proper magnetization. Indications can be preserved on clear plastic tape.	Requires relatively smooth surface. Careless use of magnetization prods may leave false indications.
Radiographic (RT)	Detects porosity, slag, voids, irregularities, lack of fusion. Film negative is permanent record.	Detects must occupy more than about 1.20% of thickness to register. Only cracks partial to impinging beam register. Radiation hazards Exposure time increases with thickness.
Ultrasonic (UT)	Detects cracks in any orientation, Slag, lack of fusion, inclusions, lamellar tears, voids. Can detect a favorably oriented planar reflector smaller than 1mm. Regularly calibrate on 1½ mm dia. drilled hole. Can scan almost any commercial thickness.	Surface must be smooth. Equipment must be frequently calibrated. Operator must be qualified. Exceedingly coarse grains will give false indications. Certain geometric configurations give false indication of flaws.

CHAPTER 10

DESIGN CONSIDERATIONS FOR MEMBERS SUBJECTED TO CONCENTRATED FORCES

This Chapter covers strength design considerations pertaining to flanges and webs of member section subjected to concentrated forces.

10.1 INTRODUCTION

When concentrated forces are applied to beams, beam bearing at supports and reactions of beam flanges at connections to columns; local effect of such forces on flanges and webs comprising member section in the vicinity of these concentrated forces should be considered. As in most compression-related situations there are two possibilities: yielding or instability.

In this Chapter, web local yielding, web crippling, flange local bending; web transverse compression buckling and sidesway buckling resulting from concentrated forces are treated. Also the transmission of concentrated forces in beam - to - column connections is covered.

10.1.1 Design Basis

Sections 10.2 through 10.7 apply to single and double concentrated forces as indicated in each section. A single concentrated force is tensile or compressive. Double concentrated forces, one tensile and one compressive, form a couple on the same side of the loaded member.

Transverse stiffeners are required at locations of concentrated tensile forces in accordance with Section 10.2 for the limit state of flange local bending, and at unframed ends of beams and girders in accordance with Section 10.8. Transverse stiffeners or doubler plates are required at locations of concentrated forces in accordance with Sections 10.3 through 10.6 for the limit states of web local yielding, crippling, side sway buckling, and compression buckling. Doubler plates or diagonal stiffeners are required in accordance with Section 10.7 for the limit state of panel-zone web shear.

Transverse stiffeners and diagonal stiffeners required by Sections 10.2 through 10.8 shall also meet the requirements of Section 10.9. Doubler plates required by Sections 10.3 through 10.6 shall also meet the requirements of Section 10.10.

10.2 FLANGE LOCAL BENDING

This Section applies to both tensile single-concentrated forces and the tensile component of double-concentrated forces. A pair of transverse stiffeners extending at least one-half the depth of the web shall be provided adjacent to a concentrated tensile force centrally applied across the flange when the required strength of the flange exceeds ϕR_n,

Where:

ϕ = 0.65

R_n = $6.25\, t_f^2\, F_{yf}$ 10.1

F_{yf} = specified minimum yield stress of the flange, t/cm^2

t_f = thickness of the loaded flange, cm

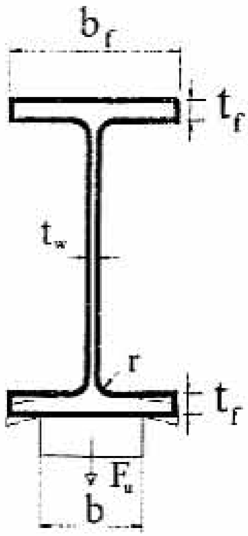


Figure 10.1 Flange Local Bending

If the concentrated force (F_u) is applied at a length b (Fig. 10.1) across the member flange less than $0.15b_f$, where b_f is the member flange width, Equation 10.1 need not be checked. When the concentrated force to be resisted is applied at a distance from the member end that is less than $10t_f$, R_n shall be reduced by 50 percent. When transverse stiffeners are required, they shall be welded to the loaded flange to develop the welded portion of the stiffener. The weld connecting transverse stiffeners to the web shall be sized to transmit the unbalanced force in the stiffener to the web. Also, refer to Section 10.9.

10.3 WEB LOCAL YIELDING

This section applies to single-concentrated forces and both components of double-concentrated forces. Either a pair of transverse stiffeners or a doubler plate, extending at least one-half the depth of the web, shall be provided adjacent to a concentrated tensile or compressive force when the required strength of the web at the toe of the fillet exceeds ϕR_n,

Where: $\phi = 0.95$

and R_n is determined as follows:

a- When the concentrated force to be resisted is applied at a distance from the member end that is greater than the depth of the member d, (Fig. 10.2)

$$R_n = (5k + N)F_{yw}\, t_w \quad \text{......} \quad 10.2$$

b- When the concentrated force to be resisted is applied at a distance from the member end that is less than or equal to the depth of the member d, (Fig. 10.2).

$$R_n = (2.5k + N)F_{yw}\, t_w \quad \text{......} \quad 10.3$$

In Equations 10.2 and 10.3, the following definitions apply:

F_{yw} = specified minimum yield stress of the web, t/cm^2

N = length of bearing (not less than k for end beam reactions), cm

k = distance from outer face of the flange to the web toe of the fillet, cm

t_w = web thickness, cm

When required, for a tensile force normal to the flange, transverse stiffeners shall be welded to the loaded flange to develop the connected portion of the stiffener. When required for a compressive force normal to the flange, transverse stiffeners shall either bear on or be welded to the loaded flange to develop the force transmitted to the stiffener. The weld connecting transverse stiffeners to the web shall be sized to transmit the unbalanced force in the stiffener to the web. Also, refer to Section 10.9. Alternatively, doubler plates can be used, refer to Section 10.10.

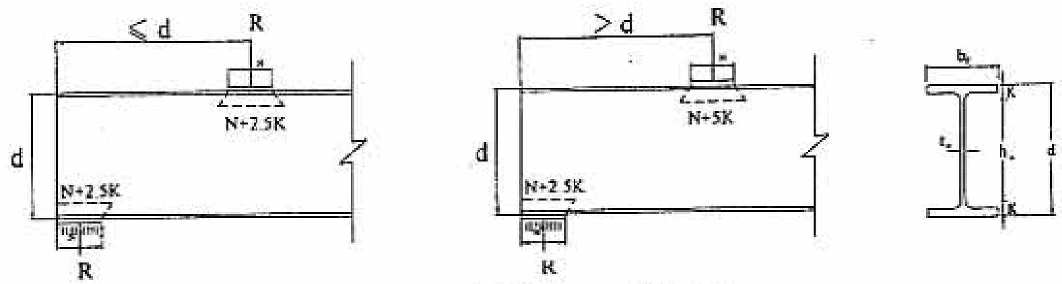


Figure 10.2 Web Local Yielding

10.4 WEB CRIPPLING

The web crippling shown in Figure 10.3 applies to both compressive single-concentrated forces and the compressive component of double-concentrated forces. Either a transverse stiffener, a pair of transverse stiffeners, or a doubler plate, extending at least one-half the depth of the web, shall be provided adjacent to a concentrated compressive force when the required strength of the web exceeds ϕR_n,

Where: $\phi = 0.70$

and R_n is determined as follows:

a- When the concentrated compressive force to be resisted is applied at a distance from the member end that is greater than or equal to $d/2$,

$$R_n = 0.3627\, t_w^2\ [1 + 3[N/d][t_w/t_f]^{1.5}][E\, F_{yw}\, t_f / t_w]^{0.5} \quad \text{........................ 10.4}$$

b- When the concentrated compressive force to be resisted is applied at a distance from the member end that is less than $d/2$,

- For $N/d \leq 0.2$,

$$R_n = 0.1813\, t_w^2\ [1 + 3\,[N/d][t_w/t_f]^{1.5}][E\, F_{yw}\, t_f\ / t_w]^{0.5} \quad \text{.................. 10.5a}$$

- For $N/d > 0.2$,

$$R_n = 0.1813\, t_w^2\ [1 + [4\, N/d - 0.2]\,[t_w/t_f]^{1.5}][E\, F_{yw}\, t_f / t_w]^{0.5} \quad \text{.............. 10.5b}$$

In Equations 10.4 and 10.5, the following definitions apply:

d = overall depth of the member, cm

t_f = flange thickness, cm

When transverse stiffeners are required, they shall either bear on or be welded to the loaded flange to develop the force transmitted to the stiffener. The weld connecting transverse stiffeners to the web shall be sized to transmit the unbalanced force in the stiffener to the web. Also, refer to Section 10.9. Alternatively, when doubler plates are required, refer to Section 10.10.

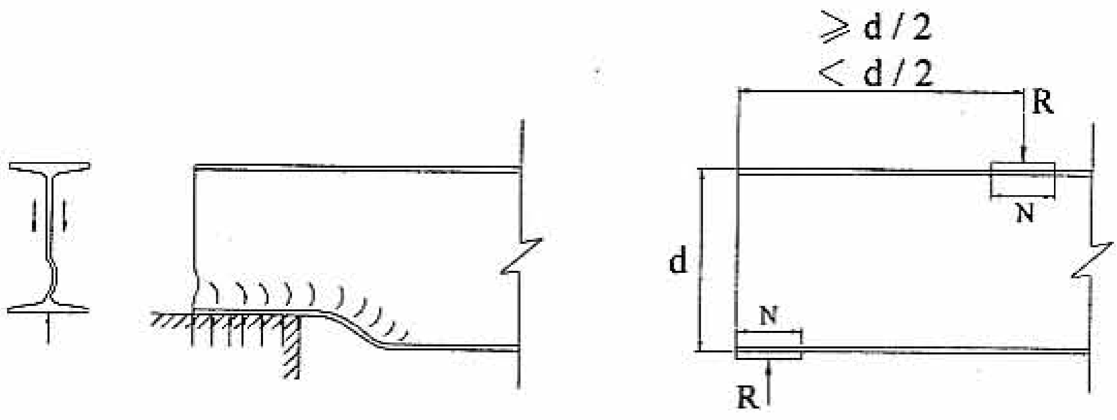


Figure 10.3 Web Crippling

10.5 WEB SIDESWAY BUCKLING

The web sidesway shown in Figure 10.4 applies only to compressive single-concentrated forces applied to members where relative lateral movement between the loaded compression flange and the tension flange is not restrained at the point of application of the concentrated force. The design strength of the web is ϕR_n.

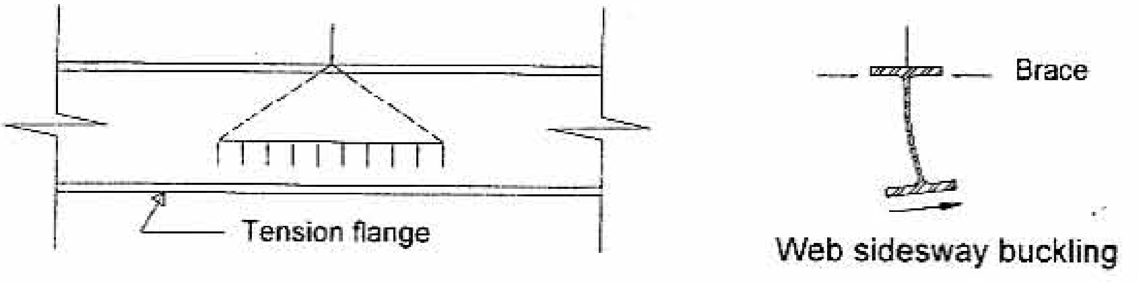


Figure 10.4 Web Sidesway Buckling

Where: $\phi = 0.80$

and ϕR_n is determined as follows:

a- If the compression flange is *restrained against rotation*:

- For $(h / t_w) / (\ell / b_f) \leq 2.3$,

$$R_n = [C_r\, t_w^3\, t_f / h^2]\,[1 + 0.4\{[h / t_w] / [\ell / b_f]\}^3] \qquad 10.6$$

- For $(h / t_w) / (\ell / b_f) > 2.3$, the limit state of sidesway web buckling does not apply.

When the required strength of the web exceeds ϕR_n, local lateral bracing shall be provided at the tension flange or either a pair of transverse stiffeners or a doubler plate, extending at least one-half the depth of the web, shall be provided adjacent to the concentrated compressive force.

When transverse stiffeners are required, they shall either bear on or be welded to the loaded flange to develop the full-applied force. The weld connecting transverse stiffeners to the web shall be sized to transmit the force in the stiffener to the web. Also, refer to Section 10.9. Alternatively, when doubler plates are required, they shall be sized to develop the full-applied force. Also, refer to Section 10.10.

b- If the compression flange is *not restrained against rotation*:

- For $(h / t_w) / (\ell / b_f) \leq 1.7$,

$$R_n = [C_r\, t_w^3\, t_f / h^2]\,[0.4\{[h / t_w] / [\ell / b_f]\}^3] \qquad 10.7$$

- For $(h / t_w) / (\ell / b_f) > 1.7$, the limit state of sidesway web buckling does not apply.

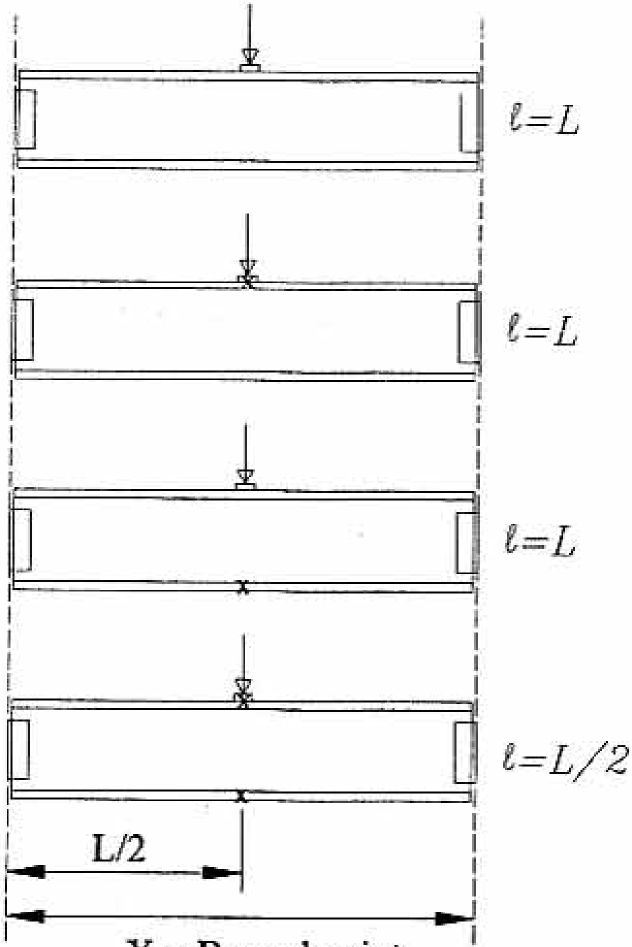


Figure 10.5 Unbraced Flange Length

When the required strength of the web exceeds ϕR_n, local lateral bracing shall be provided at both flanges at the point of application of the concentrated forces.

In Equations 10.6 and 10.7, the following definitions apply:

- ℓ = is as shown in Fig. 10.5; the largest laterally unbraced length along either flange at the point of load, cm
- b_f = tension flange width, cm
- t_f = tension flange thickness, cm
- t_w = web thickness, cm
- h = clear distance between flanges less the fillet or corner radius for rolled shapes; distance between adjacent lines of fasteners or the clear distance between flanges when welds are used for built-up shapes, cm
- C_r = ($6.62\text{x}10^2$ t/cm^2) when $M_u < M_y$ at the location of the applied force
  = ($3.31\text{x}10^2$ t/cm^2) when $M_u \geq M_y$ at the location of the applied force

10.6 WEB TRANSVERSE COMPRESSION BUCKLING (VERTICAL BUCKLING)

The web vertical buckling shown in Figure 10.6 applies to a pair of compressive single-concentrated forces or the compressive components in a pair of double-concentrated forces, applied at both flanges of a member at the same location. Either a single transverse stiffener, or pair of transverse stiffeners, or a doubler plate, extending the full depth of the web, shall be provided adjacent to concentrated compressive forces at both flanges when the required strength of the web exceeds ϕR_n,

$\phi = 0.85$

and

$$R_n = 0.165\{ t_w^{\,3} [E F_{yw}]^{0.5}\}/ h \qquad 10.8$$

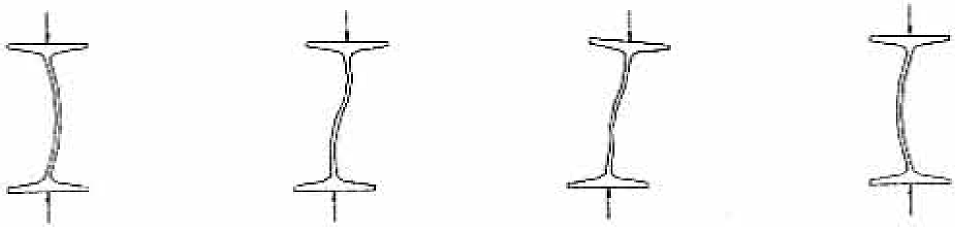


Figure 10.6 Web Vertical Buckling

When the pair of concentrated compressive forces to be resisted is applied at a distance from the member end that is less than $d/2$, R_n shall be reduced by 50 percent. When transverse stiffeners are required, they shall either bear on or be welded to the loaded flange to develop the force transmitted to the stiffener. The weld connecting transverse stiffeners to the web shall be sized to transmit the unbalanced force in the stiffener to the web. Also, refer to Section 10.9. Alternatively, when doubler plates are required, refer to Section 10.10.

10.7 PANEL-ZONE WEB SHEAR

The column web shear stresses may be high within the boundaries of the rigid connection of two or more members whose webs lie in a common plane. Such webs should be reinforced when the calculated factored force ΣF_u along plane A-A in Fig. (10.7) exceeds the column web design strength ϕR_v, where:

$$\Sigma F_u = (M_{u1}/d_{m1}) + (M_{u2}/d_{m2}) - V_u \qquad 10.9$$

And

M_{u1} = the sum of moments at end of member 1 due to factored gravity and lateral load

M_{u2} = the difference between moments at end of member 2 due to factored gravity and lateral load

Either doubler plates or diagonal stiffeners shall be provided within the boundaries of the rigid connection of members whose webs lie in a common plane when the required strength exceeds ϕR_v.

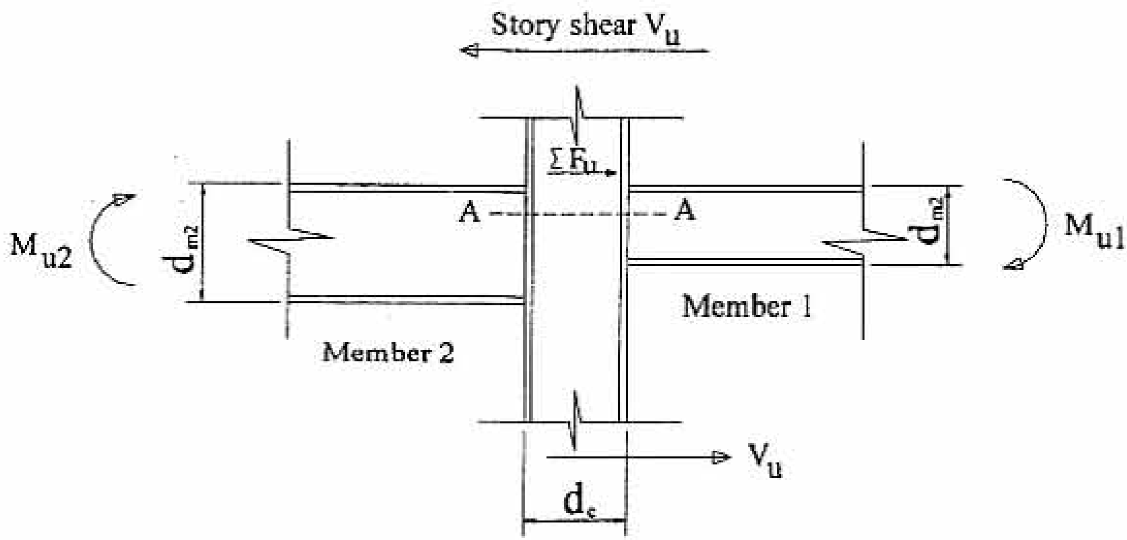


Figure 10.7 Forces in Panel Zone

Where: $\phi = 0.85$
and R_v is determined as follows:

a- When the effect of panel-zone deformation on frame stability is not considered in the analysis (elastic first order analysis), refer to Figure 10.8.

- For $P_u \le 0.4P_y$

$$R_v = 0.60F_y\, d_c\, t_w \quad \text{.......... 10.10}$$

- For $P_u > 0.4P_y$

$$R_v = 0.60F_y\, d_c\, t_w\, [1.4 - (P_u/P_y)] \quad \text{.......... 10.11}$$

b- When frame stability, including plastic panel-zone deformation, is considered in the analysis, refer to Figure 10.9.

- For $P_u \le 0.75P_y$

$$R_v = 0.60F_y\, d_c\, t_w\, [1+3\, (b_{cf}\, t_{cf}^2 / d_b\, d_c\, t_w)] \quad \text{.......... 10.12}$$

- For $P_u > 0.75P_y$

$$R_v = 0.60F_y\, d_c\, t_w\, [1+3\, (b_{cf}\, t_{cf}^2 / d_b\, d_c\, t_w)]\, [1.9 - 1.2(P_u/P_y)] \quad \text{.......... 10.13}$$

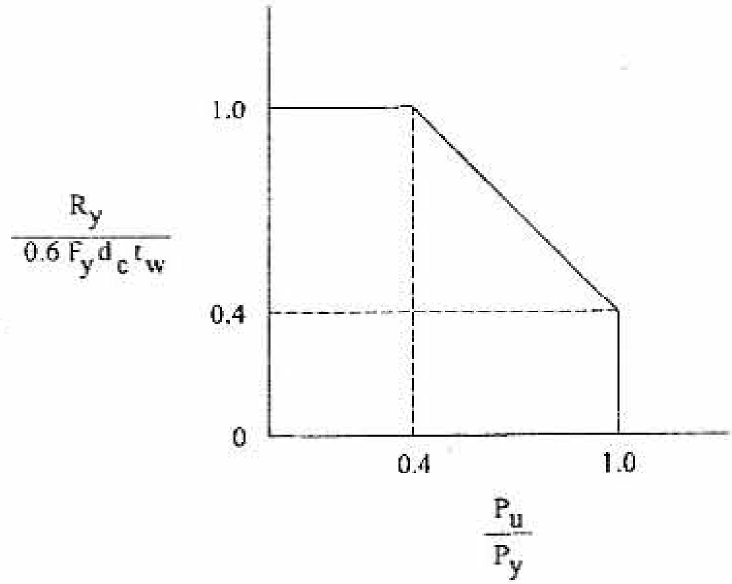


Figure 10.8 Interactions of Shear and Axial Force in a Panel Zone- Elastic

In Equations 10.10 through 10.13, the following definitions apply:

t_w = column web thickness, cm

b_{cf} - width of column flange, cm

t_{cf} = thickness of the column flange, cm

d_c = column depth, cm

d_b = beam depth, cm

F_y = yield strength of the column web, t/cm^2

P_y = $F_y A$, axial yield strength of the column, ton

P_u = factored axial load of the column, ton

A = column cross-sectional area, cm^2

When doubler plates are required, they shall meet the criteria applicable to members subject to major axis bending and shall be welded to develop the proportion of the total shear force which is to be carried. Alternatively, when diagonal stiffeners are required, the weld connecting diagonal stiffeners to the web shall be sized to transmit the stiffener force caused by unbalanced moments to the web. Also, refer to Section 10.9.

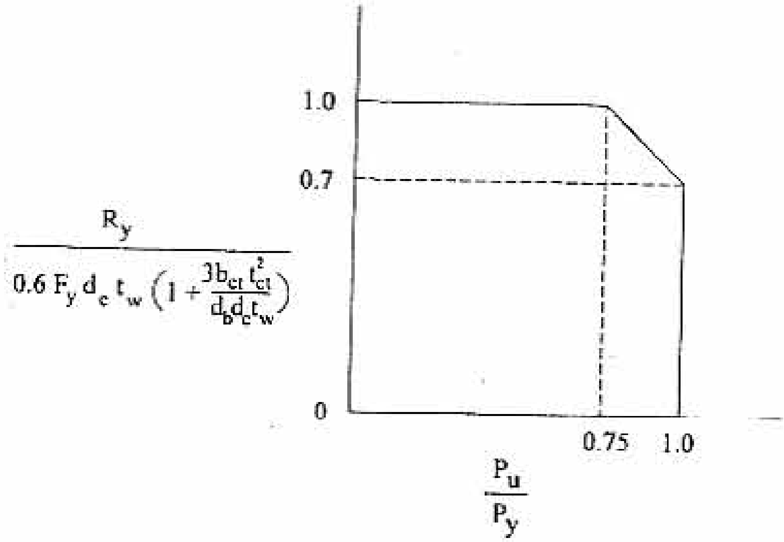


Figure 10.9 Interaction of Shear and Axial Force in a Panel Zone- Inelastic

10.8 UNFRAMED ENDS OF BEAMS AND GIRDERS

At unframed ends of beams and girders not otherwise restrained against rotation about their longitudinal axes, a pair of transverse stiffeners, extending the full depth of the web, shall be provided. Also, refer to Section 10.9.

10.9 ADDITIONAL STIFFENER REQUIREMENTS FOR CONCENTRATED FORCES

Web local yielding at the beam to column connection should be checked according to section 10.3 as shown in Fig. 8.9. If the conditions are not satisfied, transverse stiffeners can be used (see Fig. 8.9d & Fig. 10.10) that should satisfy the following criteria:

$$b_{st} + t_w / 2 \ge b_f / 3 \qquad \text{.......... 10.14}$$

$$t_s \ge t_f / 2 \qquad \text{.......... 10.15}$$

$$2 b_{st}\, t_{st} \ge b_b\, t_b - \left(t_b + 2t_p + 5k\right) t_{wc} \qquad \text{.......... 10.16}$$

Where:

b_{st} = Stiffeners width, cm

t_w = Column Web thickness, cm

b_f = Beam flange width, cm

t_{st} = Stiffeners width thickness, cm

t_f = Beam flange thickness, cm

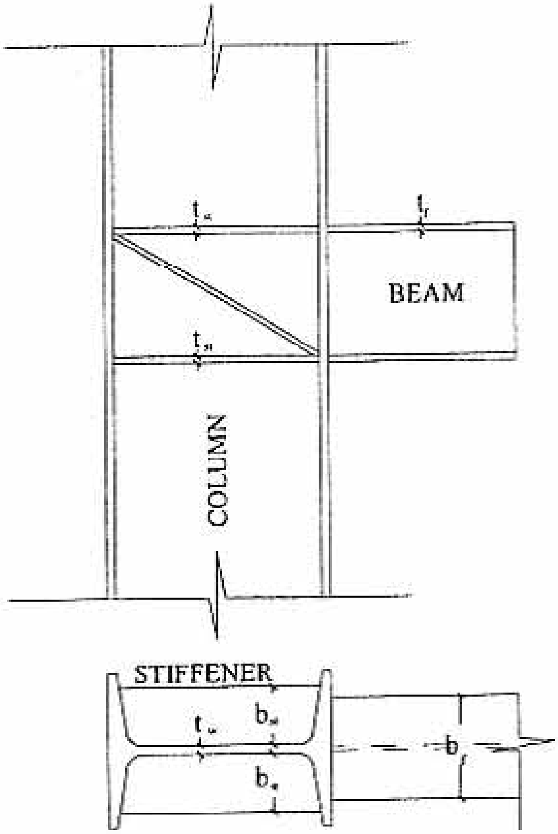


Figure 10.10 Beam to Column Connection

- In order to prevent the local buckling of these stiffeners, the following equation should be fulfilled:

$$b_{st} / t_{st} \leq 25 / \sqrt{F_y} \qquad \text{.......... 10.17}$$

- In case of using diagonal stiffeners in the direction of the diagonal compression (see Fig. 8.9d & Fig. 10.10), the following equation should also be fulfilled:

$$2\, b_{st}\, t_{st} = \left[(M / d_b) - (0.35\, F_y) h_c\, t_{wc}\right] / (0.58\, F_y \cos\theta) \qquad \text{.......... 10.18}$$

Full depth transverse stiffeners for compressive forces applied to a beam or plate girder flange *shall* be designed as axially compressed members (columns) in accordance with the design requirements for flexural buckling of compression members, with an effective length of $0.8h_w$, a cross section composed of two stiffeners and a strip of the web having a width of 25 t_w at interior stiffeners and $12t_w$ at the ends of members, (see Fig. 10.11). The weld connecting bearing stiffeners to the web shall be sized to transmit the excess web shear force to the stiffener.

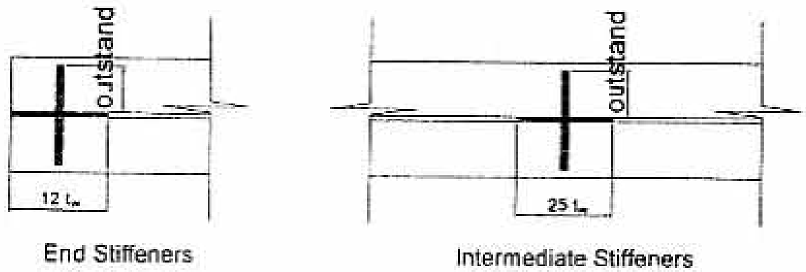


Figure 10.11 Transverse Stiffeners

10.10 ADDITIONAL DOUBLER PLATE REQUIREMENTS FOR CONCENTRATED FORCES

Doubler plates required by Sections 10.3 through 10.6 shall also comply with the following criteria:

a- The thickness and extent of the doubler plate shall provide the additional material necessary to equal or exceed the strength requirements.
b- The doubler plate shall be welded to the flanges of the section in order to develop the proportion of the total force transmitted to the doubler plate.

CHAPTER 11

FATIGUE

11.1 SCOPE

11.1.1 General

This chapter presents a general method for the fatigue assessment of structures and structural elements that are subjected to repeated fluctuations of stresses.

Members subjected to cyclic loading (Fatigue) should satisfy the following:

a- The ultimate strength requirements due to factored loads according to the provisions of the previous chapters.
b- The requirements of cyclic loading (Fatigue) due to unfactored (service) loads according to the provisions of this chapter.

Members subjected to stresses resulting from wind or earthquake forces only, need not be designed for Fatigue requirements. The occurrence of full design wind or earthquake loads is too infrequent to warrant consideration in fatigue design.

The provisions of this chapter are applicable to structures with suitable corrosion protection or subject only to mildly corrosive atmospheres, such as normal atmospheric conditions.

The cyclic load resistance determined by the provisions of this chapter are only applicable to structures subject to temperatures not exceeding 150°C.

Cracks that may form in fluctuating compression regions are self-arresting. Therefore, these compression regions are not subjected to fatigue failure.

11.1.2 Definitions

Fatigue: Damage in a structural member through gradual crack propagation caused by repeated stress fluctuations.

Design Life: The period in which a structure is required to perform safely with an acceptable probability that it will not fail or require repair.

Stress Range: The algebraic difference between two extreme values or nominal stresses due to fatigue loads. This may be determined through standard elastic analysis.

Fatigue strength: The stress range determined form test data for a given number of stress cycles.

Fatigue limit: The maximum stress range for constant amplitude cycles that will not form fatigue cracks.

Detail category: The designation given to a particular joint or welded detail to indicate its fatigue strength. The category takes into consideration the local stress concentration at the detail, the size and shape of the maximum acceptable discontinuity, the loading condition, metallurgical effects, residual stresses, fatigue cracks shapes, the welding procedure, and any post-welding improvement.

11.2 BASIC PRINCIPLES

11.2.1 General

a- The differences in fatigue strength between grades of steel are small and may be neglected.

b- The differences in fatigue damage between stress cycles having different values of mean stress but the same value of stress range may be neglected.

c Cracks generally occur at welds or at stress concentration due to sudden changes of cross-sections. Very significant improvements in fatigue strength can be achieved by reducing the severity of stress concentrations at such points.

d- When fatigue influences the design of a structure, details should be precisely defined by the designer and should not be amended in any way without the designer's prior approval. Similarly, no attachments or cutouts should be added to any part of the structure without notifying the designer.

e- Structures in which the failure of a single element could result in a collapse or catastrophic failure should receive special attention when fatigue cracks are a possibility. In such cases, the nominal fatigue stress ranges shall be limited to 0.8 times the values given in Table 11.3 or in Figure 11.1.

f- Slotted holes shall not be used in bolted connections for members subjected to fatigue.

11.2.2 Factors Affecting Fatigue Strength

The fatigue strength of the structural elements depends upon:

a- The applied stress range.

b- The detail category of the particular structural component or joint design.

c- The number of stress cycles.

11.2.3 Fatigue Loads

For cranes and supporting structures for machinery and equipment, the fatigue loads used to calculate the maximum applied stress range are the full specified traveling crane, machinery, or equipment load including dynamic effect.

11.2.4 Fatigue Assessment Procedure

a- The fatigue assessment procedure should follow the following equation:

$$\gamma F_{sra} \leq \phi F_{sm} \quad \text{.......... 11.1}$$

Where:

γ = load factor and is equal to 1.0
ϕ = resistance factor and is equal to 1.0
F_{sm} = nominal fatigue resistant stress range
F_{sra} = maximum applied unfactored fatigue stress range

b- The effect of applied stress cycles is characterized by the maximum applied unfactored stress range (F_{sra}). The maximum unfactored stress range can be computed from the applied fatigue loads using an elastic method of analysis. The fatigue loads should be positioned to give the maximum straining actions at the studied detail. In some structures such as cranes, consideration should be given to possible changes in usage.

c- In non-welded details or stress relieved welded details subjected to stress reversals, the effective stress range to be used in the fatigue assessment shall be determined by adding the tensile portion of the stress range and 60% of the compressive portion of the stress range. In welded details subjected to stress reversals, the stress range to be used in the fatigue assessment is the greatest algebraic difference between maximum stresses.

d- The number of constant stress cycles to be endured by the structure during its design life is given in Table 11.1 for crane structures. The number of cycles given in Table 11.1 is subject to modifications according to the competent authority requirements.

e- Each structural element has a particular detail category as shown in Table 11.2. The classification is divided into three parts which correspond to the following three basic groups:

Group 1: non-welded details, plain materials, and bolted plates.
Group 2: welded structural elements, with or without attachments.
Group 3: fasteners (bolts and welds).

f- The fatigue resistance of a structural part is characterized by the nominal fatigue resistant stress range (F_{sm}) which is obtained from Table 11.3 or Figure 11.1 for the specified number of constant cycles and the particular detail category.

g- When subjected to tensile fatigue loading, the nominal fatigue resistant stress range for High Strength Bolts friction type shall not exceed the values given in Table 11.4.

Table 11.1 Number of Loading Cycles – Crane Structures

ADA*	Field of Application	Number of Constant Stress Cycles
5	Occasional use	100,000
25	Regular use with intermittent operation	500,000
100	Regular use with continuous operation	2,000,000
> 100	Severe continuous operation	According to actual use

* ADA = Average daily application for 50 years design life

Table 11.2 Classification of Details
Group 1 : Non-Welded Details

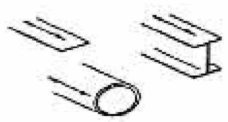
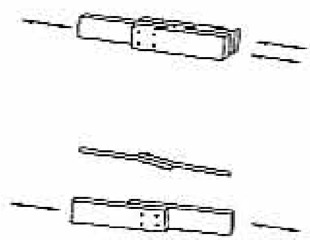
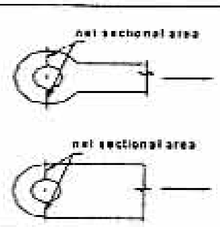
Description	Illustration	Class
1.1. Base metal with rolled or cleaned surfaces; flame cut edges with a surface roughness less than 25 μm		A
1.2. Base metal with sheared or flame cut edges with a surface roughness less than 50 μm		B
2.1. Base metal at gross section of high strength bolted slip resistant (friction) connections, except axially loaded joints which induce out of plane bending in connected material.		B B*
2.2. Base metal at net section of fully tensioned high strength bolted bearing type connections		D
2.3. Base metal at net section of other mechanically fastened joints (ordinary bolts & rivets).		
3. Base metal at net section of eye-bar head or pin plate.		E

Table 11.2 Classification of Details (Cont.)
Group 2 : Welded Structural Elements

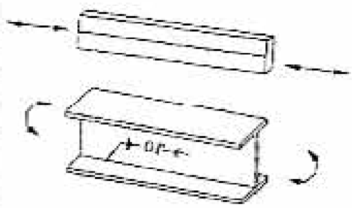
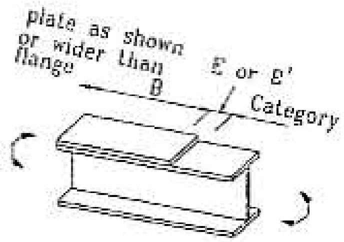
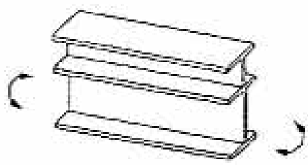
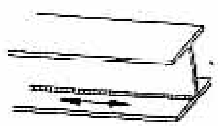
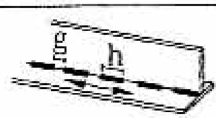
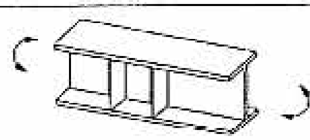
Description	Illustration	Class
4.1. Base metal in members without attachments, built up plates or shapes connected by continuous full penetration groove welds or by continuous fillet welds carried out from both sides without start stop positions parallel to the direction of applied stress.		B
4.2. Same as (4.1.) with welds having stop - start positions.		B'
4.3. Base metal in members without attachments, built-up plates or shapes connected by continuous full penetration groove welds with backing bars not removed, or by partial penetration groove welds parellel to the direction of applied stress.		B'
5. Base metal at continuous manual longitudinal fillet or full penetration groove welds carried out from one side only. A good fit between flange and web plates is essential and a weld preparation at the web edge such that the root face is adequate for the achievement of regular root penetration.		C
6. Base metal at zones of intermittent longitudinal welds with gap ratio g/h < 2.5		D
7. Base metal at zones containing copes in longitudinally welded T-joints.		D
8. Base metal at toe of welds on girder webs or flanges adjacent to welded transverse stiffeners.		C

Table 11.2 Classification of Details
Group 2 : Welded Structural Elements (Cont.)

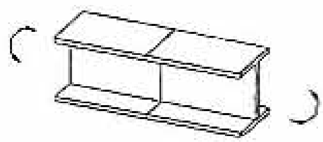
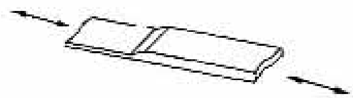
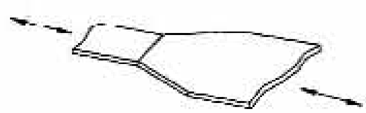
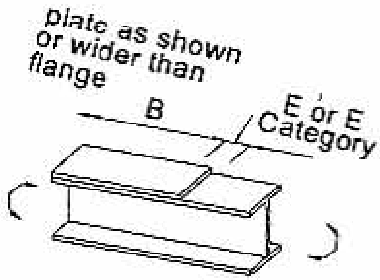
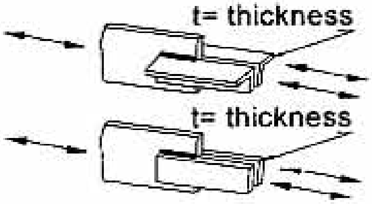
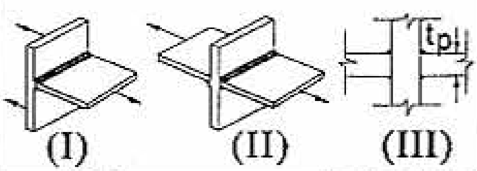
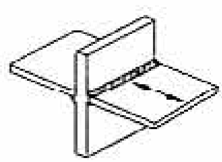
Description	Illustration	Class
9.1. Base metal and weld metal at full penetration groove welded splices (weld made from both sides) of parts of similar cross sections ground flush, with grinding in the direction of applied stress and weld soundness established by radiographic or ultrasonic inspection.		B
9.2. Same as (9.1.) but with reinforcement not removed and less than 0.10 of weld width.		C
9.3. Same as (9.2.) with reinforcement more than 0.10 of weld width.		D
10.1. Base metal and weld metal at full penetration groove welded splices (weld made from both sides) at transitions in width or thickness, with welds ground to provide slopes no steeper than 1 to 2.5 with grinding in the direction of applied stress, and with weld soundness established by radiographic or ultrasonic inspection.		B'
10.2. Same as (10.1.) but with reinforcement not removed and less than 0.10 of weld width.		C
10.3. Same as (10.2.) with slopes more than 1 to 2.5		D
10.4. Same as (10.1.) to (10.3.) but with welds made from one side only.		E'

Table 11.2 Classification of Details
Group 2 : Welded Structural Elements (Cont.)

Description	Illustration	Class
11.1. Base metal and weld metal at transverse full penetration groove welded splices on a backing bar. The end of the fillet weld of the backing strip is more than 10 mm from the edges of the stressed plate		D
11.2. Same as (11.1) with the fillet weld less than 10 mm from the edges of the stressed plate.		E
12.1. Base metal at ends of partial length welded cover plates narrower than the flange having square or tapered ends, with or without welds across the ends or wider than the flange with welds at the ends. Flange thickness ≤ 20 mm		E
Flange thickness > 20 mm		E'
12.2 Base metal at ends of partial length welded cover plates wider than the flange without end welds.		E'
13. Base metal at axially loaded members with fillet welded connections. t ≤ 25 mm		E
t > 25 mm		E'
14. Base metal at transverse end connections of tension loaded plate elements using a pair of fillet welds on opposite sides of the plate.		Equ. 11.2
15.1. Base metal and weld metal at full penetration weld in cruciform joints made of a weld soundness established by radiographic or ultrasonic inspection.		D
15.2. Same as (15.1) with partial penetration or fillet welds of normal quality.		E'

$F_{SRN} = R_{FIL} \times F_{SRN}$ Category C

$$R_{FIL} = 1.72\left(\frac{0.06 + 0.72\left(\frac{w}{t_p}\right)}{t_p^{0.167}}\right) \leq 1.0 \qquad \text{.............................11.2}$$

Table 11.2 Classification of Details
Group 2 : Welded Structural Elements (Cont.)

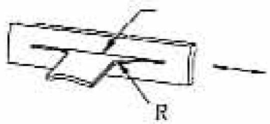
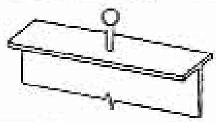
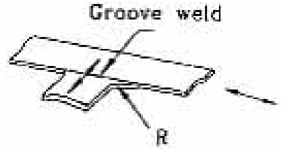
Description	Illustration	Class
16. Base metal at plug or slot welds.		E
17. Base metal and attachment at fillet welds or partial penetration groove welds with main material subjected to longitudinal loading and weld termination ground smooth R > 50 mm		D
R < 50 mm		E
18. Base metal at stud- type shear connector attached by fillet weld or automatic end weld.		C
19.1. Base metal at details attached by full penetration groove welds subject to longitudinal loading with weld termination ground smooth. Weld soundness established by radiographic or ultrasonic inspection R > 610 mm		B
610 mm > R > 150 mm		C
150 mm > R > 50 mm		D
R < 50 mm		E
19.2. Same as (19.1.) with transverse loading, equal thickness, and reinforcement removed. R > 610 mm		B
610 mm > R > 150 mm		C
150 mm > R > 50 mm		D
R < 50 mm		E

Table 11.2 Classification of Details
Group 2 : Welded Structural Elements (Cont.)

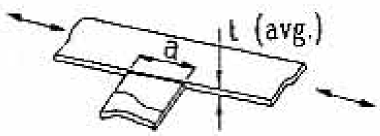
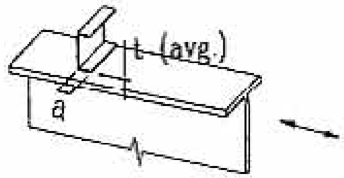
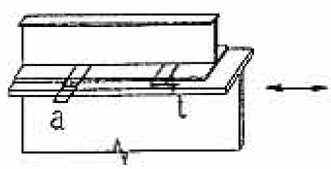
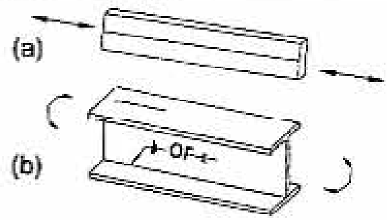
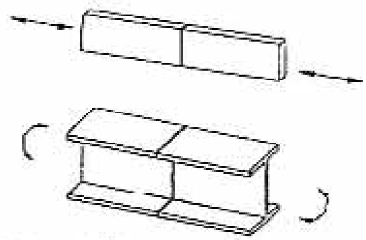
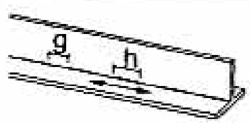
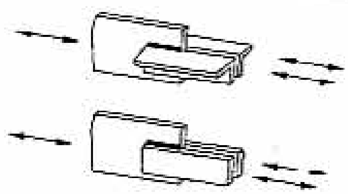
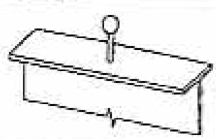
Description	Illustration	Class
19.3. Same as (19.2.) but reinforcement not removed R > 610 mm		C
610 mm > R > 50 mm		C
150 mm > R > 50 mm		D
R > 50 mm		E
19.4. Same as (19.2.) but with unequal thickness R > 50 mm		D
R < 50 mm		E
19.5. Same as (19.4.) but with reinforcement not removed and for all R		E
20. Base metal at detail attached by full penetration groove welds subject to longitudinal loading 50-mm< a <12t or 100 mm		D
a >12t or 100 mm (t<25 mm)		E
a >12t or 100 mm (t>25 mm)		E'
21. Base metal at detail attached by fillet welds or partial penetration groove welds subject to longitudinal loading a < 50 mm		C
50 mm< a <12t or 100 mm		D
a >12t or 100 mm (t<25 mm)		E
a >12t or 100 mm (t>25 mm)		E

Table 11.2 Classification of Details
Group 3 : Fasteners (Welds and Bolts)

Description	Illustration	Class
22.1. Weld metal of full penetration groove welds parallel to the direction of applied stress (weld from both sides)		B
22.2. Same as (22.1.) but with weld from one side only.		C
22.3. Weld metal of partial penetration transverse groove weld based on the effective throat area of the weld.		F
23.1 Weld metal of continuous manual or automatic longitudinal fillet welds transmitting a continuous shear flow.		D
23.2 Weld metal of intermittent longitudinal fillet welds transmitting a continuous shear flow.		E
23.3 Weld metal at fillet welded lab joints.		E'
24. Transversally loaded fillet welds.		E'
25. Shear on plug or slot welds.		F
26. Shear stress on nominal area of stud-type shear connectors.(Failure in the weld or heat affected zone.)		F
27.1. Hight strength bolts in single or double shear (fitted bolt of bearing type).		C
27.2. Rivets and ordinary bolts in shear.		D
28. Bolts and threaded rods in tension (on net area)		F

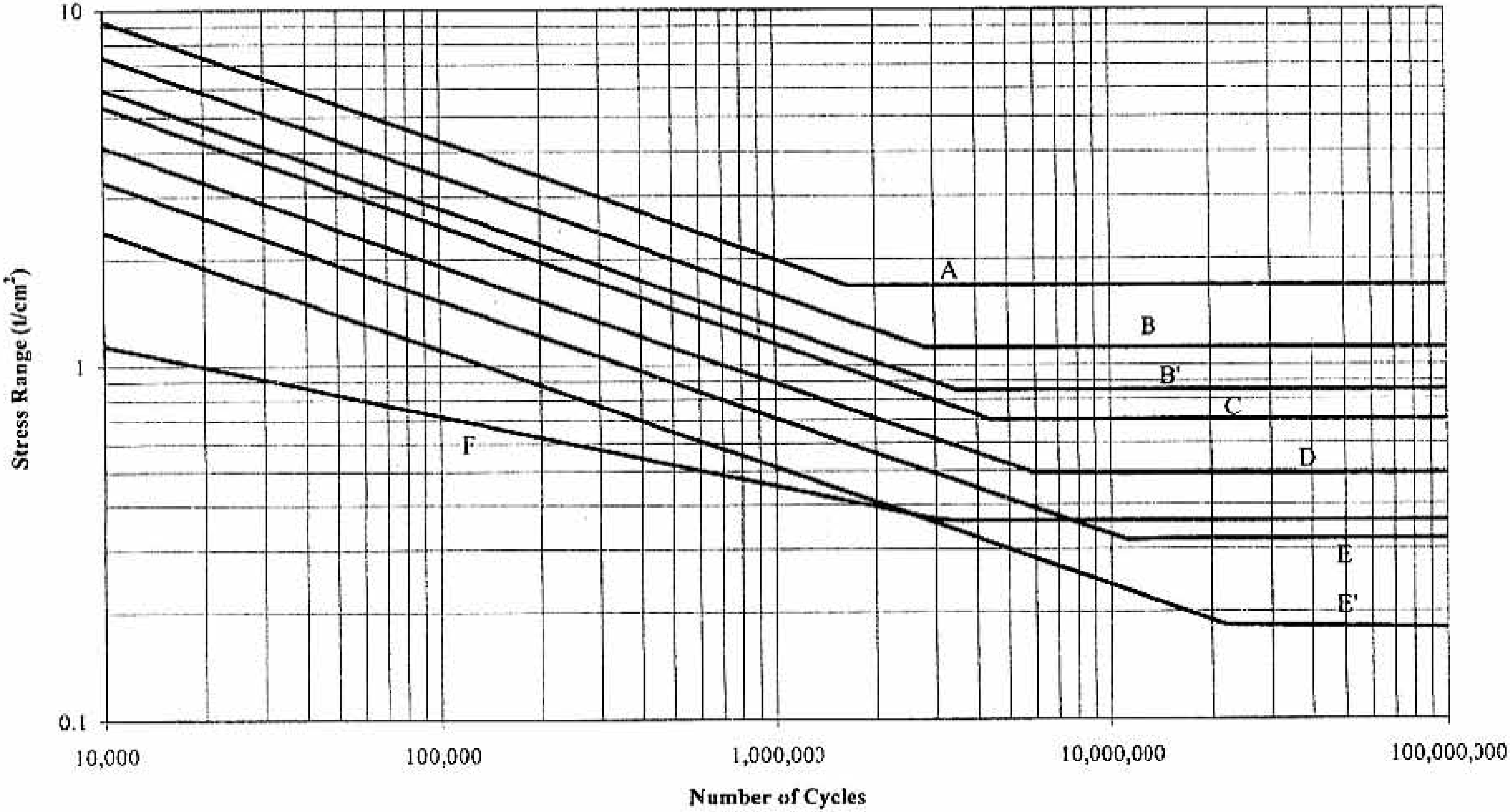


Figure 11.1 Stress Range Versus Number of Cycles

Table 11.3 Nominal Fatigue Resistant Stress Range F_{srn} for Number of Constant Stress Cycles (N)

Detail Class	F_{sm} (t/cm^2)			
	100,000	500,000	2,000,000	Over 2,000,000
A	4.30	2.52	1.68	1.68
B	3.42	2.00	1.26	1.12
B'	2.77	1.62	1.02	0.85
C	2.48	1.45	0.91	0.70
D	1.92	1.12	0.71	0.49
E	1.53	0.89	0.56	0.32
E'	1.11	0.65	0.41	0.18
F	0.72	0.52	0.40	0.36

Table 11.2 and Figure 11.1 are based on the following equation:

$$Log\ N = log\ a - m\ log\ F_{sm}$$

Where:

N = the number of constant stress cycles

F_{sm} = the nominal fatigue resistant stress range

m and log a = constants that depend on the detail category as follows:

Detail Category	m	Log a
A	3	6.901
B	3	6.601
B'	3	6.329
C	3	6.181
D	3	5.851
E	3	5.551
E'	3	5.131
F	5	4.286

Table 11.4 Nominal Fatigue Stress Range for High Strength Bolts

Number of Cycles	Nominal Fatigue Stress Range (t/cm^2)	
	Bolts Grade 8.8	Bolts Grade 10.9
$N \leq 20{,}000$	2.9	3.6
$20{,}000 < N \leq 500{,}000$	2.6	3.2
$500{,}000 < N$	2.0	2.5

CHAPTER 12

COMPOSITE STEEL – CONCRETE CONSTRUCTION

This chapter deals with composite construction used in buildings and consists of beams, columns, beam-columns and slabs. Section 12.1 applies to steel beams supporting concrete slabs that are interconnected such that they act together to resist bending. Provisions included apply to simple and continuous composite beams constructed with or without temporary shoring. Composite beams must be provided with shear connectors, or else completely encased in concrete.

Sections 12.2 and 12.3 apply respectively to columns and beam-columns composed of rolled or built-up steel shapes encased in concrete or steel tubing filled with concrete. Section 12.4 applies to composite floor slabs.

12.1 COMPOSITE BEAMS

12.1.1 Scope

This section applies to beams composed of either rolled or built-up steel sections, with or without concrete encasement, acting in conjunction with a reinforced concrete slab. The two elements are connected so as to form a composite section acting as one unit. Fig. 12.1 shows two typical cross sections of composite beams.

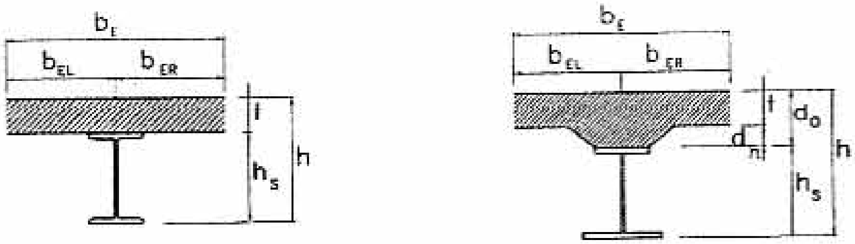


Figure 12.1 Typical Cross-Sections of Composite Beams

12.1.2 Components of Composite Beams

12.1.2.1 Steel Beam

All steel parts used in the composite beams shall comply with their relevant specifications. The steel beam may be a rolled section, a rolled section with a cover plate attached to the tension flange, a plate girder, or a lattice girder. A composite beam design may become more economic when the tension flange of the steel section is larger than the compression flange. The connection of the steel beam compression flange to the web must be designed for the shear flow calculated for the composite section.

During construction, the compression flange must satisfy local buckling and lateral torsional buckling requirements. After construction, however, the composite section is exempt from such requirements.

12.1.2.2 Concrete Slab

The concrete used for composite construction shall comply with the current Egyptian Code of Practice for the Design of Reinforced Concrete Structures. The minimum accepted value for the characteristic cube concrete strength, f_{cu}, is 250 kg/cm^2 for buildings. For deck slabs subjected directly to traffic (without wearing surface) such as garages, the value of f_{cu} shall not be less than 400 kg/cm^2.

The slab may rest directly on the steel beam or have a concrete haunch to increase the moment of inertia of the composite section. It is also possible to use a formed steel deck with the deck ribs oriented parallel or perpendicular to the steel beam. The concrete slab may also be pre-stressed with cast-in place connection to the steel beam.

12.1.2.3 Shear Connectors

Since the bond strength between the concrete slab and the steel beam is not dependable, mechanical shear connectors must be provided. They are fastened to the top flange of the steel beam and embedded in the concrete slab to transmit the longitudinal shear and prevent any slippage between the concrete slab and the steel beam. Furthermore, they prevent slab uplift.

There are several types of shear connectors such as studs, channels and angles, which are discussed in detail in Section 12.1.8.

12.1.3 Methods of Construction

Two different methods of construction are used:

Case I: Without Shoring

When no intermediate shoring is used under the steel beams or the concrete slab during casting and setting of the concrete slab, the steel section alone supports the dead and construction loads. After the slab has reached 75% of its required characteristic strength, f_{cu}, the composite section supports the live loads and the superimposed dead loads (e.g. flooring and walls).

Case II: With Shoring

When an effective intermediate shoring system is utilized during casting and setting of the concrete slab, the composite section supports both the dead and live loads. Shoring shall not be removed until the concrete has attained 75% of its required characteristic strength, f_{cu}.

12.1.4 Design of Composite Beams

12.1.4.1 Positive Bending

The positive design flexural strength $\phi_b M_n$ shall be determined as follows:

a- **For compact beam webs**

i.e. $(d_w / t_w) \le (127 / \sqrt{F_y})$

$\phi_b = 0.80$; M_n shall be determined from the plastic stress distribution on the composite section. M_n may be governed by the tensile yield strength of the steel section or the compressive stress of the concrete slab.

In this case all loads are considered resisted by the composite cross section (i.e. always Case-II). However, the steel beam alone shall be capable of supporting dead and construction loads.

b- **For non-compact webs of steel beams**

i.e. $(222 / \sqrt{F_y}) > (d_w / t_w) > (127 / \sqrt{F_y})$

$\phi_b = 0.85$; M_n shall be determined from the superposition of elastic stresses, considering the effect of shoring (i.e. Case-I or Case-II). The first yield is considered in this case as the flexural strength limit.

12.1.4.2 Negative Bending

The negative design flexural strength $\phi_b\ M_n$ shall be determined for the steel section alone.

Alternatively, the negative design flexural strength $\phi_b\ M_n$ shall be computed with: $\phi_b = 0.80$ and M_n determined from the plastic stress distribution on the composite section, provided that:

a- Steel beam is an adequately braced compact section.
b- Shear connectors connect the slab to the steel beam in the negative moment region.
c- Slab reinforcement parallel to the steel beam, within the effective width of the slab, is properly developed.

12.1.4.3 Methods of Analysis
12.1.4.3.1 Elastic Analysis

Elastic analysis may be used for beams and frames whether the web of the steel beam is compact or not. Similar to the practice in reinforced concrete design, the use of a constant stiffness for each element is adopted. The stiffness calculated using a weighted average of moments of inertia in the positive and negative moment regions may take the following form:

$$I = a\ I_{pos} + b\ I_{neg} \qquad \text{12.1}$$

Where:

I_{pos} = effective moment of inertia for positive moment, using the transformed section properties (I_v) as described in Section 12.1.4.7 below.

I_{neg} = effective moment of inertia for negative moment, typically using the steel section inertia (I_s) alone, unless the three conditions *listed above in Section 12.1.4.2 are satisfied.*

For continuous beams subject to gravity loads only: $a = 0.6$ and $b = 0.4$
For moment resisting frames: $a = b = 0.5$

12.1.4.3.2 Plastic Analysis

Plastic analysis is allowed for composite beams only when the steel section in the positive moment region has a compact web, and when the whole steel section in the negative moment region is compact.

No compactness limitations are needed for encased beams, but plastic analysis is permitted only if the direct contribution of concrete to the strength of sections is neglected; i.e. the concrete is relied upon only to prevent buckling. The steel web alone shall resist the vertical shear stresses of the composite beam, neglecting any contribution from the concrete slab.

12.1.4.4 Span to Depth Ratio

The ratio of the beam span, L, to the beam overall depth including concrete slab, h, lies generally between 16 and 22. For limiting the girder depth, L/h may exceed 22, provided that the deflection check as per Section 12.1.4.9 is satisfied.

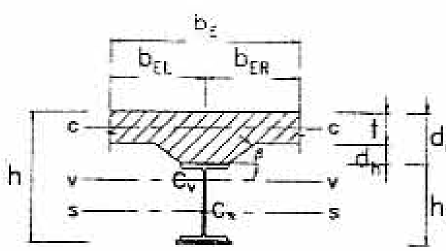


Figure 12.2 Section Dimensions and Notations

12.1.4.5 Thickness of Concrete Slab

The minimum concrete slab thickness is as follows:

- For roof slabs $t \geq 8.0$ cm
- For repeated floors $t \geq 10.0$ cm
- For floors supporting moving loads (e.g., garages) $t \geq 12.0$ cm

Slabs can be provided with haunches inclined with a slope not steeper than 1 horizontal : 3 vertical (i.e. tan $\beta \leq 3$, Fig. 12.2 and Fig. 12.3). The height of the haunch proper, d_h, is normally chosen not more than 1.50 (one and a half times) the slab thickness, t. In addition, the total depth, h, of the composite section is preferably chosen not greater than two and a half times the depth of the steel beam, h_s.

12.1.4.6 Effective Width of Concrete Slab

The effective width of the concrete slab b_E is the sum of the effective widths for each side of the beam center-line ($b_{EL}+b_{ER}$), Fig. 12.4, each of which shall not exceed:

- $L/8$ = One-eighth of the beam span
- B = One-half the distance to center-line of the adjacent beam
- b^* = the distance to the edge of the slab

Where: L is the beam span, center-to-center of supports, for both simple and continuous spans.

If the two adjacent spans in a continuous beam are unequal, the value of b_E, to be used for calculating bending stress and longitudinal shear in the regions of negative moments, shall be the mean of the values obtained for each span separately.

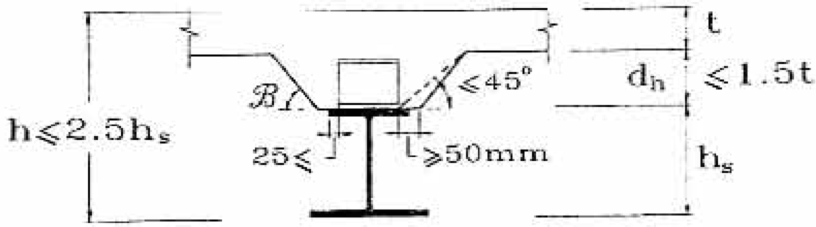


Figure 12.3 Dimensioning of Haunches

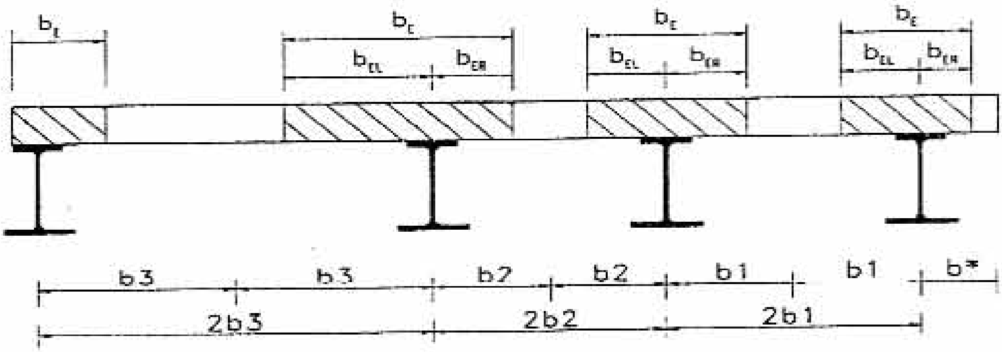


Figure 12.4 Effective Width of Concrete Slab

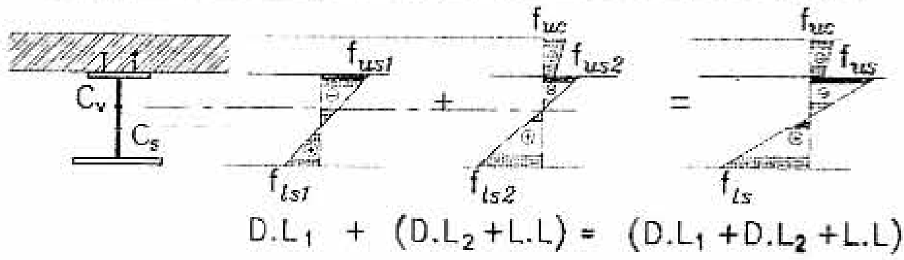
12.1.4.7 Calculation of Working Stresses and Deflections

According to the working stress design method (which is to be adopted for steel beams with non-compact webs as per Section 12.1.4.1.b and for the calculation of service load deflections), the composite beam shall be transformed to an equivalent virtual section using the modular ratio, n. The value of $n = E_s/E_c$ may be taken as the nearest whole number (but not less than 7). Table 12.1 lists the recommended values of n for some grades of concrete.

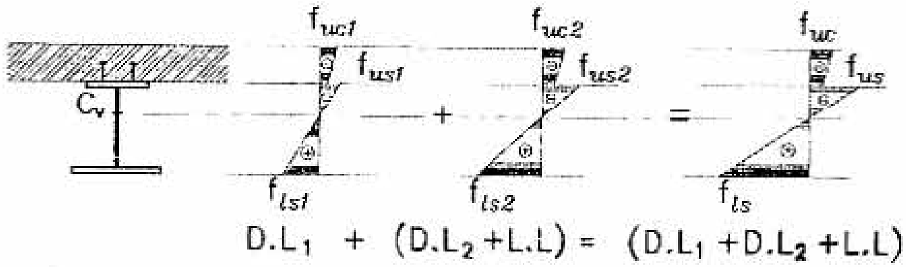
Bending stresses in the composite section (steel beam, concrete slab, and longitudinal reinforcement) shall be calculated in accordance with the elastic theory, ignoring concrete in tension and assuming no slippage between the steel beam and concrete slab. Fig. 12.5 illustrates the distribution of bending stresses for composite beams constructed with or without shoring.

Table 12.1 Recommended Values of the Modular Ratio (n)

Concrete Characteristic Cube Strength, f_{cu} (kg/cm^2)	Modulus of Elasticity of Concrete, E_c (t/cm^2)	Modular Ratio, n
250	220	10
300	240	9
400	280	8
$\geq$ 500	310	7



(a) Without Shoring (Case I)



(b) With Shoring (Case II)

D.L.$_1$ = Dead Loads + construction Loads,

D.L.$_2$ = Super Imposed Dead Loads (e.g. flooring and walls)

L.L. = Live Loads

Figure 12.5 Working Stress Distribution

Maximum bending stresses in the steel section shall comply with section 12.1.4.1.b, whereas, maximum bending stresses in the concrete slab shall not exceed the design strength limits given by the Egyptian Code of Practice for the Design of Reinforced Concrete Structures.

The steel web alone shall resist the vertical shear stresses of the composite beam neglecting any contribution from the concrete slab.

12.1.4.8 Continuous Beams

The composite construction of continuous beams makes it possible to further reduce the depth and deflection of the beams. Two methods may be adopted to design the section of the continuous beam at intermediate supports (i.e., zones of negative bending moments):

a- Steel section alone may be designed to support all loads, dead and live.

b- Steel reinforcement within the concrete slab effective width and extending parallel to the beam span, with an adequate anchorage length, in accordance with the provisions of the Egyptian Code of Practice for the Design of Reinforced Concrete Structures, may be used as a supplementary part of the steel section. In such a case, shear connectors must be extended above supports without exceeding the maximum pitch limits.

In the negative moment regions, the lower flange of the steel beam shall be checked against lateral and local buckling. The point of contra-flexure may generally be treated as a brace point.

12.1.4.9 Deflections

If the beam is shored during construction, Case-II, the composite section (using the virtual elastic moment of inertia) will support both dead-load- and live-load-deflections. However, if the construction is not shored, Case-I, the total deflection will be the sum of the dead load deflection of the steel beam and the live load deflection of the composite section. Similar to typical steel beams, deflection limits are the same.

12.1.5 Design for Creep and Shrinkage

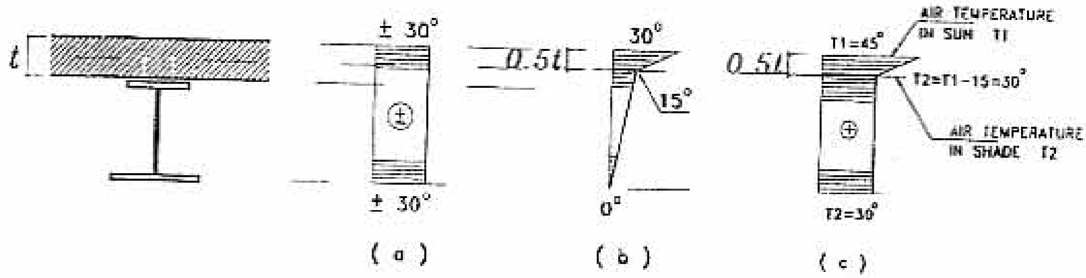
If construction is without shoring, i.e. Case-I, and live loads are not of the prolonged type, such as storage structures and garages, creep and shrinkage effects may be neglected.

If shoring is used to provide support during the hardening of concrete, Case-II, the dead load and live load deflections will be function of the composite section properties. Account must be taken for the fact that concrete is subject to creep under long-time loading (i.e., dead load) and that shrinkage will occur. This inelastic behavior may be approximated by multiplying the modular ratio, n, by a factor of two. The result is a reduced moment of inertia for the composite section, which is used in computing the dead load deflections and stresses.

When the live loads are expected to remain for extended periods of time, such as storage structures and garages, the conservative approach is to use the reduced composite moment of inertia for both dead and live loads (i.e., using $2n$ instead of n).

12.1.6 Design for Temperature Effect

The variation of temperature shall be assumed according to the Egyptian Code of Practice for Calculating Design Loads and Forces on Structures. In general, a 30°C uniform variation (increase or decrease) of the overall temperature of the structure is assumed. Due consideration shall be given for the fact that although the coefficient of thermal expansion for both steel and concrete is identical, the coefficient of thermal conductivity of concrete is only about 2% of that of steel. Therefore, the top of the concrete slab and other levels through the depth of the beam shall be assumed as shown in Fig. 12.6 especially for exposed slabs.



(a) Uniform Temperature Change

(b) Variable Temperature Change

(c) Variable Temperature Change

Figure 12.6 Temperature Distribution

12.1.7 Design of Encased Beams

A beam totally encased in concrete cast integrally with the slab, as shown in Fig. 12.7, may be assumed to be interconnected to the concrete by natural bond, without additional anchorage, provided that:

a- Concrete cover over the beam sides, top and bottom flange is at least 40 mm.
b- Top flange of the beam is at least 50 mm above the bottom of the slab.
c- Concrete encasement contains adequate mesh or other reinforcing steel throughout the whole depth to prevent spalling of the concrete.

Prior to hardening of the concrete, the steel section alone must be proportioned to support all dead and construction loads.

After hardening of concrete the completely encased steel beam is restrained from both local and lateral torsional buckling. Two alternatives can be used for the design in this case:

a- The composite section properties shall be used in calculating bending stresses, neglecting concrete in tension.
b- The steel beam alone is proportioned to resist the positive moment produced by all loads, live and dead, using the plastic moment design strength of the steel section, neglecting the composite action.

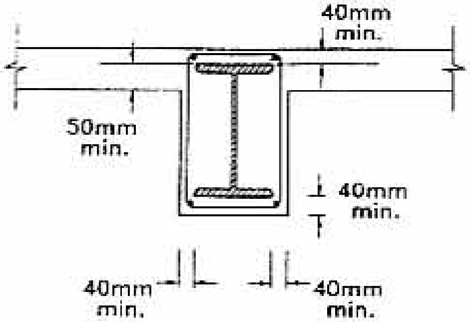


Figure 12.7 Encased Beam

12.1.8 Shear Connectors
12.1.8.1 Design Basis

- Shear connectors shall be provided throughout the beam length to transfer the longitudinal shear force at the interface between the concrete slab and steel beam.
- The number of shear connectors between critical sections shall not be less than the design longitudinal shear force, Q_L, divided by the design shear resistance of a connector, q_u.
- Shear connectors shall be capable of providing resistance against slab uplift.
- Shear connectors and transverse reinforcement in concrete slab shall be detailed to avoid longitudinal shear failure and splitting of the concrete slab.
- Placement and details of shear connectors shall be established to possess sufficient deformation capacity to justify plastic behavior when plastic analysis is used.
- A full composite action is obtained when the number of shear connectors between critical sections is sufficient to develop the plastic moment strength of composite beams and allows no slip when elastic stress distribution is used.
- A partial composite action is obtained when the shear strength of connectors governs the moment capacity of partially composite beams. Elastic computations such as those for deflections and fatigue shall include the effect of slip.
- Concrete-encased beams are assumed to be interconnected to concrete by natural bond without shear connectors provided that concrete cover and reinforcement are detailed as specified herein.

12.1.8.2 Longitudinal Shear Force
12.1.8.2.1 Moment Resistance Computed Using Plastic Stress Distribution
12.1.8.2.1a Full composite action

The total design longitudinal shear force, Q_{LF}, to be resisted by shear connectors spaced in accordance with Section 12.1.8.3 between the point of maximum positive moment and the point of zero moment shall be taken as the smaller of the following:

$$Q_{LF} = 0.67x10^{-3} A_c F_{cu} / \gamma_c + A_{sr} F_{yr} / \gamma_{sr} \quad \text{.......... 12.2}$$

Or $$Q_{LF} = A_s F_y / \gamma_s \quad \text{.......... 12.3}$$

Where:

F_{cu} = characteristic cube strength of concrete after 28 days in kg/cm^2
γ_c = partial factor of safety for concrete, 1.5
A_c = area of concrete slab within effective width in cm^2
A_{sr} = area of longitudinal steel reinforcement in slab effective width
F_{yr} = yield stress of steel reinforcing bars in t/cm^2
γ_{sr} = partial factor of safety for steel bars, 1.15
A_s = area of steel cross section in cm^2
F_y = yield stress of steel beam in t/cm^2
γ_s = partial factor of safety for structural steel, 1.10

In continuous beams where adequately developed longitudinal reinforcing steel in negative moment regions is considered to act compositely with steel beam, the total design longitudinal shear force between the point of maximum negative moment and the point of zero moment shall be taken as follows:

$$Q_{LF} = A_{sr} F_{yr}/\gamma_{sr} \qquad \text{......... 12.4}$$

12.1.8.2.1b Partial Composite Action

The total design longitudinal shear force, Q_L, to be resisted by shear connectors between the point of maximum positive moment, M_{app}, and the point of zero moment can be taken as follows:

$$Q_L = (M_{app}-M_{st})\, Q_{LF} / (M_p-M_{st}) \qquad \text{......... 12.5}$$

Where:

M_{st} = design plastic resistance to positive moment of steel section alone.
M_p = design plastic resistance of composite section to positive moment with full shear connection.

In negative moment regions, the design longitudinal shear force supported by shear connectors between the point of maximum negative moment and the point of zero moment will be computed by Eq. 12.5.

12.1.8.2.2 Moment Resistance Computed Using Elastic Stress Distribution

For the case of non-compact web and the neutral axis lying in the web, the design longitudinal shear per unit length shall be calculated by the elastic theory using the elastic properties of the virtual section.

In positive moment regions, shear connectors shall be designed to transfer the average horizontal shear force between the point of zero moment and the point of maximum positive bending moment. In negative moment regions, shear connectors shall be designed to transfer the average horizontal shear force between the point of zero moment and the point of maximum negative bending moment if top slab reinforcement or concrete slab is included in the composite section, or else shear connectors may be located at maximum spacing as per Section 12.1.8.3.

12.1.8.3 Placement and Spacing

Shear connectors required each side of the point of maximum bending moment, positive or negative, may be distributed uniformly between that point and the adjacent points of zero moment. However, the number of shear connectors between concentrated loads and the nearest point of zero moment shall be sufficient to develop the required horizontal shear between the concrete slab and the steel beam.

The spacing e (section 12.1.8.2.2) is inversely proportional to Q_u, and the connectors are to be arranged closer to each other at the supports and at bigger intervals near the middle of the beam.

Except for stud connectors, the minimum center-to-center spacing of shear connectors shall not be less than the total depth of the slab including haunch, d_o. The maximum center-to-center spacing of connectors shall not exceed the least of the following:

- 60 cm
- Three times the total slab thickness, $(3d_o)$
- Four times the connector height including hoops or anchors.

However, the maximum spacing of connectors may be exceeded over supports of continuous beams to avoid placing connectors at locations of high tensile stresses in the steel beam upper flange.

12.1.8.4 Design Shear Resistance of Shear Connectors

This section applies to the calculation of the design horizontal shear load, q_u, for one connector. The value of q_u computed from the following formulas shall not exceed the design horizontal load, q_{con}, provided by the connector connection to the beam flange. Should other types of shear connectors be used, an appropriate test program shall be conducted to determine the allowable load per shear connector.

12.1.8.4.1 Anchors and Hoops

The design horizontal load for each leg of anchors and hoops (Fig. 12.8) satisfying the requirements of Section 12.1.8.5.1 shall be computed as follows:

$$q_u = A_s F_{ys} \cos\beta \, (1 + \sin^2\alpha)^{1/2} / \gamma_{sr} \le q_{con} \qquad \text{......} \; 12.6$$

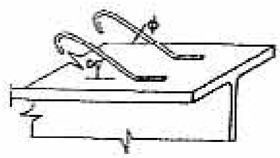
Where:

A_s = cross sectional area of anchor or hoop
F_{ys} = yield stress of anchor or hoop material

γ_{sr} = partial safety factor for steel reinforcing bars, 1.15

β = angle in horizontal plane between anchor and longitudinal axis of the beam, see Fig. 12.9

α = angle in the vertical plane between anchor or hoop and the beam upper flange



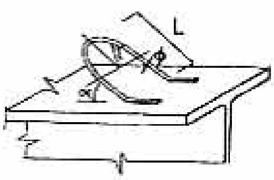


Figure 12.8 Anchors and Hoops Shear Connectors

12.1.8.4.2 Block Connectors

Block connectors such as bar, T-section, Channel section and Horseshoe meeting the requirements of section 12.1.8.5.2 can be used as shear connectors (Fig. 12.9). The front face shall not be wedge shaped and so stiff that uniform pressure distribution on concrete can be reasonably assumed at failure. The design horizontal load transmitted by bearing can be computed from the following equation:

$$q_{u,block} = 0.7x10^{-3}\,\eta A_1\,F_{cu}\,/\,\gamma_c \qquad 12.7$$

Where:

η = $(A_2/A_1)^{1/2} \leq 2.0$

A_1 = area of connector front face

A_2 = bearing area on concrete, shall be taken as the front area of the connector, A_1, enlarged at a slope of 1:5 to the rear face of the adjacent connector. Only parts of A_2 falling in the concrete section shall be taken into account

γ_c = partial safety factor for concrete, 1.5

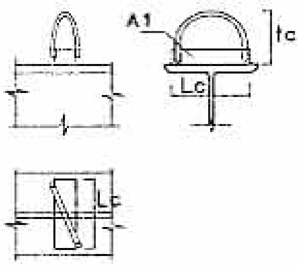
Block connectors shall be provided with anchors or hoops sharing part of the horizontal load supported by the connector, provided that due account shall be taken of the differences of stiffness of the block connector and the anchors or hoops. The design horizontal load per connector can be computed from the following:

$$q_u = q_{u,block} + 0.5\,q_{u,anchor} \leq q_{con} \qquad 12.8$$

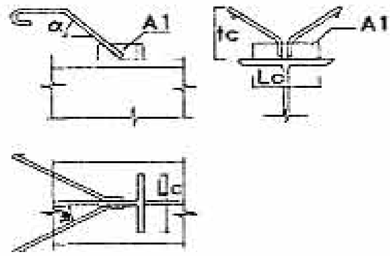
and $$q_u = q_{u,block} + 0.7\,q_{u,hoop} \leq q_{con} \qquad 12.9$$

Where:

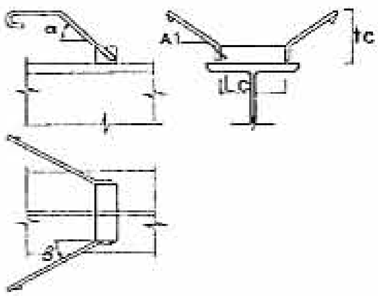
$q_{u,anchor}$ = horizontal load supported by anchor (Section 12.1.8.4.1)
$q_{u,hoop}$ = horizontal load supported by hoop (Section 12.1.8.4.1)



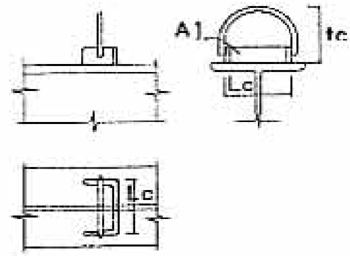
Block Connector with hoop



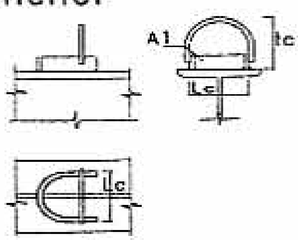
T–section with anchor



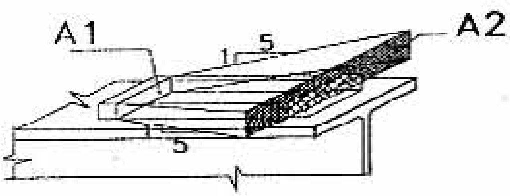
Block Connector with anchor



Channel section with hoop



Horseshoe connector with hoop



Definition of area (A2)

Figure 12.9 Block Shear Connectors

12.1.8.4.3 Stud Shear Connectors

The design horizontal load, q_u, for one stud connector (Fig.12.10a) conforming to the requirements stated in section 12.1.8.5.3 shall be computed from the following formula:

$$q_u = 8.1x10^{-3}\, A_{sc}\,(F_{cu}\, E_c)^{½} \leq q_{con} \quad \text{......} \quad 12.10$$

Where: A_{sc} = cross sectional area of stud connector, cm^2

12.1.8.4.4 Channel Shear Connectors

The design horizontal load, q_u, for one channel shear connector (Fig. 12.10b) conforming to the requirements stated in Section 12.1.8.5 shall be computed from the following equation:

$$q_u = 5.7x10^{-3}\,(t_f + 0.5t_w)\,L_c\,(F_{cu}\,E_c)^{1/2} \leq q_{con} \quad \text{......} \quad 12.11$$

Where:

t_f = flange thickness of channel shear connector, cm
t_w = web thickness of channel shear connector, cm
L_c = length of channel shear connector, cm

12.1.8.4.5 Angle Connector

The design horizontal load for an angle connector (Fig. 12.10c) welded to the beam top flange and satisfying the requirements of Section 12.1.8.5.4 shall be computed as follows:

$$q_u = 8.4x10^{-3}\,L_c\,t_c^{3/4}\,F_{cu}^{\;2/3} \leq q_{con} \quad \text{......} \quad 12.12$$

Where:

L_c = length of the angle connector, cm
t_c = width of the outstanding leg of the angle connector, cm

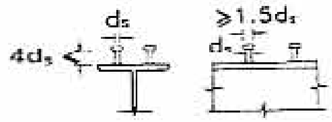
It is recommended to provide a reinforcement bar to prevent uplift of the concrete slab, the minimum diameter of the bar shall be computed from the following:

$$\phi \leq 0.38\,(q_u / F_{ys})^{1/2} \quad \text{......} \quad 12.13$$

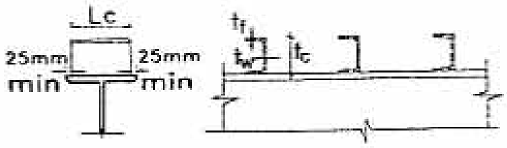
Where:

ϕ = diameter of the reinforcing bar, cm
F_{ys} = yield stress of the reinforcing bar, t/cm^2

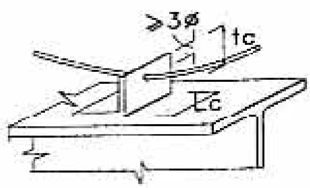
The length of the reinforcing bar on each side of the angle connector standing leg shall be computed based on the allowable bond strength of concrete following the provisions of the Egyptian Code of Practice for Reinforced Concrete Design.



(a) Stud Connectors



(b) Channel Connectors



(c) Angle Connectors

Figure 12.10 Shear Connectors

12.1.8.5 Shear Connectors Requirements

12.1.8.5.1 Anchors and Hoops

a- Anchors and hoops (Fig. 12.8) designed for longitudinal shear should point in the direction of diagonal tension. Where thrust can occur in both directions, connectors pointing in both directions should be provided.

b- Hoop connectors (Fig. 12.8) shall satisfy the following:

$r \geq 7.5\phi$, $L \geq 4r$ & *concrete cover* $\geq 3\phi$

c- Development length and concrete cover of anchors shall be based on the design concrete bond stresses as per the Egyptian Code of Practice for Reinforced Concrete Design.

12.1.8.5.2 Block Connectors

a- Block connectors (Fig. 12.9) shall be provided with anchoring devices to prevent uplift of concrete slab.

b- The height of bar connectors shall not exceed four times its thickness.

c- T sections shall be hot rolled section or a part of it with a flange width not exceeding ten times the flange thickness. The height of the T-section shall not exceed ten times the flange thickness or 150 mm which ever is the least.

d- Channel sections shall be hot rolled with a web width not exceeding 25 times the web thickness. The height of the channel section shall not exceed 15 times the web thickness nor 150 mm which ever is the least.

e- The height of the horseshoe shall not exceed 20 times the web thickness nor 150 mm which ever is the least.

12.1.8.5.3 Stud connectors

The length of the stud connector (Fig. 12.10a) shall not be less than four times its diameter, d_s, after installation. The value of d_s shall not be less than 16 mm and not more than 22 mm. The nominal diameter of the stud head shall not be less than one and half times the stud diameter, d_s. The value of d_s shall not exceed twice the thickness of the steel beam top flange.

Except for formed steel decks, the minimum center-to-center spacing of studs shall be six times the diameter ($6d_s$) measured along the longitudinal axis of the beam; and four times the diameter ($4d_s$) transverse to the longitudinal axis of the supporting composite beam (Fig. 12.11). If stud connectors are placed in a staggered configuration, the minimum transversal spacing of stud central lines shall be $3d_s$. Within ribs of formed steel decks, the minimum permissible spacing is $4d_s$ in any direction.

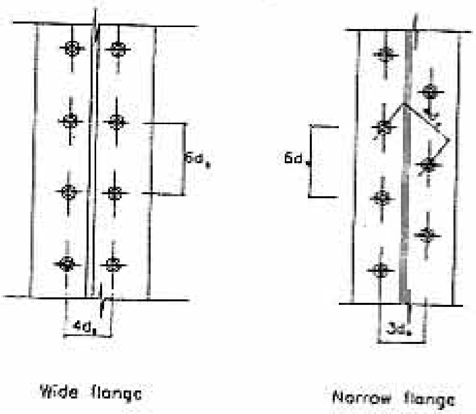


Figure 12.11 Minimum Spacing of Stud Connectors

12.1.8.5.4 Angle Connectors

The height of the outstanding leg of an angle connector shall not exceed ten times the angle thickness or 150 mm which ever is the least. The length of the angle connector shall not exceed 300 mm.

12.1.8.5.5 Connection to Steel Flange

The connection between the shear connector and the beam flange shall be designed to resist the horizontal shear load, q_u, transferred by the connector (Section 12.1.8.4).

12.1.8.5.6 Uplift of Concrete Slab

a- Shear connectors shall be capable of providing resistance to uplift of concrete slab by designing it to support a tensile force perpendicular to the steel flange of at least 10% of the design horizontal load, q_u, carried by the connector (Section 12.1.8.4). If necessary, shear connectors shall be provided with anchoring devices.

b- The surface of the connector that resists separation forces (i.e. the inside of a hoop or the underside of a head of a stud) shall extend not less than 40 mm clear above the slab bottom reinforcement.

12.1.8.5.7 Concrete Cover

a- In order to ensure adequate embedment of shear connectors in concrete slab, the connector shall have at least 50 mm of lateral concrete cover (Fig. 12.12). On the other hand, the minimum concrete cover on top of the connector shall not be less than 15 mm.

b- Except for formed steel slab; the sides of the haunch should lie outside a line drawn at maximum of 45° from the outside edge of the connector. The lateral concrete cover from the side of the haunch to the connector should be not less than 50 mm.

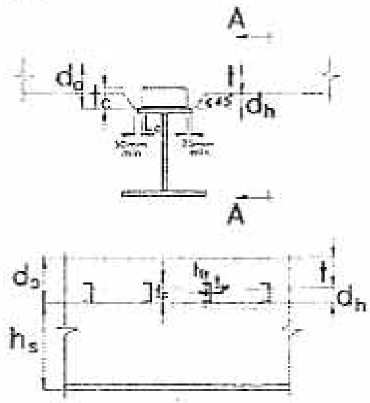


Figure 12.12 Concrete Cover Requirements

12.1.8.5.8 Transverse Reinforcement in Concrete Slab

Transverse reinforcement in the slab at the location of shear connectors shall be designed as per the Egyptian Code of Practice for Reinforced Concrete Structures to avoid longitudinal shear failure or splitting of the slab at the edge of the steel beam upper flange.

12.1.8.5.9 Dimensions of Steel Flange

The thickness of steel flange to which the connector is fastened shall be sufficient to allow proper welding and proper transfer of load from the connector to the web plate without local failure or excessive deformations. The distance between the edge of a connector and the edge of the beam flange to which it is welded should not be less than 25 mm.

12.1.8.6 Elastic Properties of Partially Composite Beams

Elastic calculations for stress and deflection of partially composite beams should include the effect of slip. The effective moment of inertia I_{eff} for a partially composite beam can be approximated as follows:

$$I_{eff} = I_s + (I_F - I_s)(\Sigma q_u / Q_{LF})^{1/2} \qquad 12.14$$

Where:

I_s = moment of inertia for the steel section only
I_F = moment of inertia for the fully un-cracked transformed section
Σq_u = design shear strength of shear connectors between the point of maximum positive moment and the point of zero moment to either side
Q_{LF} = design longitudinal shear force for full shear connection in accordance with Section 12.1.8.2.1a

The effective section modulus, S_{eff}, referred to the tension flange of the steel section for a partially composite beam can be approximated by:

$$S_{eff} = S_s + (S_F - S_s)(\Sigma q_u / Q_{LF})^{1/2} \qquad 12.15$$

Where:

S_s = section modulus for the steel section only referred to tension flange
S_F = section modulus for fully un-cracked composite section referred to tension flange or steel section

12.2 COMPOSITE COLUMNS
12.2.1 Scope

This section applies to the design of steel columns fabricated from rolled or built-up steel sections and encased in concrete or concrete-filled hollow steel pipes or tubing. Typical types of composite columns are illustrated in Fig. 12.13.

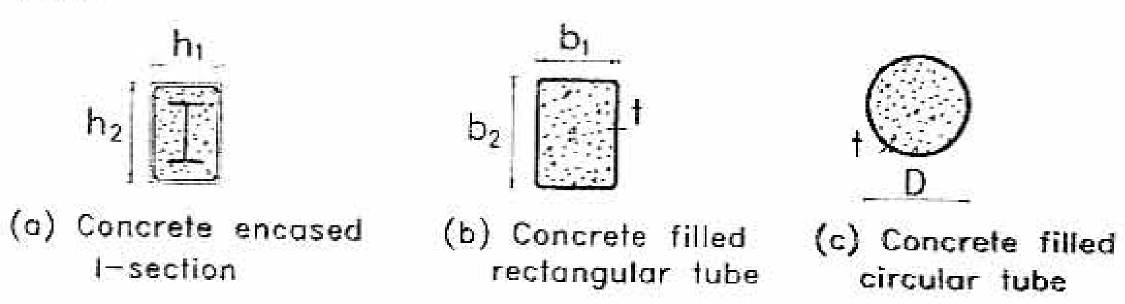


Figure 12.13 Sections for Composite Columns

12.2.2 Requirements

In order to qualify as a composite column, the following requirements shall be fulfilled:

a- The total cross sectional area of the structural steel section shall not be less than four percent (4%) of the gross column area. If this condition is not satisfied, the member will be classified as a reinforced concrete column and its design will be handled by the Egyptian Code of Practice for Reinforced Concrete Structures.

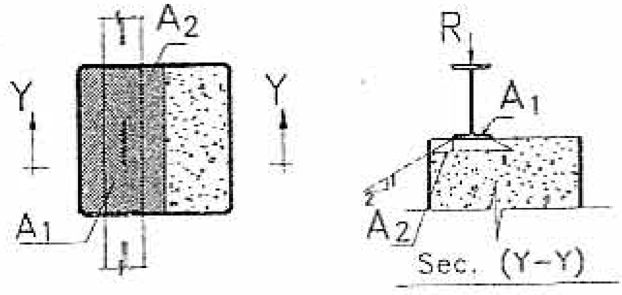
b- The characteristic cube strength of concrete, F_{cu}, shall not be less than 250 kg/cm^2, nor greater than 500 kg/cm^2.

c- Multiple steel shapes in the same cross section shall be interconnected with lacing, tie plates, or batten plates to prevent buckling of individual shapes before hardening of concrete.

d- Concrete encasement shall be reinforced with longitudinal load carrying bars and lateral ties (stirrups) to restrain concrete and prevent cover spalling. The spacing of lateral ties shall not exceed two thirds of the least dimension of the composite section, or 30 cm which ever is smaller. The cross sectional area of lateral ties and longitudinal bars shall be at least 0.02 cm^2 per cm of bar spacing. Concrete cover over lateral ties or longitudinal bars shall not be less than 4 cm.

e- To avoid local buckling, the minimum wall thickness of steel rectangular tubing filled with concrete is equal to $b(F_y/3E_s)^{1/2}$ for each face of width b of the tube section. The minimum wall thickness for circular sections of outside diameter, D, is $D(F_y/8E_s)^{1/2}$.

f- To avoid overstressing of concrete at connections, the portion of the load carried by concrete shall not exceed the allowable bearing stress that will be computed as per the Egyptian Code of Practice for Reinforced Concrete Structures, see Fig. 12.14.



$$F_{app.} = R/A_1 < 0.3 f_{cu} \sqrt{\frac{A_2}{A_1}}$$

$$\text{where } \sqrt{\frac{A_2}{A_1}} \leq 2.0$$

Figure 12.14 Bearing on Composite Columns

12.2.3 Design Strength

The design strength, $\phi_c\ P_n$, for symmetric axially loaded composite columns shall be computed on the steel section area utilizing a modified radius of gyration, yield stress and young's modulus, r_m, F_{ym} and E_m respectively, to account for composite behavior.

$$P_u = \phi_c P_n \quad \text{......} \quad 12.16$$

$$= \phi_c A_s F_{cr} \quad \text{......} \quad 12.17$$

For inelastic buckling, $\lambda_m \leq 1.1$ $\quad F_{cr}=(1-0.348\,\lambda_m^{2})\,F_{ym}$ 12.18

For elastic buckling, $\quad \lambda_m \geq 1.1 \quad F_c = 0.648\,F_{ym}/\lambda_m^2$ 12.19

Where:

$$F_{ym} = F_y + c_1 F_{yr} (A_r/A_s) + c_2 F_{cu} (A_c/A_s) \quad \text{......} \quad 12.20$$

$$E_m = E_s + c_3 E_c (A_c/A_s) \quad \text{......} \quad 12.21$$

$$\lambda_m = \text{Slenderness ratio} = L_b(F_{ym}/E_m)^{1/2} / \pi r_m \quad \text{......} \quad 12.22$$

Where:

L_b	=	buckling length, bigger of in-plane and out-of-plane buckling lengths
r_m	=	radius of gyration of the steel shape, pipe or tubing except that for steel shapes encased in concrete it shall not be less than 0.3 times the overall width of the composite column in the plane of bending
F_{ym}	=	modified yield stress, t/cm^2, $\geq F_y$
F_y	=	yield stress of steel, t/cm^2
ϕ_c	=	strength reduction factor for compression members, 0.80
F_{yr}	=	yield stress of longitudinal steel reinforcement, t/cm^2
E_m	=	modified Young's modulus, t/cm^2, $\geq E_s$
E_s	=	Young's Modulus of Steel, 2100 t/cm^2
E_c	=	Young's Modulus of concrete, t/cm^2
A_s	=	area of steel section, pipe or tubing, cm^2
A_r	=	area of longitudinal steel reinforcement, cm^2
A_c	=	area of concrete, cm^2, excluding A_s and A_r
$c_1, c_2,$ and c_3	=	numerical coefficients taken as follows: - For concrete encased sections, $c_1 = 0.7$, $c_2 = 0.48$, and $c_3 = 0.20$ - For concrete filled pipes or tubing, $c_1 = 1.0$, $c_2 = 0.68$, and $c_3 = 0.40$

12.3 COMPOSITE BEAM-COLUMNS

The interaction of axial compression and flexure for doubly symmetric composite members shall be limited by the following:

For $P_u/(\phi_c P_n) \geq 0.20$

$$P_u/(\phi_c P_n) + (8/9)\{ M_{ux}/(\phi_b M_{nx}) + M_{uy}/(\phi_b M_{ny}) \} \leq 1.0 \qquad 12.23$$

For $P_u/(\phi_c P_n) < 0.20$

$$P_u/(2\phi_c P_n) + \{ M_{ux}/(\phi_b M_{nx}) + M_{uy}/(\phi_b M_{ny}) \} \leq 1.0 \qquad 12.24$$

Where:

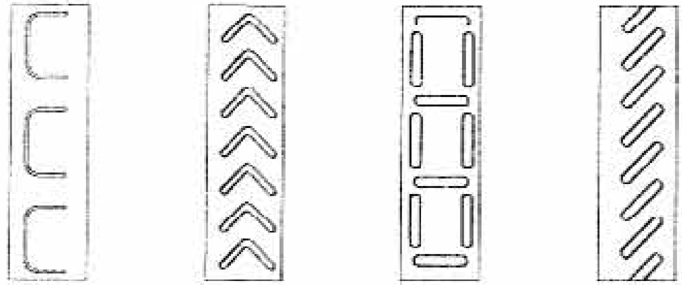
P_u = required compressive strength
P_n = nominal compressive strength determined in accordance with Sec. 12.2.3
M_u = required flexural strength considering the second order effect as per Sec.2.2.2
M_n = nominal flexural strength determined from plastic stress distribution on the composite cross section except that for $P_u/\phi_c P_n \leq 0.3$ the flexural strength shall be taken equal to the flexural strength at $P_u = 0.0$
F_{ym} = modified yield stress as per Eq. 12.20
ϕ_c, ϕ_b = resistance factor for axial compression and flexure respectively.
λ_c = column slenderness parameter defined in Eq. 12.22

For members subjected to axial tension and flexure, the interaction equations shall be utilized except that composite action of concrete will be neglected.

12.4 COMPOSITE SLABS

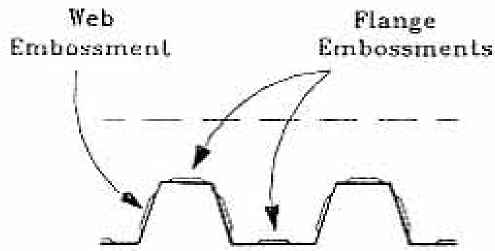
12.4.1 Scope

Composite steel floor deck is cold-formed steel deck which acts as a permanent form and as the positive bending reinforcement for the structural concrete. When suitably fastened, the steel deck also acts as a working platform. After the concrete hardens the steel deck and the concrete are interlocked by mechanical means as shown in Fig. 12.15 (e.g. flange and web embossments), the shape of the deck (e.g. re-entrant-trough profiles), surface bond (e.g. open-trough profiles), or by a combination of these means. Depending on surface bond alone (open-trough profiles without embossments) to develop the necessary interlocking is not enough; and is therefore not allowed.



Typical Embossment Shapes

Re-entrant Trough Profiles



Open Trough Profile With Embossments

Figure 12.15 Steel Deck with Embossments

12.4.2 Materials

12.4.2.1 Composite Steel Deck

Composite steel floor deck shall conform to the specifications for design of cold-formed structural steel members. The steel used shall have a minimum yield point of 2.30 t/cm^2. The delivered thickness of the uncoated steel shall not be less than 95% of the design nominal thickness.

12.4.2.1a Finish

Since the composite deck is the positive bending reinforcement for the slab it must be designed to last the life of the structure; a minimum finish is galvanized coating of 260 g/m^2 for both sides (average of 18 microns of Zinc coating per side).

12.4.2.1b Concrete

Concrete shall be in accordance with the applicable sections of the Egyptian Code Requirements for Reinforced Concrete. Minimum characteristic compressive strength (f_{cu}) shall be 250 kg/cm^2. Admixtures containing chloride salts shall not be used to avoid corrosion of the steel deck.

12.4.3 Design (Deck as a Form)

12.4.3.1 The section properties of the steel floor deck (as a form in bending) shall be computed in accordance with the specifications for design of cold-formed structural steel members. In calculating the section properties of the deck, Code provisions may require that the compression flange be reduced to an effective width, but when used as tensile reinforcement for the composite slab, the total area of the cross section is used.

12.4.3.2 Stress in the deck shall be calculated under the combined weights of wet concrete, metal deck, and the following construction live loads: 100 kg/m^2 uniformly distributed load or 80 kg concentrated load distributed on a 35-cm-section of deck, whichever gives greater stresses.

12.4.3.3 Calculated theoretical deflections of the deck, as a form, shall be based on the weight of the concrete (as determined by the design slab thickness) and the weight of the steel deck, uniformly loaded on all spans and shall be limited to $L/180$ or 18 mm, whichever is smaller. The deflection calculations do not have to take into account construction loads as these are considered to be temporary loads; the deck is designed to always be in the elastic range, so removal of temporary loads should allow the deck to recover.

12.4.3.4 Bearing Lengths

The deck must be adequately attached to the structure to prevent slip off. A minimum of 40 mm of end bearing length must be provided.

12.4.4 Installation

12.4.4.1 Welding

Floor deck units shall be anchored to supporting members, including bearing walls, with nominal 16 mm diameter puddle welds or equivalent at all edge ribs plus a sufficient number of interior ribs to provide a maximum average spacing of 300 mm. The maximum spacing between adjacent points of attachments shall not exceed 450 mm.

Welding washers shall be used when welding steel floor deck of less than 0.7 mm thickness. If studs are being applied through the deck onto the structural steel the stud welds can be used to replace the puddle welds. The deck should be welded to act as a working platform and to prevent blow off.

12.4.4.2 Mechanical Fasteners

Mechanical fasteners (powder-actuated, screws, pneumatically driven fasteners, etc.) are recognized as viable anchoring methods, provided the type and spacing of the fasteners satisfies the design criteria. Documentation in the form of test data, design calculations, or design charts should be submitted by the fastener manufacturer as the basis for obtaining approval.

12.4.4.3 Lapped and Butted Ends

Deck ends may be either butted or lapped over supports. Standard tolerance for ordered length is plus or minus 15 mm. If stud shear connectors are used, deck units should be butted and not lapped. Gaps are acceptable at butted ends.

12.4.4.4 Differential Deflection

Shall be controlled by fastening together side laps of floor deck units as recommended by the steel deck manufacturer.

12.4.5 Design Steel Deck and Concrete as a Composite Unit

12.4.5.1 General

The composite slab shall be designed as a reinforced concrete slab with the steel deck acting as the positive reinforcement. Slabs shall be designed as simple or continuous spans under uniform loads. Nevertheless, most published live load tables are based on simple span analysis of the composite systems; that is, the slab is assumed to crack over each support. If the designer requires a continuous slab, then the negative reinforcing should be designed using conventional reinforced concrete design techniques. The welded wire mesh, chosen for temperature reinforcing, does no usually supply enough area for continuity. The deck shall not be considered as compression reinforcing for the slab in negative moment regions.

12.4.5.2 Testing

The deck manufacturer shall have performed or witnessed by a specialized engineer, a sufficient number of tests on the composite deck/slab system. to have determined load/deflection characteristics and the mode of failure under uniform or symmetrically placed point loads. Based on the test information, the design load rationale shall be established by the ultimate strength analysis method based on:

a- Flexural strength or

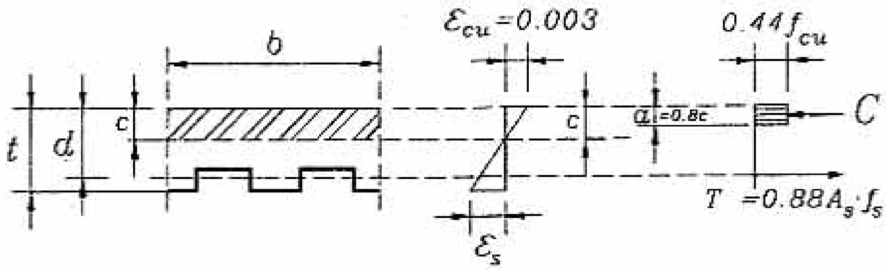
b- Shear-bond strength

12.4.5.2a Flexural Strength Design

As in the case of ordinary reinforced concrete construction, loads are found by applying overload factors to service dead and live loads to calculate the total factored moment M_u. The design moment strength is obtained by multiplying the nominal strength of the composite section by the resistance reduction factor $\phi = 0.85$. The total factored moment M_u must be less than the design moment strength ϕM_n, i.e.,

$$
\begin{aligned}
M_u &\leq \phi M_n \\
&\leq 0.85\, M_n
\end{aligned}
$$

The area of the steel deck, located at the centroid of the deck profile, is considered to be the reinforcement of the concrete slab as shown in Fig.12.16.



$$M_n = T \cdot Y_{cr} \quad \& \quad \phi = 0.85$$

$$\phi\, M_n = 0.85\ (0.88 A_s \cdot f_s) \cdot (d - \tfrac{a}{2}) = 0.75\ A_s \cdot f_s \cdot (d - \tfrac{a}{2})$$

Figure 12.16 Flexural Strength Design

12.4.5.2b Shear-Bond Strength Design

Shear and bond stresses, due to bending of the composite slab, develop along the interface between steel deck and concrete. These stresses must not cause horizontal slip between the two components. For this reason, a variety of shear transfer devices is used such as closely spaced embossments which resist both horizontal and vertical forces that tend to separate the deck from concrete by horizontal slippage or vertical separation. The equation for shear-bond strength can be expressed in a straight line best-fit formula as:

$$Q_n = \{k\sqrt{f_{cu}} + m\ A_s/(b\ L')\}\, b\, d \qquad \text{.......... 12.25}$$

Where:

Q_n = nominal shear bond strength per unit width
f_{cu} = compressive characteristic strength of concrete
A_s = cross-sectional area of steel deck per unit width
b = unit width, generally 1 m
d = effective slab depth measured from top of concrete to centeroid of steel-deck cross-section

L' = shear span, i.e., for two-point load test, distance from load to nearby support, Fig.12.17.
k, m = constants to be determined for each type of deck by standardized testing and evaluation procedure detailed in Fig.12.17. k is 85% of the intercept on the $Q_n/(b\,d\,\sqrt{f_{cu}})$ axis; and m is 85% of the slope of the best fit straight line for the relation between $Q_n/(b\,d\,\sqrt{f_{cu}})$ and $A_s/(bL'\sqrt{f_{cu}})$

The total factored shear forces due to dead and live loads (Q_u) must be less than the shear-bond strength of the composite slab multiplied by a resistance factor ϕ = 0.75, i.e.,

$$Q_u \le \phi Q_n$$
$$\le 0.75\, Q_n$$

12.4.5.2c Effect of Shoring

As long as the concrete placed on an unshored steel deck has nc hardened, the entire weight of the fresh concrete and the steel deck is carrie by the deck alone. Only *additional* dead loads (such as flooring) and live load applied after the concrete has hardened cause bond stresses. On the othe hand, for shored slabs, all loads cause bond stresses. This difference shoul be accounted for, both in evaluating test results and in the design of the actua structure.

For test results it is always conservative to establish the constants k anc m based on the shear Q equal to one of the two applied concentrated loads P regardless of the shoring conditions during tests.

The need for temporary shoring shall be investigated and, if required, i shall be left in place until the concrete slab attains 75% of its specifiec compressive strength.

12.4.5.3 Concrete

12.4.5.3a For unshored construction the compressive stress in concrete shall be checked against live load and flooring only. For shored construction the compressive stress in the concrete shall be checked against the total dead and live loads.

12.4.5.3b Minimum Cover of Concrete above the top of the floor deck shall be 50 mm. When additional (negative bending) reinforcement is placed over the deck, the minimum cover of concrete above the reinforcement shall be 20 mm.

12.4.5.4 Deflection

Deflection of the composite slab shall not exceed $L/360$ under the working superimposed (live) load. The deflection of the slab/deck combination can best be predicted by using the average of the cracked and uncracked moment of inertia as determined by the transformed section method of analysis.

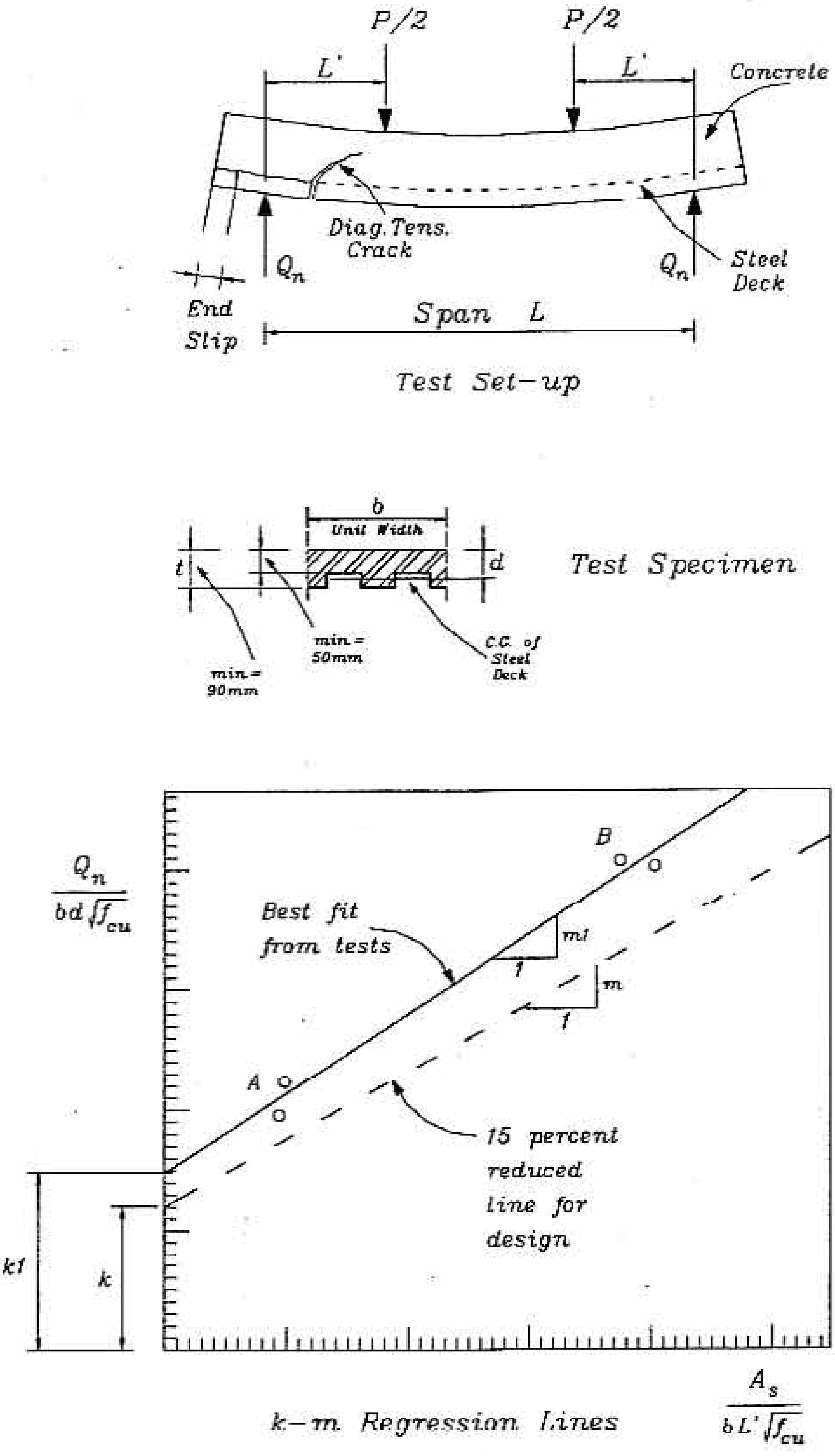


Figure 12.17 k-m Test and Regression Analysis

12.4.5.5 Temperature and Shrinkage reinforcement, consisting of welded wire fabric or reinforcing bars, shall have a minimum area of 0.001 (0.1%) times the area of concrete above the deck, but shall not be less than the area provided by 8Ø3 mm /m welded wire mesh. For those products so manufactured to use wire mesh as shear transfer means, wires welded to the top of the deck may be considered to act as shrinkage or temperature reinforcement.

A welded wire mesh used with a steel area given by the above formula, will generally not be sufficient to be the total negative reinforcement. However, the mesh has shown that it does a good job of crack control, especially if kept near the top of the slab (20 to 30 mm cover) at support location (drapped wire mesh) as shown in Fig. 12.18.

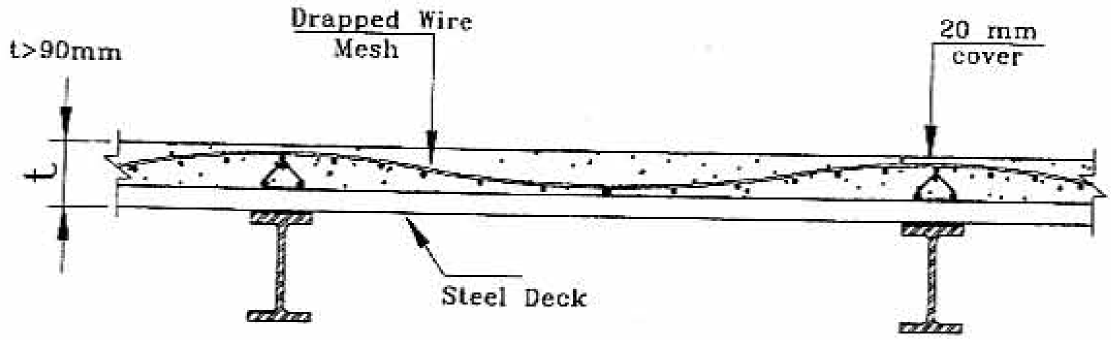


Figure 12.18 Drapped Wire Mesh

12.4.6 Construction Practice

12.4.6.1 All deck sheets shall have adequate bearing and fastening to all supports so as not to lose support during construction. Deck areas subject to heavy or repeated traffic, concentrated loads, impact loads, wheel loads, etc. shall be adequately protected by planking or other approved means to avoid overloading and /or damage during construction. Damaged deck (sheets containing distortions or deformations caused by construction practices) shall be repaired, replaced, or shored to the satisfaction of the architect before placing concrete.

For temporary construction loads prior to concreting, it should be safe to assume that the deck will support a minimum uniform load of 250 kg/m^2 without further investigation.

12.4.6.2 Prior to concrete placement, the steel deck shall be free of soil, debris, standing water, loose mill scale and all other foreign matter.

12.4.6.3 Care must be exercised when placing concrete so that the deck will not be subjected to any impact that exceeds the design capacity of the deck. Concrete shall be placed from a low level to avoid impact in a uniform manner over the supporting structure and spread towards the center of the deck span. If buggies are used to place the concrete, runways shall be planked and the buggies shall only operate on planking. Planks shall be of adequate stiffness to transfer loads to the steel deck without damaging the deck.

12.4.7 General Comments

12.4.7.1 Parking Garages

Composite floor deck may be used in parking structures provided that the following precautions are observed:

a. Slabs should be designed as continuous spans with negative bending reinforced over the supports;
b. Additional reinforcing should be included to deter cracking caused by large temperature differences and to provide load distribution; and,
c. In areas where salt water or high humidity may deteriorate the deck, protective measures must be taken. The top surface of the slab must be effectively sealed so that the water cannot migrate through the slab to the steel deck; a minimum of 390 g/m^2 galvanization (average of 27 microns of Zinc coating per side) is recommended, and, the exposed bottom surface of the deck should be additionally protected with a durable paint.

The protective measures must be maintained for the life of the building. If the protective measures cannot be assured, the steel deck can be used as a stay in place form and the concrete can be reinforced with mesh or bars as required.

12.4.7.2 Cantilevers

When cantilevers are encountered, the deck acts only as a permanent form; top reinforcing steel must be designed by the structural engineer.

12.4.7.3 Fire Ratings

Many different fire rated assemblies that use composite floor deck are available. Consult each manufacturer for a list of ratings.

12.4.7.4 Dynamic Loads

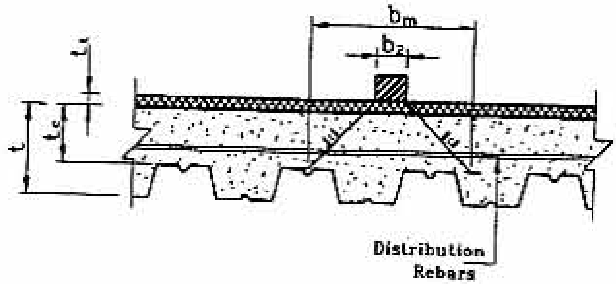
Dynamic loading, e.g., fork lifts, can, over a long period of time, interfere with the mechanical bond between the concrete and deck which achieves its composite action via web indents. Reinforcing steel running perpendicular to the deck span and placed on top of the deck ribs is often used with this type of loading to distribute concentrated loads.

12.4.7.5 Other Criteria

Composite Steel floor deck, may be used in a variety of ways, some of which do not lend themselves to a standard "steel deck" analysis for span and loading. There are, in these cases, other criteria which must be considered besides that given by this code. Engineer should make sure that this investigation starts with a review of the applicable Codes and that any special conditions are included in the design.

12.4.7.6 Concentrated and Line Loads

Composite steel floor decks subjected to concentrated-point-load bigger than 750 kg or to line-load bigger than 500 kg/m shall be reinforced under the load with proper reinforcement and anchorage length as shown in Fig.12.19.



$b_m = b_2 + 2t_c + 2t_t$

t_t =thick. of a durable topping if any

Figure 12.19 Additional Reinforcement for Concentrated and Line Loads

12.4.7.7 Concrete Slab Edges

Concrete slab edges shall be provided with end closures, e.g., channels, angles, or plates, as shown in Fig. 12.20. End closures have to be fixed to the steel beams before casting the concrete slab. Besides minimizing grout loss during casting of concrete, end closures enhance the shear connectivity between concrete slab and steel beams at zones of maximum shear forces. End closures also help in resisting forces arising from shrinkage and creep.

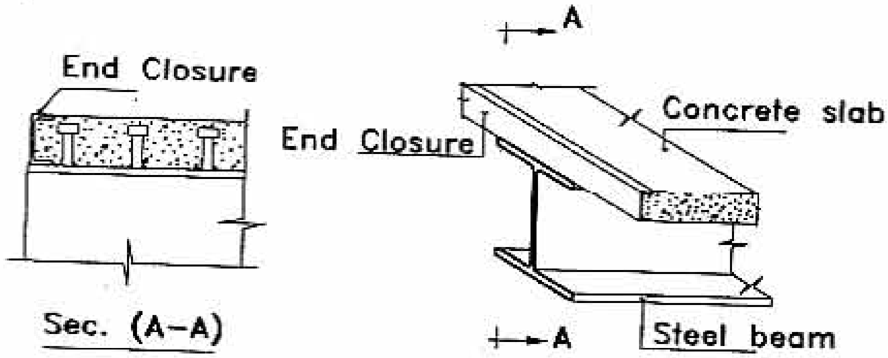


Figure 12.20 End Closure for Concrete Slab

CHAPTER 13

MEMBERS MADE OF COLD-FORMED SECTIONS

13.1 GENERAL

This Chapter shall apply to the design of members made of cold-formed steel sheet, strip or plate and used for load carrying purposes in buildings.

13.2 CLASSIFICATION OF ELEMENTS

Components of cold-formed members are generally flat slender thin plates. Cold-formed sections shall be designed as slender sections. The individual plate elements are classified as stiffened, unstiffened and multiple stiffened elements depending on the stiffening arrangement provided.

13.3 MAXIMUM AND MINIMUM THICKNESS

The provisions of this Chapter apply primarily to steel sections with a thickness of not more than 8 mm although the use of thicker material is not precluded. The minimum thickness of plates for cold-formed members used for load-carrying purposes in buildings shall be taken as 1.5 mm while for sheets the minimum thickness shall be 0.5 mm.

13.4 PROPERTIES OF SECTIONS

The properties of sections shall be determined for the full cross section of the member except that the section properties for compression elements shall be based on the effective design width as specified in Chapter 2 for stiffened elements and unstiffened elements (see Section 2.3), and the section properties for tension elements shall be based on the net area. The effective design width for compression elements with edge stiffeners or multiple stiffened elements and the stiffener requirements are detailed in Section 13.10.

13.5 MAXIMUM FLAT WIDTH -THICKNESS RATIOS FOR COMPRESSION ELEMENTS

The following Table gives the maximum allowable flat width-thickness ratios for compression elements. The definition of flat width for the different elements is shown in Fig. 13.1.

Table 13.1 Maximum Allowable Flat Width-Thickness Ratios for Compression Elements

Description	Maximum $\bar{b}/t$ or C/t
Unstiffened compression elements (C_1 and C_2).	40
Stiffened compression element having one longitudinal edge connected to a web or flange element, the other stiffened by: ($\bar{b}_1$) Simple lip. Any other kind of stiffener: *i)* when $I_s < I_a$ ii) when $I_s \geq I_a$	 60 60 90
Stiffened compression element with both longitudinal edges connected to other stiffened elements ($\bar{b}_2$).	300

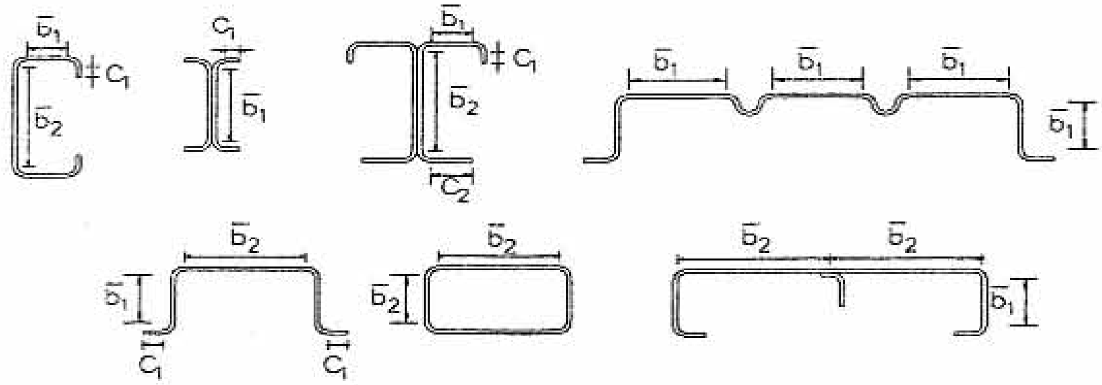


Figure 13.1 Definition of Flat Width of Compression Elements

Where I_a and I_s are the adequate and the actual moment of inertia of the stiffener as detailed in Section 13.10.2.

13.6 MAXIMUM FLAT WEB DEPTH-THICKNESS RATIOS FOR FLEXURAL MEMBERS

The following Table gives the maximum flat web width-thickness ratios for flexural elements. The definition of flat width for the web elements is shown in Fig. 13.2.

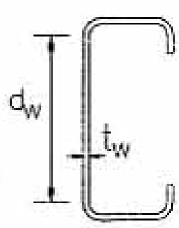


Figure 13.2 Definition of Flat Web Depth

Table 13.2 Maximum Flat Web Depth-Thickness Ratios for Flexural Members

Description	Maximum d_w/t_w
Unstiffened webs	200
Webs which are provided with bearing stiffeners only	260
Webs which are provided with bearing stiffeners and intermediate transverse stiffeners	300

Where:

d_w = depth of flat portion of web measured along the plane of web
t_w = web thickness

Where a web consists of two or more sheets, the d_w/t_w ratio shall be computed for the individual sheets.

13.7 MAXIMUM ALLOWABLE DEFLECTION

The following table gives recommended deflection limits for some structural members. Circumstances may arise where greater or lesser values would be more appropriate. Other members may also require a deflection limit to be specified, e.g., sway bracing.

The determination of the moment of inertia, I, used in computing beam deflection, shall be based on the effective section properties, for which the effective widths are computed for the compressive stresses developed from the applied bending moment. The actual compressive stresses due to applied moment shall be used to compute the normalized plate slenderness, $\bar{\lambda}_p$, rather than F_y in Section 2.3.

Table 13.3 Maximum Allowable Deflection

Vertical deflection of beams (δ_2 as defined in Section 14.2) due to unfactored live load without dynamic effect	
Beams carrying plaster or other brittle finish	Span / 300
All other beams	Span / 200
Cantilevers	2 x Length / 180
Purlins and side girts (rails)	To suit the characteristics of the particular cladding system.
Horizontal deflection of columns in other than pitched roof frames due to unfactored live and wind loads	
Tops of columns in single-storey buildings	Height / 300
In each storey of a building with more than one storey	Height of storey under consideration / 300

13.8 EFFECT OF COLD FORMING

The increase in yield strength due to cold forming may be considered by replacing the material yield strength, F_y, by, F_{ya}, the average yield strength of the cold-formed section. The value of F_{ya} may be determined from the following relationship:

$$F_{ya} = F_y + \frac{5\ N\ t^2}{A}(F_u - F_y) \leq 1.15\ F_y \quad \text{.........................} \quad 13.1$$

Where:

N = is the number of full 90° bends in the compression part of the section with an internal radius $\leq 5t$. Fractions of 90° bends should be counted as fractions of N,

A = is the gross area of the full flange of flexural members (including the corners and excluding the lips) or the full cross sectional area of tension or compression members under consideration

The full effect of cold working on the yield strength may be used for calculating the tensile strength of elements. For elements of flat width, $\bar{b}$, and thickness, t, under compression, the value of F_{ya} should be modified as follows to provide the appropriate compression yield strength, F_{yac}.

For stiffened elements:

$$\text{for } \bar{b} / t \leq 24\sqrt{2.8 / F_y} \qquad F_{yac} = F_{ya} \quad \text{.........................} \quad 13.2$$

$$\text{for } \bar{b} / t \geq 48\sqrt{2.8 / F_y} \qquad F_{yac} = F_y \quad \text{.........................} \quad 13.3$$

For unstiffened elements:

$$\text{for } \bar{b} / t \leq 8\sqrt{2.8 / F_y} \qquad F_{yac} = F_{ya} \quad \text{.........................} \quad 13.4$$

$$\text{for } \bar{b} / t \geq 16\sqrt{2.8 / F_y} \qquad F_{yac} = F_y \quad \text{.........................} \quad 13.5$$

The increase in yield strength due to cold working should not be utilized for members that undergo welding, annealing, galvanizing or any other heat treatment after forming that may produce softening.

13.9 DESIGN STRENGTH OF MEMBERS

The design strength of members shall follow the requirements presented earlier in Chapter 3 for tension members, Chapter 4 for compression members, and Chapter 5 for flexural members. However, the slender section properties of compression elements in compression members or beams shall be based on the effective section properties, as detailed in Chapter 2 for elements under uniform stress or under stress gradient. The design strength for cylindrical tubular members shall be as given in Section 13.13.

13.10 EFFECTIVE WIDTH OF OTHER COMPRESSION ELEMENTS

13.10.1 Effective Width of Uniformly Compressed Elements with Circular Holes

The effective width b_e shall be determined as follows:

For $0.5 \geq d_h / \bar{b} \geq 0$ and $\bar{b}/t \leq 0.7$, d_h = diameter of holes and the distance between centers of holes $\geq 0.5\ \bar{b}$ and $\geq 3\ d_h$, the effective width is calculated using a reduction factor ρ as $b_e = \rho\ \bar{b}$.

Where:

$$\rho = (\bar{\lambda}_p - 0.15 - 0.05\ \psi - 0.8\ d_h / \bar{b}) / \bar{\lambda}_p^{\,2} \leq 1 \qquad 13.6$$

And

$\bar{\lambda}_p$ = the normalized plate slenderness as detailed in Chapter 2.

13.10.2 Effective Widths of Compression Elements with an Edge Stiffener or an Intermediate Stiffener

13.10.2.1 Effective Width of Uniformly Compressed Elements with an Edge Stiffener

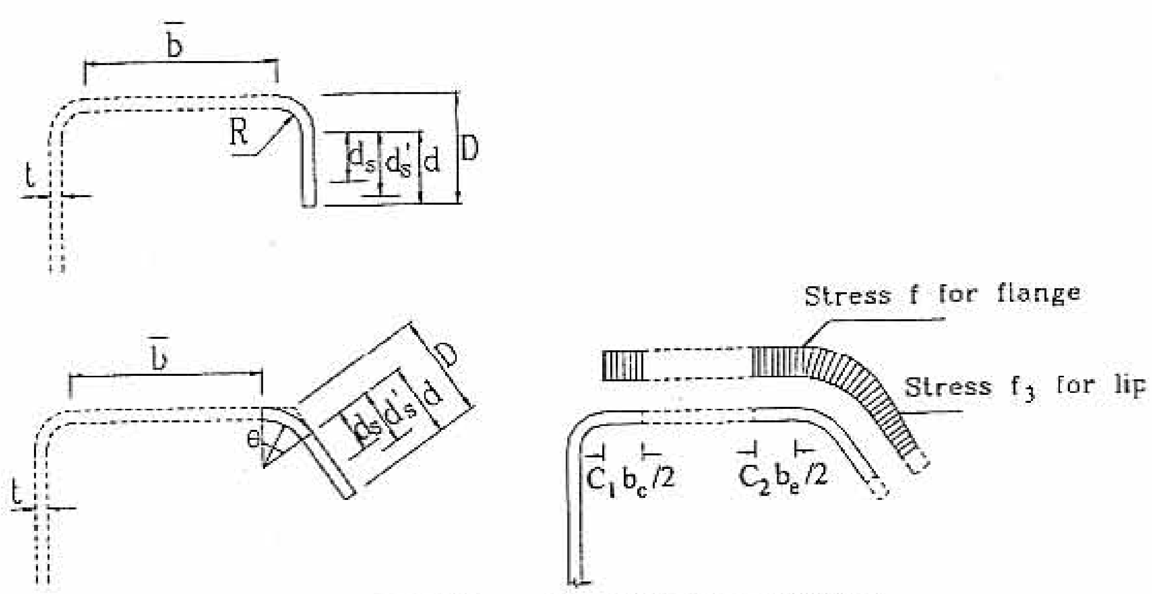


Figure 13.3 Elements with Edge Stiffener

EFFECTIVE WIDTH OF COMPRESSION ELEMENTS WITH AN EDGE STIFFENER

Calculate S, $\bar{b}/t$ and $D/\bar{b}$				
For $\bar{b}/t \le S/3$ and $D/d \le 0.25$ ……………13.7	For $S/3 < \bar{b}/t < S$ Calculate $D/\bar{b}$ ……………13.8		For $\bar{b}/t \ge S$ Calculate $D/\bar{b}$ ……………13.9	
$I_a = 0$ (no edge stiffener required) $b_e = \bar{b}$	$0.25 < D/\bar{b} \le 0.8$	$0.25 \ge D/\bar{b}$	$0.25 < D/\bar{b} \le 0.8$	$0.25 \ge D/\bar{b}$
	$k_\sigma = [4.82 - 5(D/\bar{b})] (I_s/I_a)^{1/2} + 0.43 \le 5.25 - 5(D/\bar{b})$	$k_\sigma = 3.57\,(I_s/I_a)^{1/2} + 0.43 \le 4$	$k_\sigma = [4.82 - 5(D/\bar{b})] (I_s/I_a)^{1/3} + 0.43 \le 5.25 - 5(D/\bar{b})$	$k_\sigma = 3.57\,(I_s/I_a)^{1/3} + 0.43 \le 4$
	$I_a = 399\,\{(\bar{b}/t)/S - 0.33\}^3\, t^4$ $I_s = td^3/12$		$I_a = \{[115\,(\bar{b}/t)/S] + 5\}\, t^4$ $I_s = td^3/12$	
	$C_2 = I_s/I_a \le 1$ and $C_1 = 2 - C_2$ $d_s = C_2\, d_s' = (I_s/I_a)\, d_s'$ $A_s' = d_s'\, t$ and $A_s = (I_s/I_a)\, A_s'$		$C_2 = I_s/I_a \le 1$ and $C_1 = 2 - C_2$ $d_s = C_2\, d_s' = (I_s/I_a)\, d_s'$ $A_s' = d_s'\, t$ and $A_s = (I_s/I_a)\, A_s'$	
	Then calculate $\bar{\lambda}_p = \frac{\bar{b}/t}{44}\sqrt{\frac{F_y}{k_\sigma}}$, for ψ=1 calculate $b_e = \rho\,\bar{b}$ where, $\rho = (\bar{\lambda}_p - 0.15 - 0.05\psi)/\bar{\lambda}_p^2 \le 1$			
	Effective width of stiffener is determined as: $d_s' = \rho\, d$ with $k_\sigma = 0.43$		Effective width of stiffener is determined as: $d_s' = \rho\, d$ with $k_\sigma = 0.43$	

In the previous equations:

S	$= 1.28\sqrt{E/F_y}$	13.10
F_y	= yield stress	
K_σ	= plate buckling factor	
b_0	= dimension defined in Fig. 13.4	
$D, d, \overline{b}$	= dimensions defined in Fig. 13.3	
d_s	= reduced effective width of the stiffener, d_s shall be used in computing the overall effective section properties	
D'_s	= effective width of the stiffener according to Table 2.12	
C_1, C_2	= coefficients defined according to Fig. 13.3 to calculate the effective width instead of Table 2.12	
A_s	= reduced area of the stiffener. It shall be used in computing the overall effective section properties. The centroid of the stiffener is to be considered located at the centroid of the full area of the stiffener	
I_a	= adequate moment of inertia of the stiffener, so that each component element can behave as a stiffened element	
I_s, A'_s	= moment of inertia of the full section of the stiffener about its own centroidal axis parallel to the element to be stiffened, and the effective area of the stiffener, respectively. For edge stiffeners, the round corner between the stiffener and the element to be stiffened shall not be considered as part of the stiffener	

For the stiffener shown in Fig. 13.3.

I_s	$= (d^3 t \sin^2\theta)/12$	13.11
A'_s	$= d'_s t$	13.12

13.10.2.2 Effective Width of Uniformly Compressed Elements with One Intermediate Stiffener

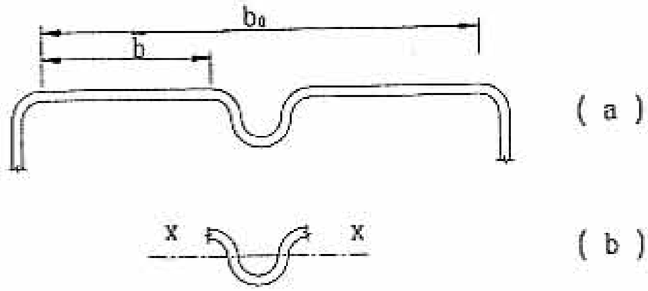


Figure 13.4 Elements with One Intermediate Stiffener

EFFECTIVE WIDTH OF COMPRESSION ELEMENTS WITH ONE INTERMEDIATE STIFFENER

<table>
<tr><td colspan="3">Calculate S, b_o/t</td></tr>
<tr><td>For $b_o/t \leq S$ ....13.13</td><td>For $S < b_o/t < 3S$ ....13.14</td><td>For $b_o/t \geq 3S$ ....13.15</td></tr>
<tr><td rowspan="3">$I_a = 0$
(no intermediate stiffener required)
$b_e = \bar{b}$
$A_s = A'_s$</td><td>$k_\sigma = 3(I_s/I_a)^{1/2} + 1 \leq 4$
$A_s = A'_s\,(I_s/I_a) \leq A'_s$</td><td>$k_\sigma = 3\,(I_s/I_a)^{1/3} + 1 \leq 4$
$A_s = A'_s\,(I_s/I_a) \leq A'_s$</td></tr>
<tr><td>$I_a = \{[50\,(b_o/t)\,/\,S] - 50\}\,t^4$
$I_s = td^3/12$</td><td>$I_a = \{[128(b_o/t)\,/\,S] - 285\}\,t^4$
$I_s = td^3/12$</td></tr>
<tr><td colspan="2">Then calculate
$\bar{\lambda}_p = \frac{\bar{b}/t}{44}\sqrt{\frac{F_y}{k_\sigma}}$
$\psi = 1$
$\rho = (\bar{\lambda}_p - 0.15 - 0.05\psi)/\bar{\lambda}_p^2 \leq 1$
$b_e = \rho\,\bar{b}$</td></tr>
</table>

13.10.3 Effective Width of Edge Stiffened Elements with Intermediate Stiffeners or Stiffened Elements with More than One Intermediate Stiffener

13.10.3.1 Minimum Intermediate Stiffener Inertia

Intermediate stiffeners of an edge stiffened element or the stiffeners of a stiffened element with more than one stiffener as shown in Fig. 13.5 shall have a minimum moment of inertia (I_{min} in cm^4) given by:

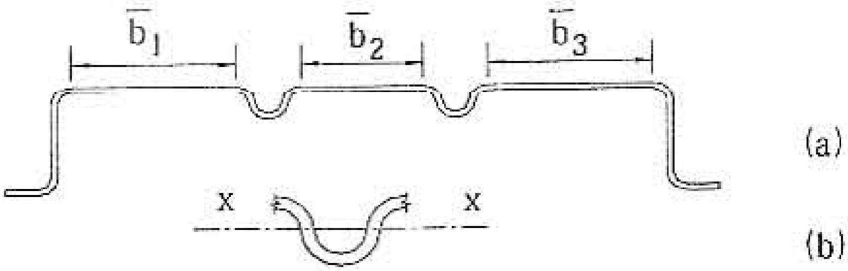


Figure 13.5 Sections with Multiple-Stiffened Compression Elements

$$I_{min} = 3.66t^4 \sqrt{(\bar{b}/t)^2 - 280/F_y} \geq 18.4t^4 \qquad 13.16$$

Where:

I_{min} = minimum moment of inertia of the full stiffener about its own centroidal axis parallel to the element to be stiffened

$\bar{b}/t$ = flat-width to thickness ratio of the larger stiffened sub element

13.10.3.2 Effective Design Width of Sub Elements

For multiple-stiffened compression elements, the effective widths of sub elements are determined by the following Equations:

a- If $\bar{b}/t \leq 60$ $\qquad b_{em} = b_e$ 13.17

b- If $\bar{b}/t > 60$ $\qquad b_{em} = b_e - 0.10\, t\, [\, \bar{b}/t - 60]$ 13.18

Where:

$\bar{b}/t$ = flat-width to thickness ratio of element or sub element

b_{em} = effective design width of element or sub element to be used in design computations

b_e = effective design width determined for single-stiffened compression element (cm) with $\bar{b}$ as shown in Fig. 13.5a (refer to Table 2.13)

13.10.3.3 Effective Stiffener Area

In computing the effective structural properties for a member having intermediate stiffeners and when the $\bar{b}/t$ ratio of the sub element exceeds 60, the effective stiffener area (A_{eff}) (edge stiffener or intermediate stiffeners) shall be computed as follows:

a- If $\bar{b}/t \leq 60$ $\quad A_{eff} = A_{st}$ 13.19

b- If $60 < \bar{b}/t < 90$ $\quad A_{eff} = \alpha \,.\, A_{st}$ 13.20

Where:

$$\alpha = [3 - 2\, b_{em}/\bar{b}] - 1/30\, [1 - b_{em}/\bar{b}]\, [\bar{b}/t]$$

c- If $\bar{b}/t \geq 90$ $\quad A_{eff} = [b_{em}/\bar{b}].A_{st}$ 13.21

Where A_{st} is the area of the relevant stiffener, Fig. 13.5.b.

In the above Equation, A_{eff} and A_{st} refer to the area of the stiffener section, exclusive of any portion of adjacent element. In the calculation of sectional properties, the centroid of the full section of the stiffener and the moment of inertia of the stiffener about its own centroidal axis shall be that of the full section of the stiffener.

13.11 BEAMS WITH UNUSUALLY WIDE FLANGES

For beams with unusually wide flanges, special consideration shall be given to the effects of shear lag and flange curling, even if the beam flanges, such as tension flanges, do not buckle.

13.11.1 Shear Lag Effect

The ratio of effective flange width to the actual width as per Section 2.3.1.3 shall not exceed the values specified in Table 13.4. The effective span length of the beam, L, is the full span for simple beams, the distance between inflection points for continuous beams, or twice the length of cantilever beams. The symbol, b_f, is defined as shown in Fig. 13.6.

Table 13.4 Maximum Ratio of Effective Flange Width to Actual Width

L/b_f	Effective Flange Width / Actual Flange Width	L/b_f	Effective Flange Width / Actual Flange Width
30	1.00	14	0.82
25	0.96	12	0.78
20	0.91	10	0.73
18	0.89	8	0.67
16	0.86	6	0.55

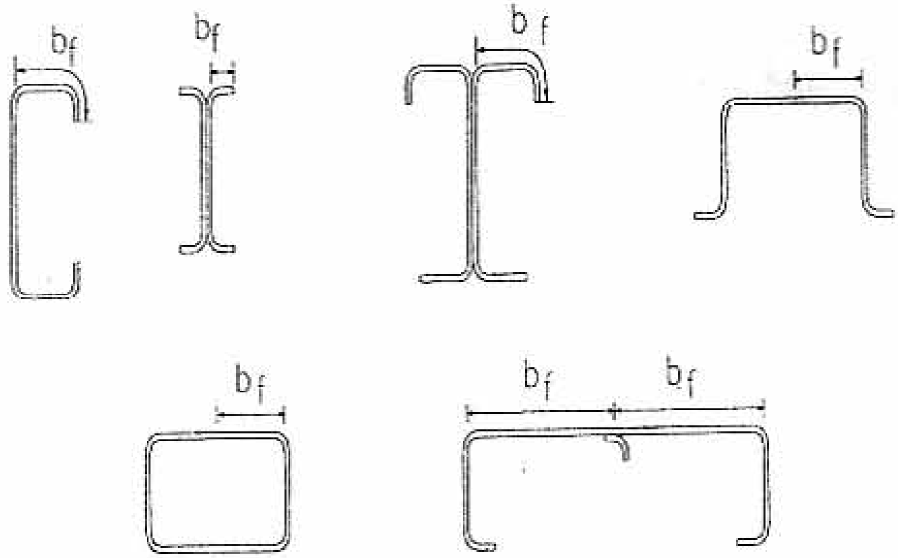


Figure 13.6 Definition of (b_f) of Compression Elements

13.11.2 Flange Curling

The width of the flange projection beyond the web, b_f, for C-beams or similar, or half the distance between webs of multiple web sections (whether the flange is in tension or compression, stiffened or unstiffened) shall not exceed the following to avoid flange curling:

$$b_f = 0.37 \{t\ d\ E / f_{av}\}^{1/2} \quad \ldots\ldots \quad 13.22$$

Where:

t = flange thickness, cm
d = overall depth of the section, cm
f_{av} = average bending stresses in the flange in full, unreduced flange width, t/cm^2

13.12 WEB REQUIREMENTS
13.12.1 Web Stiffeners
13.12.1.1 Bearing Stiffeners

Transverse bearing stiffeners attached to beam webs at points of concentrated loads or reactions shall be designed as compression members. Concentrated loads or reactions shall be applied directly into the stiffeners, or the stiffener shall be fitted accurately to the flat portion of the flange to provide direct load bearing into the end of the stiffener.

The nominal strength of a bearing stiffener equals ϕP_n where P_n is the smaller value given by Equations 13.23 or 13.24 as follows:

$$P_n = F_y A_c \quad \ldots\ldots \quad 13.23$$
$$P_n = F_{cr} A_b \quad \ldots\ldots \quad 13.24$$
$$\phi = 0.85$$

Where:

A_c = $18t^2 + A_s$ for bearing stiffeners at interior support and under concentrated load
A_c = $10t^2 + A_s$ for bearing stiffeners at end support
F_{cr} = the critical buckling stress as defined in Chapter 4
A_b = $b_1 t + A_s$ for bearing stiffeners at interior support and under concentrated load
A_b = $b_2 t + A_s$ for bearing stiffeners at end support
A_s = Cross sectional area of bearing stiffeners
b_1 = $25\,t\,[\,0.0024\,(\,L_{st}\,/\,t\,) + 0.70\,] \leq 25\,t$
b_2 = $12\,t\,[\,0.0044\,(\,L_{st}\,/\,t\,) + 0.80\,] \leq 12\,t$
L_{st} = length of bearing stiffeners
t = base thickness of beam web

The $\bar{b}\,/\,t_s$ ratio for transverse bearing stiffeners shall not exceed:

a- $58\,/\sqrt{F_y}$ for stiffened elements

b- $16.9\,/\sqrt{F_y}$ for unstiffened elements

Where: t_s is the thickness of the stiffener.

13.12.1.2 Transverse Intermediate Shear Stiffeners

Where transverse shear stiffeners are required, the spacing shall be based on the nominal shear strength, and the ratio $(a\backslash h)$ shall not exceed $\{260/(h\backslash t)^2\}$ nor 3.0.

All transverse shear stiffeners should be designed to satisfy the following requirements for spacing, moment of inertia, and gross area:

- Spacing a between stiffeners

$$a < (\,0.75\,(\,1.5\,)\,h \qquad 13.25$$

- Moment of inertia of transverse shear stiffeners with reference to an axis in the plane of the web is the greater of:

$$I_s \geq 5\,h\,t^3\,[(\,h\,/\,a\,) - 0.7\,(\,a\,/\,h\,)] \qquad 13.26$$
$$I_s \geq (\,h\,/\,50\,)^4 \qquad 13.27$$

- Gross area A_s of transverse shear stiffeners

$$A_s \geq \frac{1-C_v}{2}\left[\frac{a}{h} - \frac{(a/h)^2}{(a/h)+\sqrt{1+(a/h)^2}}\right]D.h.t \qquad 13.28$$

$$C_v = \frac{45000.k_v}{F_y(h/t)^2} \quad \text{if } C_v \leq 0.8 \qquad 13.29$$

$$C_v = \frac{190}{(h/t)}\sqrt{k_v/F_y} \quad \text{if } C_v > 0.8 \qquad 13.30$$

Where:

k_v = $4.00 + 5.34/(a/h)^2$ if $a/h \le 1.0$

k_v = $5.34 + 4.00/(a/h)^2$ if $a/h > 1.0$

a = distance between transverse shear stiffeners

h = depth of flat portion of web measured along the plane of web

D = 1.0 for stiffeners furnished in pairs

= 1.8 for single-angle stiffeners

= 2.4 for single-plate stiffeners

13.12.2 Web Crippling Strength

The nominal web crippling strength $\phi_w P_n$ (in tons) shall be determined from Tables 13.5 and 13.6 . The equations shall be applied to beams having $h/t < 200$ and $R/t < 6$.

P_n represents the nominal strength for concentrated load or reaction for one solid web connecting top and bottom flanges. For two or more webs, P_n shall be computed for each individual web and the results added to obtain the nominal load or reaction for the multiple webs.

Transverse bearing stiffeners attached to beam webs at points of concentrated loads or reactions and satisfying the conditions of Section 13.12.1.2 shall eliminate the need to check the web crippling strength.

The following shall define the different constants used to determine P_n :

C_1 = $1.22 - 0.1\,F_y$

C_2 = $1.06 - 0.06\,R/t \le 1.0$

C_3 = $1.33 - 0.14\,F_y$

C_4 = $1.15 - 0.15\,R/t \le 1.0$ but not less than 0.50

C_5 = $1.49 - 0.53\,F_y \ge 0.6$

C_6 = $0.88 + 0.063\,t$

C_7 = $1 + \frac{h/t}{750}$ when $h/t \le 150$

= 1.20 when $h/t > 150$

C_8 = $2.35/F_y$, when $h/t \le 66.5$

= $[1.1 - \frac{h/t}{665}](\frac{2.35}{F_y})$, when $h/t > 66.5$

C_9 = $0.82 + 0.079\,t$

C_{10} = $[0.98 - \frac{h/t}{865}](\frac{2.35}{F_y})$

C_{11} = $0.64 + 0.163\,t$

C_{12} = $0.7 + 0.3\,(\theta/90)^2$

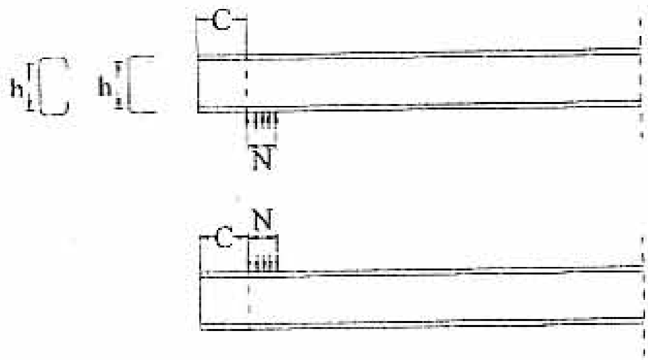
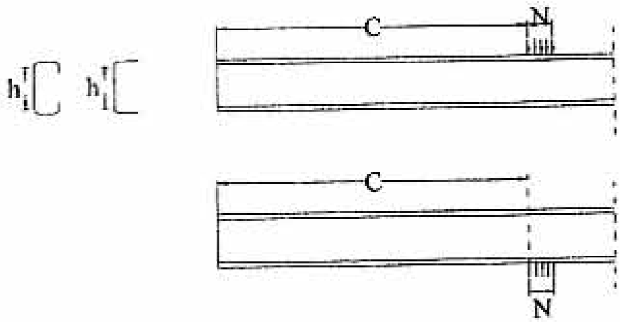
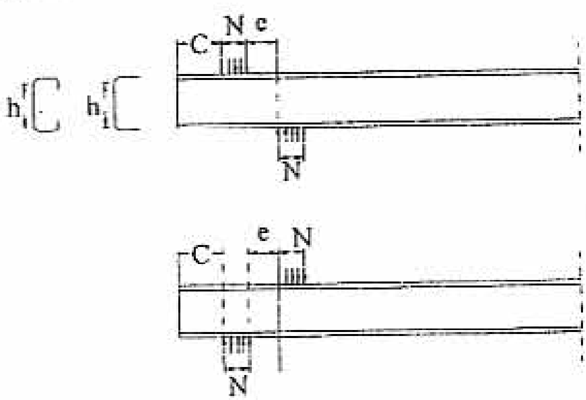
C_{13}	=	$2060 - 3.8\ (h/t)$
C_{14}	=	$1350 - 1.73\ (h/t)$
C_{15}	=	$3350 - 4.6\ (h/t)$
C_{16}	=	$1520 - 3.57\ (h/t)$
C_{17}	=	$4380 - 14\ (h/t)$
C_{18}	=	$1 + 0.004\ (N/t)$
C_{19}	=	$1 + 0.0012\ (N/t)$
C_{20}	=	$1 + 0.0013\ (N/t)$
C_{21}	=	$7.4 + 1.11\sqrt{N/t}$
C_{22}	=	$11.2 + 1.63\ \sqrt{N/t}$
h	=	depth of the flat portion of the web measured along the plane of the web.
N	=	actual length of bearing. For the case of two equal and opposite concentrated loads distributed over unequal bearing lengths, the smaller value of N shall be taken.
θ	=	angle between the plane of the web and the plane of the bearing surface, where $45^{o} \leq \theta \leq 90^{o}$
k	=	$0.425\ F_y$

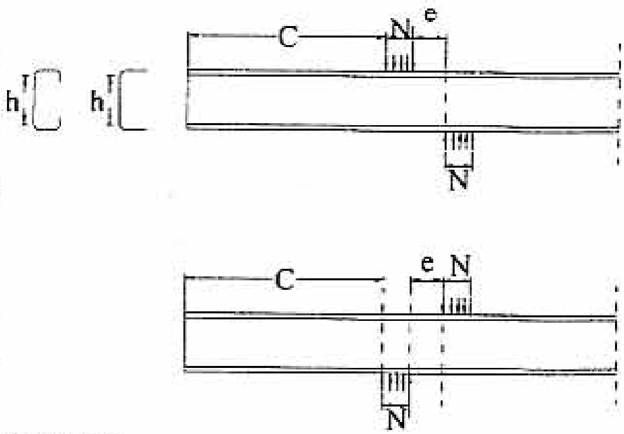
Notes:
F_y is in t/cm^2
R, h, t and N are in mm

For single unreinforced webs (without intermediate stiffeners):
$\phi_w = 0.70$

For I – sections:
$\phi_w = 0.75$

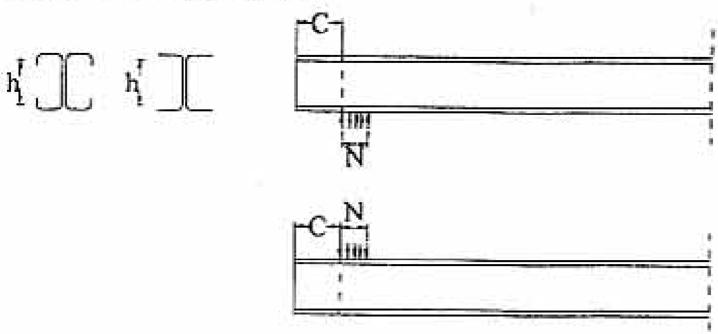
Table 13.5 Total Web Resistance, P_n, for Shapes Having Single Thickness Webs

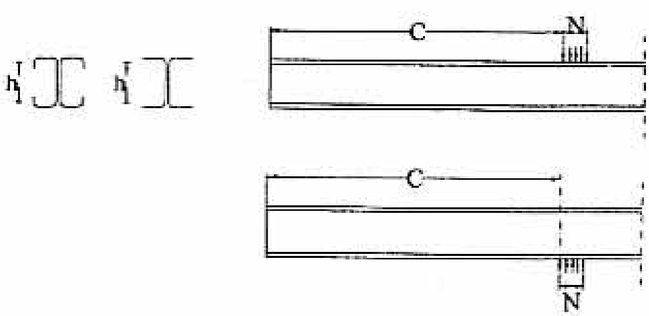
Type and position of loadings	Total web resistance, P_n, tons
Single load or reaction  $C \leq 1.5\,h$ Load or reaction near or at free end	Stiffened flanges $P_n = t^2\, kC_3C_4C_{12}\, C_{13}C_{18} \times 10^{-4}$ Unstiffened flanges $P_n = t^2\, kC_3C_4C_{12}\, C_{14}C_{18} \times 10^{-4}$
Single load or reaction  $C > 1.5\,h$ Load or reaction far from free end	Stiffened and unstiffened flanges $P_n = t^2\, kC_1C_2C_{12}\, C_{15}C_{19} \times 10^{-4}$
Two opposite loads or reactions $e < 1.5\,h$  $C \leq 1.5\,h$ Load or reactions near or at free end	Stiffened and unstiffened flanges $P_n = t^2\, kC_3C_4C_{12}\, C_{16}C_{18} \times 10^{-4}$

<table>
<tr>
<td>Two opposite loads or reactions
$e < 1.5\,h$


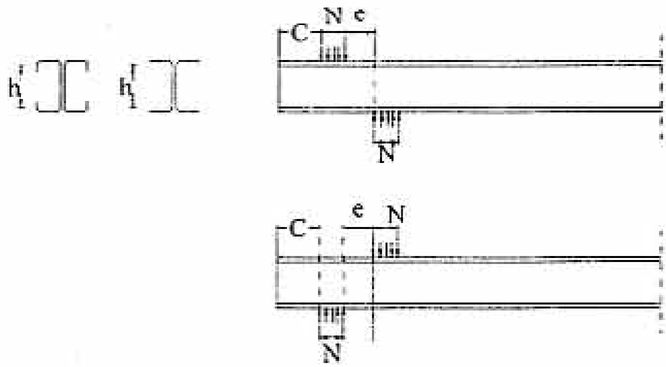
$C > 1.5\,h$
Load or reactions far from free end</td>
<td>Stiffened and unstiffened flanges
$P_n = t^2\,kC_1C_2C_{12}\,C_{17}C_{20} \times 10^{-4}$</td>
</tr>
</table>

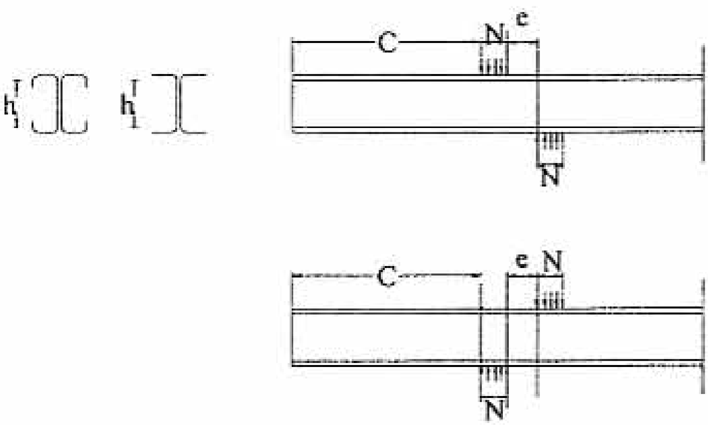
Table 13.6 Total Web Resistance, P_n, For I - Beams and Beams With Restraint Against Web Rotation

<table>
<tr>
<th>Type and position of loadings</th>
<th>Total web resistance, P_n, tons</th>
</tr>
<tr>
<td>Single load or reaction


$C \leq 1.5\,h$
Load or reaction near or at free end</td>
<td>Stiffened and unstiffened flanges
$P_n = t^2F_yC_7\,C_{21} \times 10^{-2}$</td>
</tr>
<tr>
<td>Single load or reaction


$C > 1.5\,h$
Load or reaction far from free end</td>
<td>Stiffened and unstiffened flanges
$P_n = t^2F_yC_5\,C_6\,C_{22} \times 10^{-2}$</td>
</tr>
</table>

<table>
<tr>
<td>Two opposite loads or reactions
$e < 1.5\,h$

$C \leq 1.5\,h$
Load or reactions near or at free end</td>
<td>Stiffened and unstiffened flanges

$P_n = t^2 F_y C_{10}\, C_{11}\, C_{21} \times 10^{-2}$</td>
</tr>
<tr>
<td>Two opposite loads or reactions
$e < 1.5\,h$

$C > 1.5\,h$
Load or reactions far from free end</td>
<td>Stiffened and unstiffened flanges

$P_n = t^2 F_y C_8\, C_9\, C_{22} \times 10^{-2}$</td>
</tr>
</table>

13.12.3 Combined Bending and Web Crippling Strength

Unreinforced flat webs of shapes subjected to a combination of bending and concentrated load or reaction shall be designed to meet the following requirements:

a– For shapes having single unreinforced webs (without intermediate stiffeners):

$$1.07\left(\frac{P_u}{P_n\,\phi_w}\right)+\left(\frac{M_u}{M_{nxo}\,\phi_b}\right) \leq 1.42 \qquad 13.31$$

b– For shapes having multiple unreinforced webs (without intermediate stiffeners) such as I-sections made of two C sections connected back to back:

$$0.82\left(\frac{P_u}{P_n\,\phi_w}\right)+\left(\frac{M_u}{M_{nxo}\phi_b}\right)\le 1.32 \qquad 13.32$$

Where:

- ϕ_b = resistance factor for bending
- ϕ_w = resistance factor for web crippling
- P_u = required strength for the concentrated load or reaction in the presence of bending moment
- P_n = nominal strength for the concentrated load or reaction in the absence of bending moment
- M_u = required flexural strength
- M_{nxo} = nominal flexural strength about x axis
- w = flat width of the beam flange which contacts the bearing plate
- t = thickness of the web or flange

c– For the support point of two nested Z shapes

$$\left(\frac{M_u}{M_{no}}\right)+\left(\frac{P_u}{P_n}\right)\le 1.68\ \phi_w \qquad 13.33$$

Where:

- P_u = required strength for the concentrated load or reaction in the presence of bending moment
- P_n = nominal web crippling strength
- M_u = required flexural strength at the section under consideration
- M_{no} = nominal flexural strength
- ϕ_w = 0.85

13.13 CYLINDRICAL TUBULAR MEMBERS

The thickness of the cylindrical tubular members shall be chosen such that the ratio of outside diameter to wall thickness, D/t, shall not exceed $735 / F_y$.

13.13.1 Slenderness Ratios

The maximum slenderness ratios of cylindrical tubular members shall be according to Section 2.2.1.2.

13.13.2 Effective Buckling Length ($K\ell$)

The effective buckling length ($K\ell$) of a cylindrical tubular member may be taken from Table 2.5, or obtained from an elastic critical buckling analysis.

13.13.3 Design Strength for Members under Compression

The following equations shall be used to define the nominal design strength of compression members for circular tubes $\phi_c P_n$:

For $\lambda_c \leq 1.1$ $\quad P_n = \left[1 - 0.384\lambda_c^2\right] A_e . F_y$ 13.34

For $\lambda_c > 1.1$ $\quad P_n = \left[\dfrac{0.648}{\lambda_c^2}\right] A_e . F_y$ 13.35

$\phi_c = 0.8$

Where:

$\lambda_c = \sqrt{F_y / F_e}$

F_e = The flexural buckling stress

$= \dfrac{\pi^2 E}{(K\ell / r)^2}$

The effective area to be used for calculating the axial strength, A_e, shall be determined as follows:

$$A_e = [1 - (1 - \frac{F_y}{2F_e})(1 - A_o/A)].A \qquad 13.36$$

Where:

A = Area of the full, unreduced cross section.

$$A_o = \left[\frac{75}{(D/t)F_y} + 0.667\right].A \leq A \qquad 13.37$$

13.13.4 Design Strength for Members under Bending

The design flexural strength in a cylindrical tubular member $\phi_b M_n$ shall be calculated as follows:

For $D/t \leq 140/F_y$

$M_n = 1.2\, S_e . F_y$ 13.38

For $140 / F_y < D/t \leq 580 / F_y$

$$M_n = \left[0.90 + \left(\frac{40 / F_y}{D / t}\right)\right] S_e . F_y \qquad 13.39$$

For $580 / F_y < D/t \leq 735 / F_y$

$$M_n = \left[\frac{410 / F_y}{D / t}\right] . S_e\, F_y \qquad 13.40$$

Where F_y is in t/cm^2
In all cases $\phi_b = 0.9$

The elastic section modulus S_e to be used in the calculations shall be for the full, unreduced cross section.

13.13.5 Design Strength for Members under Combined Bending and Compression

Combined bending and compression shall satisfy the requirements of Chapter 7.

13.14 SPLICES

Splices in compression or tension members shall be designed on the actual forces in the members.

13.15 CONNECTIONS

Connections of members at an intersection shall be designed on the actual forces in the members.

13.15.1 Welded Connections

The following design criteria govern Arc welded connections used for cold-formed steel structural members in which the thickness of the thinnest connected part is 4 mm or less. For welded connections where the thickness of the thinnest connected part is greater than 4 mm, the provisions of Chapter 9 shall apply.

Resistance welds, which are produced by the heat obtained from resistance to an electric current through the work parts held together under pressure by electrodes, are possible.

13.15.1.1 Arc Welds

Several types of arc welds are generally used in cold-formed steel construction such as:

- a- Groove welds
- b- Arc spot welds
- c- Arc seam welds
- d- Fillet welds
- e- Flare groove welds

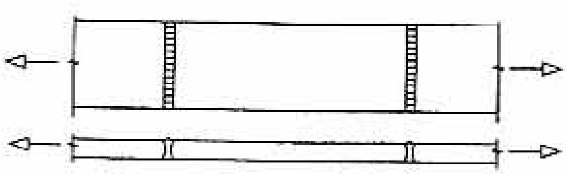


Figure 13.7 Groove Welds in Butt Joints

13.15.1.1.1 Groove Welds in Butt Joints

The nominal strength ϕP_n of a groove weld in a butt joint, welded from one or both sides, shall be determined as follows:

The Nominal strength for Groove Welds in Butt Joints		
Tension or compression normal to the effective area or parallel to the axis of the weld	Shear on the effective area , the smaller of either Eq (a) or (b)	
$P_n = L t_e F_y$ $\phi = 0.85$ 13.41	(a) $P_n = 0.6\, L\, t_e\, F_u$ $\phi = 0.75$ 13.42	(b) $P_n = L\, t_e\, F_y / \sqrt{3}$ $\phi = 0.85$...........13.43

Where:

P_n = nominal strength of a groove weld
F_u = specified minimum tensile strength of steel
F_y = specified minimum yield point of the lowest strength base steel
L = length of weld
t_e = effective throat dimension for groove weld

13.15.1.1.2 Arc Spot Welds

a- Arc spot welds should not be used to weld steel sheets where the thinnest connected part is over 4 mm thick, nor through a combination of steel sheets having a total thickness of over 4 mm.
b- Weld washers should be used when the thickness of the sheet is less than 0.7 mm. Weld washers should have a thickness of between 1.3 mm and 2 mm with a minimum pre-punched hole of 10 mm diameter.
c- The minimum allowable effective diameter d_e is 10 mm.
d- The distance measured along the line of application of force from the centerline of a weld to the nearest edge of an adjacent weld or to the end of the connected part toward which the force is directed should not be less than the value of e_{min} as given by:

$$e_{min} = \frac{P_u}{\phi\, F_u\, t} \quad \text{.......} \quad 13.44$$

Where:

P_u = factored force transmitted by an arc spot weld
t = thickness of thinnest connected sheet
F_u = specified minimum tensile strength of steel (base metal)
ϕ = 0.70

e- The distance from the centerline of any weld to the end or boundary of the connected member should not be less than 1.5d. In no case should the clear distance between welds and end of member be less than d.

The Nominal strength for an Arc Spot Weld			
Nominal load based on shear capacity of weld	Nominal load based on strength of connected sheets calculate d_a/t		
$P_n = 0.72\, F_u\, \pi\, d_e^2 / 4$ $\phi = 0.55$......13.45	For $36/\sqrt{F_u} \geq d_a/t$	For $36/\sqrt{F_u} < d_a/t < 64/\sqrt{F_u}$	For $d_a/t \geq 64/\sqrt{F_u}$
	$P_n = 2.0\, F_u\, d_a\, t$ $\phi = 0.55$...13.46	$P_n = 0.26\, F_u \left(1 + \dfrac{256\sqrt{F_u}}{d_a/t}\right) d_a\, t$ $\phi = 0.45$......13.47	$P_n = 1.25\, F_u\, d_a\, t$ $\phi = 0.45$ 13.48

f- The nominal shear on each arc spot weld ϕP_n between sheet or sheets and supporting member shall not exceed the smaller value of the loads computed by the following equations:

Where:

- d = visible diameter of outer surface of arc spot weld
- d_a = average diameter of arc spot weld at mid-thickness of t (as shown in Fig. 13.8) = $d - t$ for single sheet, and = $d - 2t$ for multiple sheets (not more than four lapped sheets over a supporting member)
- d_e = effective diameter of fused area = $0.7\, d - 1.5\, t \leq 0.55\, d$.
- t = total combined base steel thickness (exclusive of coating) of sheets involved in shear transfer
- F_y = specified minimum yield stress of steel
- F_u = specified minimum tensile strength of steel

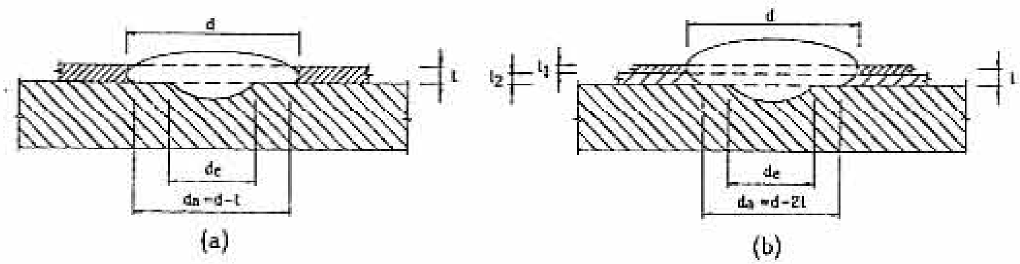


Figure 13.8 Definition of d, d_a and d_e in Arc Spot Weld
a- Single Sheet b- Double Sheet

	or either :	
$P_n = \frac{\pi d_e^2}{4} . F_u$ 13.49	For $F_u < 4$ $P_n = [6.5 - 1.5 F_y] t d_a F_u \le 1.4 t d_a F_u$ 13.50	For $F_u \ge 4$ $P_n = 0.68 t d_a F_u$ $\phi = 0.55$ 13.51

g- The uplift nominal tensile strength, ϕP_n of each concentrically loaded arc spot weld connecting sheets and supporting member , shall be computed as the smaller of either:

$\phi = 0.50.$

13.15.1.1.3 Arc Seam Welds

For arc seam welds, the nominal shear strength ϕP_n on each arc seam weld shall be taken as the smaller of the values computed by the following Equations:

The Nominal strength for Arc Seam Welds	
Nominal shear strength based on shear capacity of weld $P_n = 0.72 F_u (\pi d_e^2 / 4 + L d_e)$ 13.52	Nominal shear strength based on strength of connected sheets $P_n = 2.4 F_u (0.25 L + 0.9 d_a) t$ 13.53

Where:

d = width of arc seam weld
L = length of seam weld not including circular ends, $(L < 3d)$
ϕ = 0.55
d_e, d_a and F_u are as defined in arc spot welds

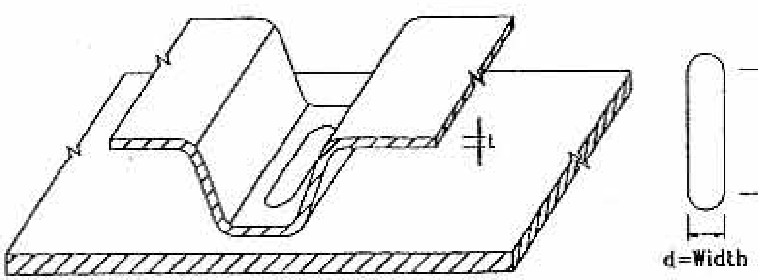


Figure 13.9 Arc Seam Welds

The requirements for minimum edge distance are the same as those for arc spot welds.

13.15.1.1.4 Fillet Welds

The Nominal strength for a fillet weld in lap and T-joints shall not exceed the values computed by Equation 13.54 for the shear strength of the fillet weld and by Equations 13.55, 13.56 and 13.57 for the strength of the connected sheets as follows:

<table>
<tr><th colspan="3">The nominal strength for a Fillet weld in lap and T-joints</th></tr>
<tr><td rowspan="5">Nominal shear strength based on shear capacity of weld

$P_n = 0.72\, F_u\, (s\, L)$
$\phi = 0.55$

............13.54</td><td colspan="2">Nominal shear strength based on strength of connected sheets</td></tr>
<tr><td colspan="2">a- Longitudinal loading</td></tr>
<tr><td>1- $L/t < 25$
$P_n = F_u\, (1 - 0.01\, L/t)\, (t\, L)$
$\phi = 0.55$
............13.55</td><td>2- $L/t \geq 25$
$P_n = 0.72\, F_u\, (t\, L)$
$\phi = 0.50$
............13.56</td></tr>
<tr><td colspan="2">b- Transverse loading
$P_n = F_u\, (t\, L)$
$\phi = 0.55$
............13.57</td></tr>
</table>

Where:

L	=	length of fillet weld
$s = s_1$ or s_2	=	leg sizes of fillet welds, whichever is smaller

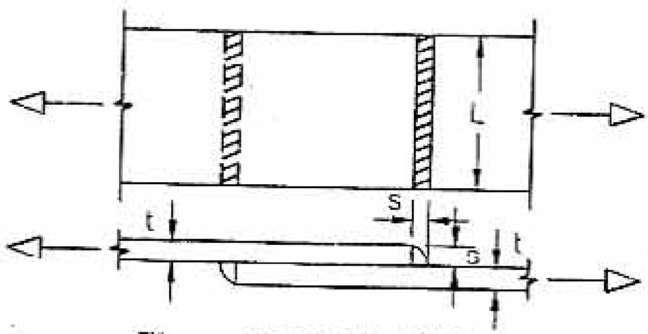


Figure 13.10 Fillet Welds

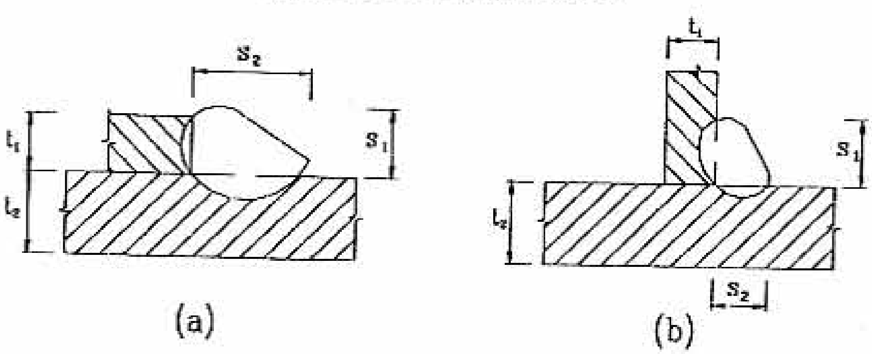


Figure 13.11 Leg Sizes of Fillet Welds
a- Lap Joint   b- T-Joint

13.15.1.1.5 Flare Groove Welds

The nominal shear strength for each flare groove weld shall be determined as follows:

<table>
<tr><th colspan="3">The nominal Shear strength for flare groove welds</th></tr>
<tr><td rowspan="4">Nominal shear strength based on shear capacity of weld
$P_n = 0.72\ F_u\ (s\ L)$
$\phi = 0.55$
............13.58</td><td colspan="2">Design load based on strength of connected sheets</td></tr>
<tr><td colspan="2">a- Longitudinal loading</td></tr>
<tr><td>For $t \leq t_w < 2t$ or if the lip height is less than the weld length
$P_n = 0.72\ F_u\ (t\ L)$
$\phi = 0.50$
............13.59</td><td>For $t_w \geq 2t$ and the lip height is equal to or greater than L
$P_n = 1.4\ F_u\ (t\ L)$
$\phi = 0.50$
............13.60</td></tr>
<tr><td colspan="2">b- Transverse loading
$P_n = 0.8\ F_u\ (t\ L)$
$\phi = 0.50$
............13.61</td></tr>
</table>

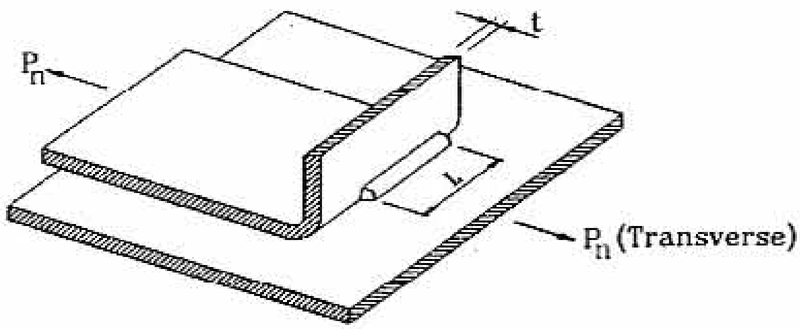


Figure 13.12a Transverse Flare Bevel Weld

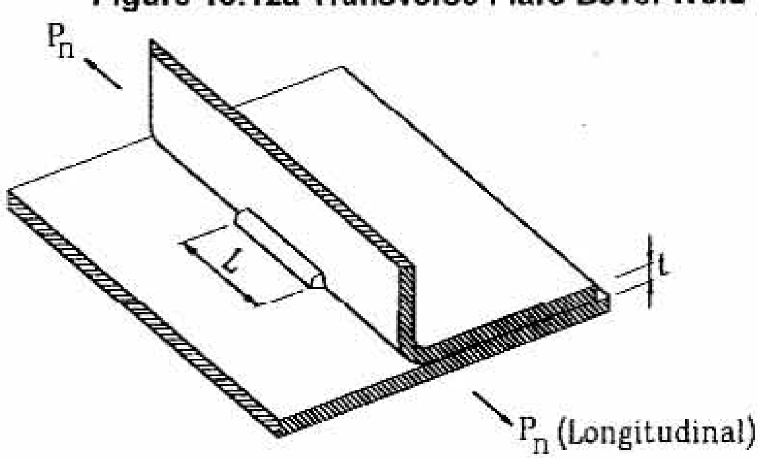


Figure 13.12b Longitudinal Flare Bevel Weld

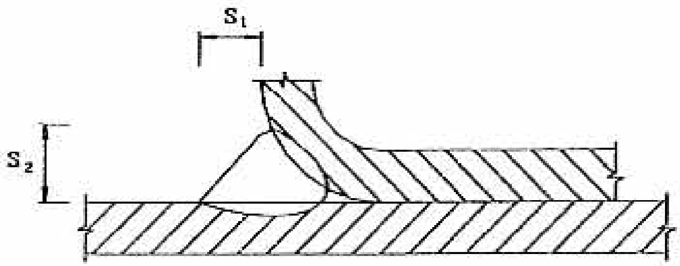


Figure 13.13 Effective Size Dimension t_w for Flare Groove Welds (Smaller of s_1 and s_2)

The definition of t_w in such case is as shown in Fig. 13.13.

13.15.1.2 Resistance Welds

The nominal shear strength of spot resistance welding ϕP_n shall be determined as given in the following Table:

Thickness of Thinnest Outside Sheet	Nominal Shear Strength per Spot P_n	Thickness of Thinnest Outside Sheet	Nominal Shear Strength per Spot P_n	Thickness of Thinnest Outside Sheet	Nominal Shear Strength per Spot P_n
(mm)	(kg)	(mm)	(kg)	(mm)	(kg)
0.50	200	1.75	1200	3.10	3150
0.75	430	2.00	1400	4.80	4450
1.00	620	2.25	1700	6.40	6550
1.25	720	2.50	2150		
1.50	990	2.75	2650		

Where $\phi = 0.60$

13.15.2 Bolted Connections

The following design criteria govern bolted connections used for cold-formed steel structural members in which the thickness of the thinnest connected part is 4 mm or less. For bolted connections where the thickness of the thinnest connected part is greater than 4 mm, the provisions of Chapter 8 shall apply.

13.15.2.1 Minimum Spacing and Edge Distance in Line of Stress

The distance (e) measured in the line of force from the center of a standard hole to the nearest edge of an adjacent hole or to the end of the connected part toward which the force is directed should not be less than the value of e_{min} determined by:

$$e_{min} = \frac{\alpha d}{0.4} \qquad 13.62$$

Where:

α = bearing stress coefficient as given in Table 8.2
d = bolt diameter

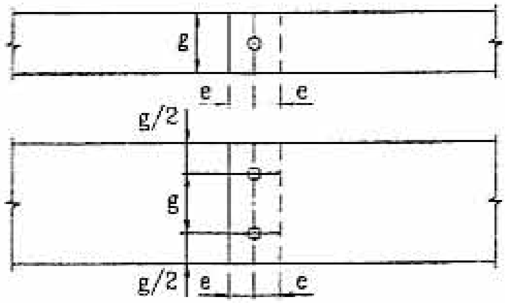


Figure 13.14 Spacing and Edge Distance of Bolts

The nominal clearance in standard holes shall be as previously outlined in section 8.3.2

In addition to the previous requirement, the following requirements concerning minimum spacing and edge distance in the line of stress shall also be considered:

a- The minimum distance between centers of bolt holes shall not be less than *3d*.
b- The distance from the center of any standard hole to the end or other boundary of the connecting member shall not be less than *1.5d*.
c- The clear distance between edges of two adjacent holes shall not be less than *2d*.
d- The distance between the edge of the hole and the end of the member shall not be less than *d*.
e- For slotted holes, the distance between edges of two adjacent holes and the distance measured from the edge of the hole to the end or other boundary of the connecting member in the line of stress shall not be less than the value of $(e_{min} - 0.5d_h)$, in which e_{min} is the required distance computed from the above Equation and d_h is the diameter of a standard hole.

13.15.2.2 Design Tensile Resistance of Net Section of Connected Parts

The Nominal tensile strength ϕP_n of the net section of a bolted connection shall be as follows:

a- With washers under both bolt head and nut

$$P_n = (1.0 - 0.9\,r + 3\,r\,d/g)\,F_u \cdot A_n < F_y\,A_n \qquad 13.63$$

$\phi = 0.60$ for double shear and $\phi = 0.50$ for single shear

b- Without washers under both bolt head and nut, or with only one washer

$$P_n = (1.0 - r + 2.5\, r\, d / g)\, F_u \,.\, A_n \; < F_y A_n \quad \text{.......................} \quad 13.64$$

$\phi = 0.60$

Where:

r = force transmitted by bolt or bolts at the section considered, divided by tension force in member at that section. If r is less than 0.2 it may be taken as zero

g = spacing of bolts perpendicular to the line of stress. In the case of a single bolt, g = gross width of sheet

P_n = nominal tensile resistance of the net section

13.15.2.3 Design Bearing Resistance between Bolts and Connected Parts

The design bearing resistance between bolts and the parts connected to them is taken as detailed in Section 8.5.3.

13.15.2.4 Design Shear Resistance of Bolts

The design shear resistance on the gross sectional area of bolts is taken as detailed in Section 8.5.2.

13.15.2.5 Design Tensile Resistance of Bolts

The design tensile resistance on the net sectional area of bolts is taken as detailed in Section 8.5.4.

13.15.3 Screws

The following requirements shall apply to self-tapping screws with $2\ mm \leq d \leq 6\ mm$. The screws shall be thread-forming or thread-cutting, with or without a self-drilling point.

Screws shall be installed and tightened in accordance with the manufacturer's recommendations.

The nominal tensile strength on the net section of each member joined by a screw connection shall not exceed the member nominal tensile strength from Chapter 3 or the connection nominal tensile strength from this section.

13.15.3.1 Minimum Spacing

The distance between the centers of fasteners shall not be less than $3d$.

13.15.3.2 Minimum Edge and End Distance

The distance from the center of a fastener to the edge of any part shall not be less than $3d$. If the connection is subjected to shear force in one direction only, the minimum edge distance shall be $1.5d$ in the direction perpendicular to the force.

13.15.3.3 Shear

13.15.3.3.1 Connection Shear

The nominal shear strength per screw, P_{ns}, shall be [redacted] follows:

For $t_2/t_1 \leq 1.0$, P_{ns} shall be taken as the smallest of:

$$P_{ns} = 4\,(t_2^3 d)^{1/2} F_{u2}$$
$$P_{ns} = 2.4\, t_1\, d\, F_{u1}$$
$$P_{ns} = 2.4\, t_2\, d\, F_{u2} \quad \ldots\ldots\ldots \quad 13.65$$

For $t_2/t_1 \geq 2.5$, P_{ns} shall be taken as the smallest of:

$$P_{ns} = 2.4\, t_1\, d\, F_{u1}$$
$$P_{ns} = 2.4\, t_2\, d\, F_{u2} \quad \ldots\ldots\ldots \quad 13.66$$

For $1.0 < t_2/t_1 < 2.5$, P_{ns} shall be determined by linear interpolation between the above two cases.

Where:

d = screw diameter, cm
P_{ns} = nominal shear strength per screw, ton
t_1 = thickness of member in contact with the screw head, cm
t_2 = thickness of member not in contact with the screw head, cm
F_{u1} = tensile strength of member in contact with the screw head, t/cm^2
F_{u2} = tensile strength of member not in contact with the screw head, t/cm^2

13.15.3.3.2 Shear in Screws

The nominal shear strength of the screw shall be provided by the screw manufacturer.

13.15.3.4 Tension

For screws which carry tension, the head of the screw or washer, if a washer is provided, shall have a diameter d_w not less than 8 mm. Washers shall be at least 1.2 mm thick.

13.15.3.4.1 Pull-Out

The nominal pull-out strength, P_{not}, shall be calculated as follows:

$$P_{not} = 0.84\, t_c\, d\, F_{u2} \quad \ldots\ldots\ldots \quad 13.67$$

Where t_c is the lesser of the depth of penetration and the thickness, t_2.

13.15.3.4.2 Pull-Over

The nominal pull-over strength, $P_{nov,}$ shall be calculated as follows:

$$P_{nov} = 1.4\ t_1\ d_w\ F_{u1} \qquad 13.68$$

Where d_w is the larger of the screw head diameter or the washer diameter, and shall be taken not larger than 12 mm.

13.15.3.4.3 Tension in Screws

The nominal tensile strength, $P_{nt,}$ per screw shall be determined by approved tests. The nominal tensile strength of the screw shall not be less than 1.25 times the lesser of P_{not} and $P_{nov.}$

13.15.4 Built-Up Sections

13.15.4.1 I-Sections Composed of Two Channels

The maximum longitudinal spacing of connectors shall be limited to the following values:

a- For compression members

$$s_{max} = L\ r_{cy}\ /\ (\ 2\ r_I\ ) \qquad 13.69$$

Where:

s_{max}	=	maximum permissible longitudinal spacing of connectors, cm
L	=	unbraced length of compression member, cm
r_I	=	radius of gyration of the I-section about the axis perpendicular to the direction in which buckling would occur for the given conditions of end support and intermediate bracing, if any, cm
r_{cy}	=	radius of gyration of one channel about the centroidal axis parallel to web, cm

b- For beams

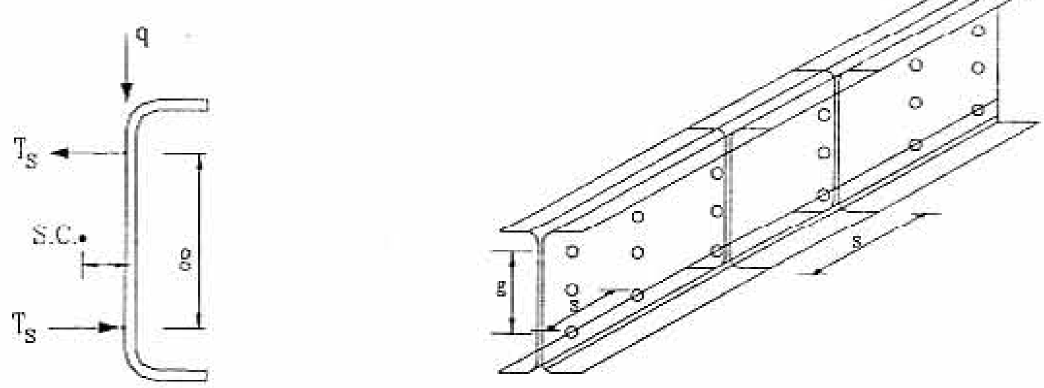


Figure 13.15 Forces on a Channel of a Built-Up Member

$$s_{max} = L / 6 \qquad 13.70$$

$$s_{max} \le \frac{2gT_s}{m.q} \qquad 13.71$$

Where:

L = span of beam, cm
g = vertical distance between the two rows of connectors nearest to the top and bottom flanges, cm
T_s = tensile strength of connectors, t
q = intensity of load, t/cm
m = distance between shear center of one channel and mid plane of its web, cm

For simple channels without stiffening lips at the outer edges:

$$m = \frac{b_f^2}{2b_f + d/3} \qquad 13.72$$

For C-shaped channels with stiffening lips at the outer edges:

$$m = \frac{b_f d.t}{4.I_x}\left[b_f d + 2D\left(d - \frac{4D^2}{3d}\right)\right] \qquad 13.73$$

Where:

b_f = projection of flanges from inside face of web, cm
d = depth of channels, cm
t = thickness of channel section, cm
D = overall depth of stiffening lip, cm
I_x = moment of inertia of one channel about its centroidal axis normal to web, cm^4

If the length of bearing of a concentrated load or reaction is smaller than the spacing of the connectors, the required strength of connectors closest to the load or reaction P is:

$$T_s = P \,.\, m / (2 \,.\, g) \qquad 13.74$$

13.15.4.2 Spacing of Connectors in Compression Elements

The spacing s, in the line of stress of welds, bolts or rivets connecting the compression cover plate or sheet to another element should not exceed:

a- That which is required to transmit the shear between the connected parts on the basis of the design strength per connector, nor
b- $s = 50\, t / \sqrt{f}$, where s is the spacing, t is the thickness of the cover plate or sheet, and f is the design stress in the cover plate or sheet, nor

c- Three times the flat width $\bar{b}$ of the narrowest unstiffened compression element related to the connection.

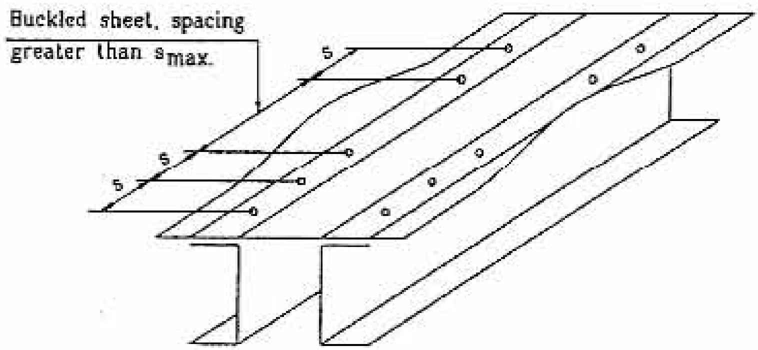


Figure 13.16 Spacing of Connectors in Compression Elements

CHAPTER 14

SERVICEABILITY DESIGN CONSIDERATIONS

This chapter is intended to provide design guidance for serviceability limit states design considerations. The general design requirement for serviceability is given in Section 1.5.4.

14.1 BASIS OF SERVICEABILITY LIMIT STATES

a- Serviceability limit states are the states in which the function of a building, its appearance, maintainability, durability, and comfort of its occupants are preserved under normal usage. Serviceability limit states for steelwork are:
   i- Deformations or deflections which adversely affect the appearance or effective use of the structure (including the proper functioning of machines or services).
   ii- Vibration, oscillation or sway which causes discomfort to the occupants of a building or damage to its contents.
   iii- Deformations, deflections, vibration, oscillation or sway which causes damage to finishes or non-structural elements.
b- Limiting values of structural behavior to ensure serviceability (e.g., maximum deflections, accelerations, etc.) shall be chosen with due regard to the intended function of the structure. Where necessary, serviceability shall be checked using realistic loads for the appropriate serviceability limit state.
c- Except when specific limiting values are agreed between the client, the designer and the competent authority, the limiting values given in this Chapter should be applied.
d- When plastic global analysis is used for the ultimate limit state, the possibility that plastic redistribution of forces and moments would also occur at the serviceability limit state should be investigated. This should be permitted only where it can be shown that it will not be repeated. It should also be taken into account in calculating the deformations.
e- Where preloaded bolts are used in Category (B) slip-critical connections (slip-resistance at the serviceability limit state) shall be satisfied, i.e. the design serviceability shear load should not exceed the design slip resistance, obtained from 8.2 2.

14. 2 DEFLECTIONS, VIBRATION, AND DRIFT

14.2.1 Deflections

Deformations in structural members and structural systems due to service loads shall not impair the serviceability of the structure.

14.2.1.1 Requirements

a- Steel structures and components shall be so proportioned that deflections are within the limits agreed between the client, the designer and the competent authority as being appropriate to the intended use and occupancy of the building and the nature of the materials to be supported.

b- Recommended limits for deflections are given in 14.2.1.2. In some cases more stringent limits (or exceptionally, less stringent limits) will be appropriate to suit the use of the building or the characteristics of the cladding materials or to ensure the proper operation of lifts etc.

c- The values given in 14.2.1.2 are empirical values. They are intended for comparison with the results of calculations and should not be interpreted as performance criteria.

d- The design provisions given in 1.5.4 for the rare combination should be used in connection with all limiting values given in section 14.2.1.2.

e- The deflections should be calculated making due allowance for any second order effects and the possible occurrence of any plastic deformations at the serviceability limit state.

14.2.1.2 Limiting values

a- The limiting values for vertical deflections δ given below are illustrated by reference to the simply supported beam provided with camber as shown in Fig. 14.1, in which:

$$\delta = \delta_1 + \delta_2 - \delta_0 \leq \delta_{max} \qquad 14.1$$

where δ_{max} is the sagging in the final state relative to the straight line joining the supports

- δ_0 is the camber (hogging) of the beam in the unloaded state
- δ_1 is the variation of the deflection of the beam due to the permanent load immediately after loading, and
- δ_2 is the variation of the deflection of the beam due to the variable loading plus any time dependent deformations due to the permanent load

b- The limiting values for vertical deflections δ, (due to the variable loading plus any time dependent deformations due to the permanent load) , for a beam without camber is given by:

$$\delta \leq \delta_{2\,max} \qquad 14.2$$

c- For buildings, the recommended limits for vertical deflections are given in Table 14.1, in which L is the span of the beam. For cantilever beams, the length L to be considered is twice the projecting length of the cantilever.

d- For crane gantry girders and runway beams, the horizontal and vertical deflections should be limited according to the use and class of the equipment.

Table 14.1 Limiting Values for Vertical Deflection in Buildings

Member	$\delta_{2\ max}$	δ_{max}
Beams and trusses in buildings carrying plaster or other brittle finish	$L/300$	$L/250$
Floors supporting columns	$L/500$	$L/400$
All other beams	$L/200$	$L/160$
Cantilevers	$L/180$	$L/140$
Crane track girders	$L/800$	$L/650$
Where δ_{max} can impair the appearance of the building	—	$L/250$

Where L is the span or twice the projecting length of the cantilever.

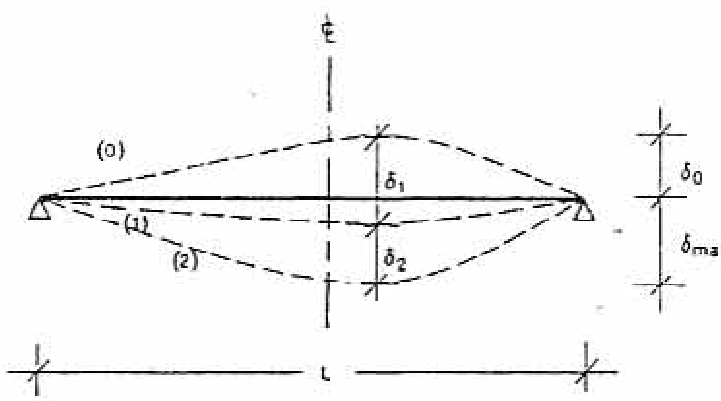


Figure 14.1 Vertical Deflection to be Considered

14.2.1.3 Camber

a- If any special camber requirements are necessary to bring a loaded member into proper relation with the work of other trades, as for the attachment of runs of sash, the requirements shall be set forth in the design documents.
b- Structural buildings may also be provided with an erection camber, as indicated in the project documents or the plans.
c- Camber may be required to maintain clearance under all conditions of loading, or it may be required on account of appearance. Camber may also result from prestressing.
d- If camber involves the erection of any member under a preload, this shall be noted in the design documents,and camber diagrams shall be shown on the erection drawings.
e- Beams and trusses detailed without specified camber shall be fabricated so that after erection any camber due to rolling or shop assembly shall be upward.

f- Simply supported main girders more than 14 m in length of truss or plate girder construction shall be provided with such a camber in accordance with Equation 14.1.

g- Rolled beams and plate girders 14 m or less in length, may not be cambered.

14.2.2 Vibration

14.2.2.1 Floor Vibration

Vibration shall be considered in designing beams and girders supporting large areas free of partitions or other sources of damping where excessive vibration due to pedestrian traffic or other sources within the building is not acceptable.

14.2.2.2 Requirements

a- Suitable provisions shall be made in the design for the effects of imposed loads which can induce impact, vibration, etc.

b- The dynamic effects to be considered at the serviceability limit state are vibration caused by machines and oscillation caused by harmonic resonance.

c- The natural frequencies of structures or parts of structures should be sufficiently different from those of the excitation source to avoid resonance.

d- The design provisions given in 1.5.4 for the frequent combination should be used in connection with all limiting values given in section 14.2.1.2.

14.2.2.3 Structures Open to the Public

a- The oscillation and vibration of structures on which the public can walk shall be limited to avoid significant discomfort to users.

b- In the case of floors over which people walk regularly, such as the floors of dwellings, offices and the like, the lowest natural frequency of the floor construction should not be lower than 3 cycles/second. This condition will be satisfied if the instantaneous total deflection $\delta_1 + \delta_2$ (as defined in 14.2.1.2 but calculated using the frequent combination) is less than 28mm. These limits may be relaxed where justified by high damping values.

c- In the case of a floor which is jumped or danced in a rhythmical manner, such as the floor of a gymnasium or dance hall, the lowest natural frequency of that floor should not be less than 5 cycles/second. This condition will be satisfied if the deflection calculated as above is not greater than 10mm.

d- If necessary, a dynamic analysis may be carried out to show that the accelerations and frequencies which would be produced would not be such as to cause significant discomfort to users or damage to equipment.

14.2.2.4 Wind - Excited Oscillations

a- Unusually flexible structures, such as very slender tall buildings or very large roofs, and unusually flexible elements, such as light tie rods, shall be investigated under dynamic wind loads both for vibrations in plane and also for vibrations normal to the wind direction.

b- Such structures should be examined for:
   i- gust induced vibrations
   ii- vortex induced vibrations.

14.2.3 Drift

Lateral deflection or drift of structures due to code-specified worst combinations of unfactored loads shall not cause collision with adjacent structures nor exceed the limiting values of such drifts which may be specified or appropriate. For buildings the permitted maximum limits for horizontal deflections at the tops of the columns are given in Table 14.2.

Table 14.2 Drift (Horizontal Deflection) in Buildings

Member	Max. Drift
Horizontal deflection at tops of columns in single-storey buildings other than portal frames	Height / 300
Horizontal deflection in each storey of a building with more than one storey	Height of storey under consideration / 300
Horizontal deflection at the top of a building with more than one storey	Total height of building / 500
Horizontal deflection at tops of columns in portal frames without overhead cranes	Height / 140
Horizontal deflection at tops of columns in portal frames with overhead cranes	To be decided according to the recommendations of the overhead crane manufacturer, but should not exceed the height / 140

14.3 PONDING

a- To ensure the correct discharge of rainwater from a flat or nearly flat roof, the design of all roofs with a slope of less than 5% should be checked to ensure that rainwater cannot collect in pools. In this check, due allowance should be made for possible construction inaccuracies and settlements of foundations, deflections of roofing materials, deflections of structural members and the effects of precamber. This also applies to floors of car parks and other open sided structures.

b- The roof system shall be investigated by structural analysis to assure adequate strength and stability under ponding conditions, unless the roof surface is provided with sufficient slope toward points of free drainage or adequate individual drains to prevent the accumulation of rainwater.

The roof system shall be considered stable and no further investigation is needed if:

$$C_p + 0.9C_s \leq 0.25 \quad \text{......} \quad 14.3$$

$$I_d \geq 3940\, S^4 \quad \text{......} \quad 14.4$$

Where:

$$C_p = 504\, L_s\, L_p^4 / I_p$$

and $$C_s = 504\, S\, L_s^4 / I_s$$

L_p = column spacing in direction of girder (length of primary members), m
L_s = column spacing perpendicular to direction of girder (length of secondary members), m
S = spacing of secondary members, m
I_p = moment of inertia of primary members, mm^4
I_s = moment of inertia of secondary members, mm^4
I_d = moment of inertia of the steel deck supported on secondary members, mm^4 per m

For trusses and steel joists, the moment of inertia I_s shall be decreased 14 percent when used in the above equation. A steel deck shall be considered a secondary member when it is directly supported by the primary members.

c- Precambering of beams may reduce the likelihood of rainwater collecting in pools, provided that rainwater outlets are appropriately located.

d- Where the roof slope is less than 3% additional calculations should be made to check that collapse cannot occur due to the weight of water collected in pools which may be formed due to the deflection of structural members or roofing material.

14.4 EXPANSION AND CONTRACTION

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

14.5 CONNECTION SLIP

For the design of slip-critical connections see Section 8.2.2.

14.6 CORROSION

When appropriate structural components shall be designed to tolerate corrosion or shall be protected against corrosion that may impair the strength or serviceability of the structure.

CHAPTER 15

DIMENSIONAL TOLERANCES

15.1 GENERAL

Steel structures consist of prefabricated elements which are assembled together in the erection stage. In order to ensure the real safety of the structure in comparison to the theoretical assumption concerning the geometry of the load application, the dimensional tolerance specified herein shall be observed.

15.2 TYPES OF TOLERANCES

15.2.1 Normal Tolerances

Normal tolerances are the basic limits for dimensional deviations necessary:

a- To satisfy the design assumptions for statically loaded structures.

b- To define acceptable tolerances for building structures in the absence of any other requirements.

15.2.2 Special Tolerances

Special tolerances are more stringent tolerances necessary to satisfy the design assumptions:

a- For structures other than normal building structures.

b- For structures in which fatigue predominates.

15.2.3 Particular Tolerances

Particular tolerances are more stringent tolerances necessary to satisfy functional requirements of particular structures or structural components, related to:

a- Attachment of other structural or non-structural components.

b- Shafts for lifts (elevators).

c- Tracks for overhead cranes.

d- Other criteria such as clearances.

e- Alignment of external face of building.

15.3 APPLICATION OF TOLERANCES

a- All tolerance values specified in the following shall be treated as normal tolerances.

b- Normal tolerances apply to conventional single-storey and multi-storey steel framed structures of residential, administrative, commercial and industrial buildings except where special or particular tolerances are specified.

c- Any special or particular tolerances required shall be detailed in the project specification.

d- Any special or particular tolerances required shall also be indicated on the relevant drawings.

15.4 NORMAL ERECTION TOLERANCES

a- The following normal tolerance limits relate to the steel structure in the unloaded state, i.e., structure loaded only by its own weight, see the following Tables and Figures.

b- Each criterion given in the tables shall be considered as a separate requirement, to be satisfied independently of any other tolerances criteria.

c- The fabrication and erection tolerances specified in 15.5 to 15.7 apply to the following reference points:

i- For a column, the actual center point of the column at each floor level and at the base, excluding any base plates.

ii- For a beam, the actual center point of the top surface at each end of the beam excluding any endplate.

d- All elements should be checked after fabrication and before erection for the allowable tolerances according to 15.5.

15.5 PERMISSIBLE DEVIATIONS OF FABRICATED ELEMENTS

Deviation		e_{max}	Fig.
Deflection of column between points which will be laterally restrained on completion of erection	f_{h1}	$\pm 0.001\ h_{11}$ generally $\pm 0.002\ h_{11}$ for members with hollow cross section h_{11} is the height between points which will be laterally restrained	15.1
Deflection of column between floor slabs	f_h	$\pm 0.001\ h_1$ generally $\pm 0.002\ h_1$ for members with hollow cross section h_1 is the height between floor slabs	15.1
Lateral deflection of compression flange of girder, relative to the weak axis, between points which will be laterally restrained on completion of erection	f_{11}	$\pm 0.001\ l_{b1}$ generally $\pm 0.002\ l_{b1}$ for members with hollow cross section l_{b1} is the length between points which will be laterally restrained	15.2
Lateral deflection of girder.	f_1	$\pm 0.001\ l_b$ generally $\pm 0.002\ l_b$ for members with hollow cross section l_b is the total length of girder	15.2
Girders and columns (depth of web h_w, width of flange b): maximum bow of web	f_w	$h_w/150$	15.3
Inclination of web between upper and lower flanges	V_w	$h_w/75$	15.3
Eccentricity of the web in relation to the center of either flange	V_{w1}	$b/40 \leq 10$ mm	15.3
Positional deviation of parts connected to a girder or column e.g. cover plate, base plate etc..	e_1	7 mm. in any direction.	15.4
Positional deviation of adjacent end plates of girders	e_1	5 mm. in any direction	15.4

Length of prefabricated components to be fitted between other components	Δl_1 Δh_c	+ 0.0 - 5 mm.	15.5
Plate girders with intermediate stiffeners (depth of web h_w, thickness of web t_w): maximum bow of web	f_w	Least panel dimension /115 For $h_w / t_w < 150$ Least panel dimension /90 For $h_w / t_w \geq 150$	15.3 and 15.6
Flanges of plate girders	Δ	$\Delta \leq c/250 \leq 6\ mm$	15.7
Unevenness of plates in the case of contact bearing surfaces		1 mm. over a gauge length of 300 mm	

15.6 PERMISSIBLE DEVIATION OF COLUMN FOUNDATIONS

a- The deviation of the center line for anchor bolts within the group of bolts at any column base shall not exceed the following:

i- For bolts rigidly cast in, between centers of bolts: a_1 = 10 mm. in any direction.

ii- For bolts set in sleeves, between centers of sleeves: a_1 = 20 mm. in any direction.

b- The distance between two adjacent columns, measured at the base of the steel structure, shall not exceed the value $a_2 = \pm$ 10 mms of the nominal distance (Fig. 8).

c- With column rows, the sum of single deviations a_2, referred to the length of the row *l* shall not exceed the value (Fig. 8):

$/a_3/ \leq 15\ mms$. For $L \leq 30\ m$.
$/a_3/ \leq 15 + 0.25\ (L - 30)\ mms$. For $L > 30\ m$. (maximum 50 mm).

15.7 PERMISSIBLE DEVIATIONS OF ERECTED STRUCTURES

Deviation		a_{max}	Fig.
Overall dimensions of the building	$\sum\Delta h$ or $\sum\Delta L$	For $L \leq 30\ m$: $\pm 20\ mm$. For $L > 30\ m$: $\pm 20 + 0.25\ (L - 30)\ mm$. (maximum 50 mm)	15.9
Level of top of floor slab Floor bearing on column	Δh	±5 mm.	15.5
Inclination of column in a multi-storey building maximum deviation for the vertical line between adjacent floor slabs	v_h	0.003 h_1 h_1 = Floor height under consideration	15.10
Inclination of column in a multi-storey building; maximum deviation for the vertical line through the intended location of the column base	v_1	$0.0035(\sum h_1)3/(n+2)$ n = Number of floors	15.11

Inclination of column in a single-storey residential building, maximum deviation for the vertical line.	v_{h1}	0.0035 h h = Single storey floor height	15.12
Inclination of column of a portal frame in an industrial building, (not supporting crane gantry), maximum deviation for the vertical line.	v_{hp}	Individual = v_{h1} or $v_{h2} \leq 0.010\,h$ $Mean = \dfrac{\left(v_{h1} + v_{h2}\right)}{2} \leq 0.002\,h$	15.13 OR 15.14
Unintentional eccentricity of girder bearing	ℓ_o	5 mm.	15.15
Distance between adjacent steel columns at any level	$\Delta \ell_s$	± 15 mm.	15.9
Distance between adjacent steel girders at any level	$\Delta \ell_f$	± 20 mm.	15.5
Positional deviation of a column base in relation to the column axis through the head of the column below(applied also in the case of indirect load transmission)	e_2	5 mm. in any direction	15.16
Deviation in level of bearing surfaces on steel columns (crane track girder level)	Δh_c	+0.0 mm. -10 mm.	15.17
Positional deviation of bearing surfaces.	e_3	± 5 mm.	15.18

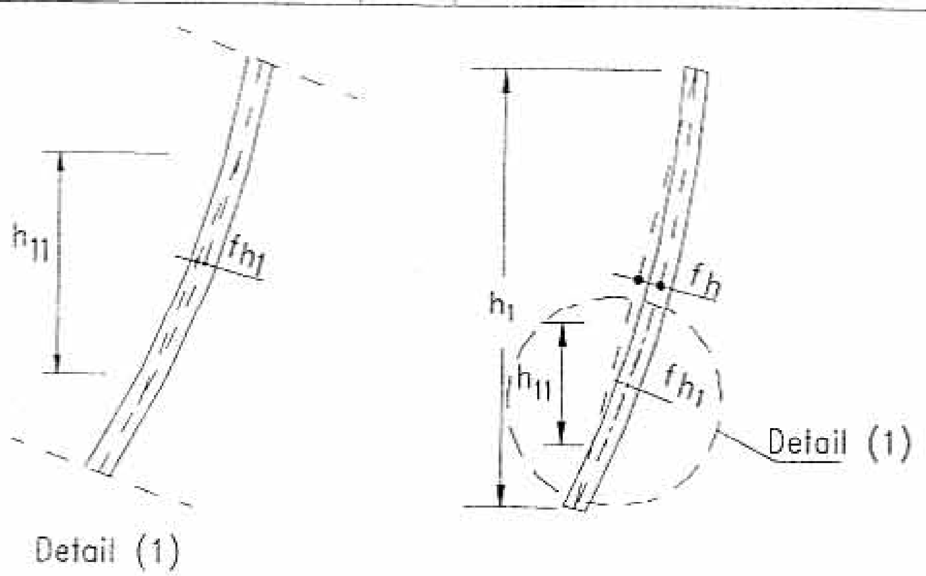


Figure 15.1 Inclination and Deflections of Columns

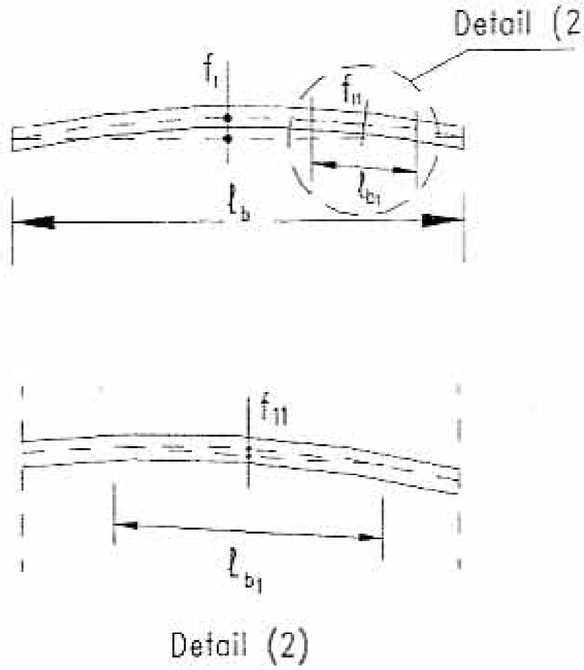


Figure 15.2 Lateral Deflection of Girders

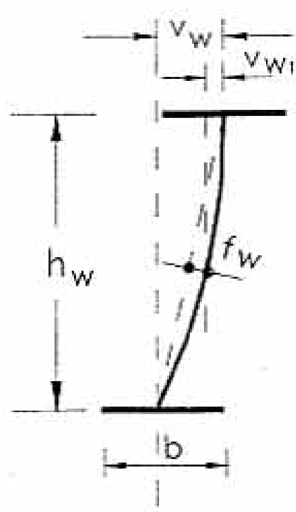


Figure 15.3 Deviations in Welded Girders

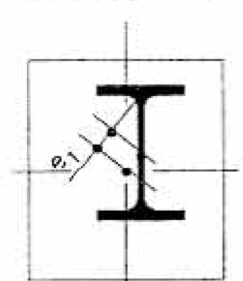
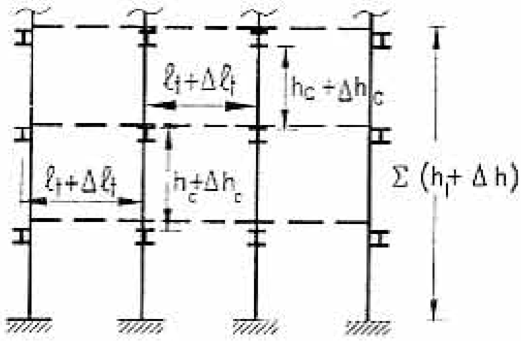


Figure 15.4 Deviation in Connecting Pieces



h_l Level of Top of Floor Slab Floor Bearing on Column

Δh Deviation From h_l

h_c Column Length With Intermediate Components

Δh_c Deviation From h_c

ℓ_t Distance Between Adjacent Girders

$\Delta\ell_t$ Deviation From ℓ_t

Figure 15.5 Deviation in Height and Length

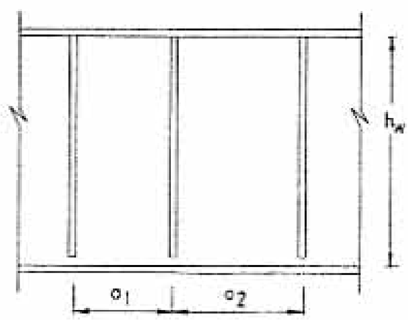


Figure 15.6 Welded Plate Girder

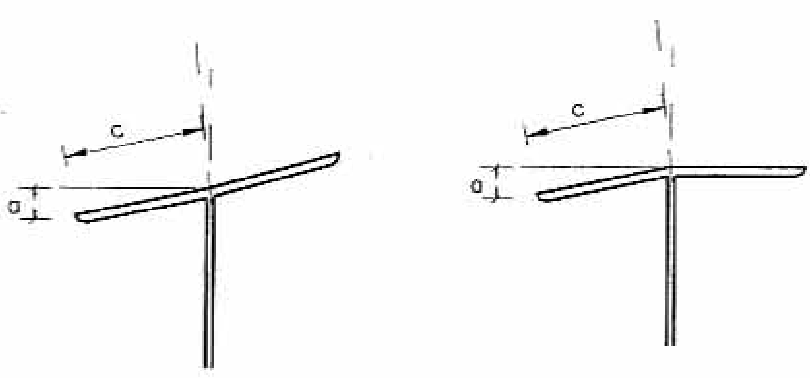


Figure 15.7 Deviation in Flanges of Welded Plate Girders

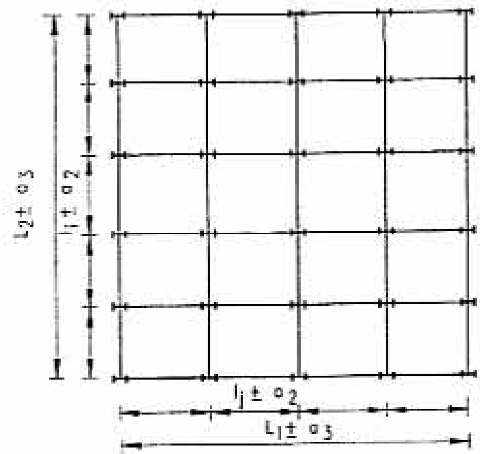


Figure 15.8 Deviation in Length at Base of Steel Structure (Plan)

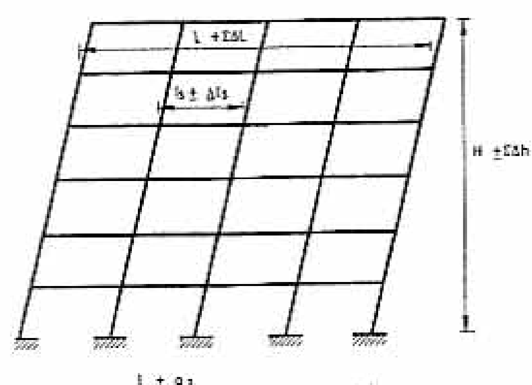


Figure 15.9 Deviation in Length (Vertical Section)

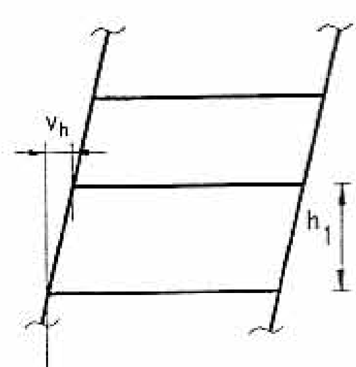


Figure 15.10 Inclination of Column in a Multi – Storey Building

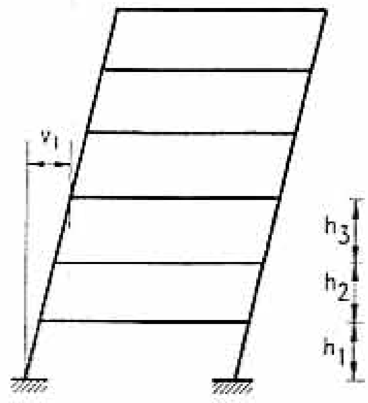


Figure 15.11 Inclination of Column in a Multi – Storey Building

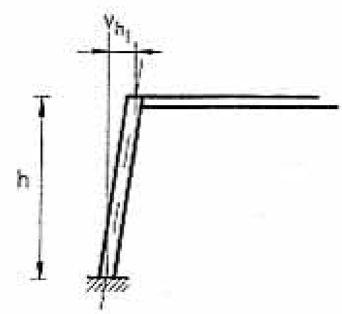


Figure 15.12 Inclination of Column in a Single – Storey Building

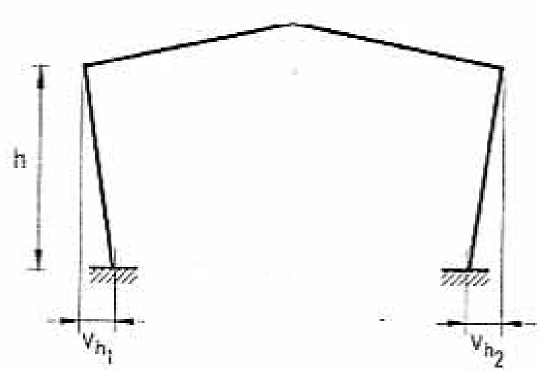


Figure 15.13 Inclination of Column in a Portal Frame

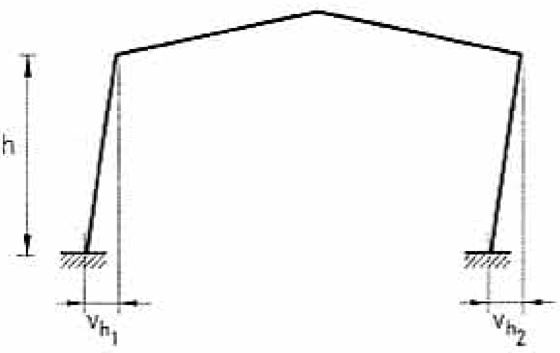


Figure 15.14 Inclination of Column in a Portal Frame

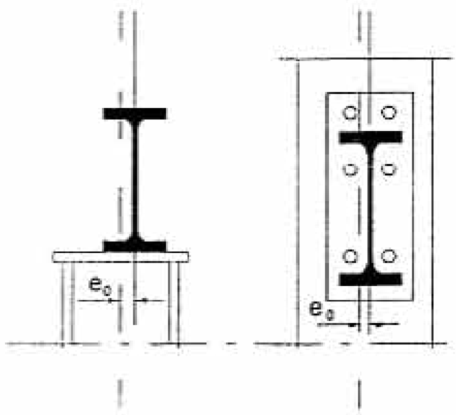


Figure 15.15 Eccentricity of Girder Bearing

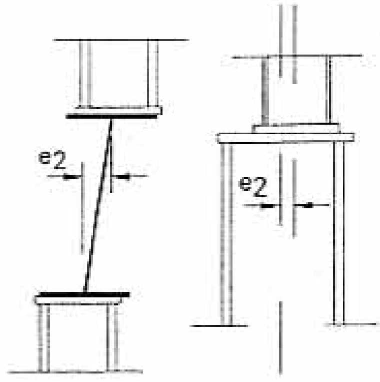


Figure 15.16 Deviations in Columns Splices

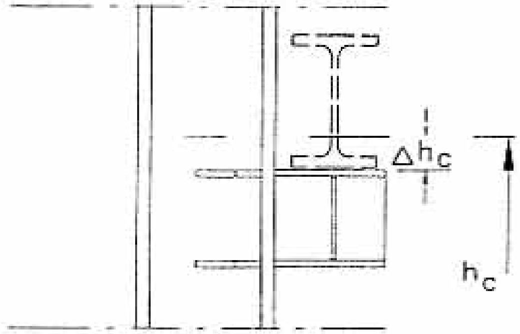


Figure 15.17 Deviation in Level of Bearing Surface

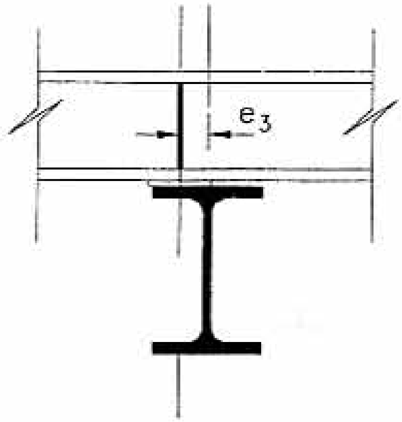


Figure 15.18 Deviation in Position of Bearing Surface

CHAPTER 16

FABRICATION, ERECTION AND FINISHING WORKS

16.1 GENERAL PROVISIONS

16.1.1 Scope

Unless otherwise specified in the Contract Documents, the trade practices that are defined in this Code shall govern the fabrication and erection of steel structures (temporary and permenant).

16.1.2 Responsibility for Design

16.1.2.1 When the Employer's Designated Representative for Design (hereinafter called EDRD) provides the design, design drawings and specifications, the Fabricator and/or the Erector shall be responsible for checking suitability, adequacy and building-code conformance of the design. The Fabricator and/or the Erector shall give prompt notice to the Employer and EDRD of any error, omission, fault or other defects in the design, design drawings or specifications.

16.1.2.2 When the Employer enters into a direct contract with the Fabricator to both design and fabricate an entire completed steel structure, the Fabricator shall be solely responsible for the suitability, adequacy and building-code conformance of the structural steel design. The Employer shall be responsible for the suitability, adequacy and building-code conformance of the non-structural steel arrangement.

16.1.3 Patents and Copyrights

The entity or entities that are responsible for the specification and/or selection of proprietary structural designs shall secure all intellectual property rights necessary for the use of those designs.

16.1.4 Existing Structures

Unless specifically otherwise specified in the tender documents, the scope of works to be carried out by the Fabricator and /or Erector shall include :

16.1.4.1 Demolition and shoring of any part of an existing structure, if required,

16.1.4.2 Protection of existing structures and its contents and equipment, so as to prevent damage from erection works,

16.1.4.3 Surveying or field dimensioning of relevant existing structures, and

16.1.4.4 Abatement or removal of Hazardous Materials.

Such works shall be performed in a timely manner so as not to interfere with or delay the Fabrication and/or the Erection works.

16.2 SHOP FABRICATION AND DELIVERY

All workmanship shall be of first class quality in every respect. The greatest accuracy shall be observed to ensure that all parts will fit properly togrther on erection.

16.2.1 Identification of Material

16.2.1.1 Material ordered to special requirements shall be marked by the supplier prior to delivery to the Fabricator's shop or other point of use.

Material ordered to special requirements, but not so marked by the Supplier, shall not be used until:

a- its identification is established by testing in accordance with the applicable Egyptian Standard Specifications; and

b- a Fabricator's identification mark, as described in Section 16.2.1.2 and 16.2.1.3, has been applied.

16.2.1.2 During fabrication, up to the point of assembling members, each piece of material ordered to special requirements shall carry a Fabricator's identification mark or an original Supplier's identification mark. The Fabricator's identification mark shall be in accordance with the Fabricator's established identification system, which shall be made available prior to the start of fabrication for the Employer's Designated Representative for Construction (hereinafter called EDRC), the Building-Code Authority and the Inspector.

16.2.1.3 Parts that are made of material ordered to special requirements shall not be given the same assembling or erection mark as members made of other material, even if they are of identical dimensions and detail.

16.2.2 Preparation of Material

16.2.2.1 Thermal cutting of structural steel by hand-guided or mechanically guided means is permitted.

16.2.2.2 Surfaces that are specified as "Finished" in the Contract Documents shall have a suitable roughness value. The use of any fabricating technique that produces such a finish is permitted.

16.2.3 Fitting and Fastening

16.2.3.1 Projecting elements of Connection materials need not be straightened in the connecting plane.

16.2.3.2 Backing bars and runoff tabs shall be used as required to produce sound welds. The Fabricator or Erector need not remove backing bars or runoff tabs unless such removal is specified in the Contract Documents. When the removal of backing bars is specified in the Contract Documents, such removal shall meet the requirements in the *relevant welding specification*. In such cases, hand flame-cutting close to the edge of the finished member with no further finishing is permitted, unless other finishing is specified in the Contract Documents.

16.2.4 Fabrication Tolerances

The tolerances on structural steel fabrication shall be in accordance with the requirements in Chapter 15 of this Code.

16.2.5 Shop Cleaning and Painting

Unless otherwise specified, steel work shall be given one coat of approved red lead paint after it has been accepted and before it is shipped from the works.

Surfaces not in contact, but inaccesible after assembly or erection, shall be painted three coats. The shop contact surfaces shall not be painted. Field contact surfaces shall receive a shop coat of paint, except main splices for chords of trusses and large girder splices involving multiple thickness of material where a shop coat of paint would make erection difficult. Field contact surfaces not painted with the shop coat shall be given an approved protective coating if it is expected that there will be a prolonged period of exposure before erection.

Surfaces, which will be in contact with concrete, shall not be painted.

Structural steel, which is to be welded, shall not be painted before welding is complete. If it is to be welded only in the fabricating shop and subsequently erected by bolting, it shall receive one coat of paint after shop welding is finished. Steel, which is to be field welded, shall be given an approved protective coating after shop welding and shop fabrication is completed.

Surfaces of iron and steel castings, either milled or finished, shall be given one coat of paint.

With the exception of abutting joints and base plates, machine-finished surfaces shall be coated as soon as practicable after being accepted, with an approved coating, before removal from the shop.

Erection marks for field indication of members and weight marks shall be painted upon surface areas previously painted with the shop coat. Material shall not be loaded for shipment until it is throughly dry, and in any case not less than 24 hours after the paint has been applied.

Structural steel that does not require shop paint shall be cleaned from any oil or grease with solvent cleaners, and of dirt and other foreign material by sweeping with a fiber brush or other suitable means. For structural steel that is required to be shop painted, the requirements in Sections 16.2.5.1 through 16.2.5.4 shall apply.

16.2.5.1 The Fabricator is not responsible for deterioration of the shop coat that may result from exposure to exceptional atmospheric or corrosive conditions that are more severe than the normal ones.

16.2.5.2 Unless otherwise specified in the Contract Documents, the Fabricator shall, as a minimum, hand clean the structural steel of loose rust, loose mill scale, dirt and other foreign matters, prior to painting, by means of wire brushing or by other methods selected by the Fabricator. If the Fabricator's workmanship on surface preparation is to be inspected by the Inspector, such inspection shall be performed in a timely manner prior to the application of the shop coat.

16.2.5.3 Unless otherwise specified in the Contract Documents, paint shall be applied by brushing, spraying, rolling, flow coating, dipping or other suitable means, at the election of the Fabricator. When the term "shop coat", "shop paint" or other equivalent term is used with no paint system specified, the Fabricator's standard shop paint shall be applied to a minimum dry-film thickness of one coat [40 micron].

16.2.5.4 Touch-up of abrasions caused by handling after painting shall be the responsibility of the Contractor that performs field painting.

16.2.6 Marking and Shipping of Materials

16.2.6.1 Unless otherwise specified in the Contract Documents, erection marks shall be applied to the structural steel members by painting or other suitable means.

16.2.6.2 Bolt assemblies and loose bolts, nuts and washers shall be shipped in separate closed containers according to length and diameter, as applicable. Pins and other small parts and packages of bolts, nuts and washers shall be shipped in boxes, crates, kegs or barrels. A list and description of the material shall appear on the outside of each closed container.

16.2.7 Delivery of Materials

16.2.7.1 Fabricated structural steel shall be delivered in a sequence that will permit efficient and economical fabrication and erection, and that is consistent with the requirements of the Contract Documents. If the Employer or EDRC wishes to prescribe or control the sequence of delivery of materials, that entity shall specify the required sequence in the Contract Documents. If the EDRC contracts separately for delivery and for erection, the EDRC shall coordinate between contractors.

16.2.7.2 Anchor Rods, washers, nuts and other anchorage or grillage materials that are to be built into concrete or masonry shall be shipped so that they will be available when needed. The EDRC shall allow the Fabricator sufficient time to fabricate and ship such materials before they are needed.

16.2.7.3 If any shortage is claimed relative to the quantities of materials that are shown in the shipping statements, the EDRC or the Erector shall promptly notify the Fabricator so that the claim can be investigated

16.2.7.4 Unless otherwise specified in the Contract Documents, and subject to the approved Shop and Erection Drawings, the Fabricator shall limit the number of field splices to that consistent with minimum project cost.

16.2.7.5 If material arrives at its destination in damaged condition, the receiving entity shall promptly notify the Fabricator and Carrier prior to unloading the material, or promptly upon discovery prior to erection.

16.3 ERECTION

16.3.1 Method of Erection

Fabricated Structural Steel shall be erected using methods and sequences that will permit efficient and economical performance of erection, and that are consistent with the requirements of the Contract Documents. If the EDRC wishes to prescribe or control the method and/or sequence of erection, or specifies that certain members cannot be erected in their normal sequence, the required method and sequence has to be specified in the Contract Documents. If the EDRC contracts separately for fabrication services and for erection services, the EDRC shall coordinate between contractors.

16.3.2 Job-Site Conditions

The EDRC shall provide and maintain the following for the Fabricator and the Erector:

a- Adequate access roads into and through the job site for the safe delivery and movement of the material to be erected and of derricks, cranes, trucks and other necessary equipment.

b- A firm, properly graded, drained, convenient and adequate space at the job site for the operation of the Erector's equipment, free from overhead obstructions, such as power lines, telephone lines or similar conditions; and

c- Adequate storage space, when the structure does not occupy the full available job site, to enable the Fabricator and/or the Erector to operate at practical speed. Otherwise, the EDRC shall inform the Fabricator and the Erector of the actual job-site conditions and/or special delivery requirements in the tender documents.

16.3.3 Foundations, Piers and Abutments

The location, strength and suitability of, and access to, all foundations, piers and abutments shall be the responsibility of the EDRC.

16.3.4 Building Lines and Bench Marks

The EDRC shall be responsible for the accurate location of building lines and bench-marks at the job site and shall furnish the Erector with a plan that contains all such information. The EDRC shall establish offset building lines and reference elevations at each level for the Erector's usage in the positioning of adjustable Items (see Section 16.3.16), if any.

16.3.5 Installation of Anchor Bolts, Foundation Bolts and Other Embedded Items

16.3.5.1 Anchor rods, foundation bolts and other embedded items shall be set by the EDRC in accordance with an approved Embedment Drawing. The variation in location of these items from the dimensions shown in the Embedment Drawings shall be as mentioned in Section 15.6

16.3.5.2 Unless otherwise specified in the Contract Documents, Anchor Rods shall be set with their longitudinal axis perpendicular to the theoretical bearing surface.

16.3.5.3 Embedded items and connection materials that are part of the work of other trades, but that will receive Structural Steel, shall be located and set by the EDRC in accordance with an approved Embedment Drawing. The variation in location of these items shall be limited to a magnitude that is consistent with the tolerances that are specified in Section 16.3.5.1 for the erection of the Structural Steel.

16.3.5.4 All work that is performed by the EDRC shall be completed so as not to delay the work of the Fabricator and/or the Erector.

The EDRC shall conduct a survey of the as-built locations of Anchor Rods, foundation bolts and other embedded items, and shall verify that all items covered in Section 16.3.5.1 meet the corresponding tolerances. When corrective action is necessary, the EDRC shall obtain the guidance and approval of the EDRD.

16.3.6 Installation of Bearing Devices

All leveling plates, leveling nuts and washers and loose base and bearing plates that can be handled without a derrick or crane are set to line and grade by the EDRC. Loose base and bearing plates that require handling with a derrick or crane shall be set by the Erector to lines and grades established by the EDRC. The Fabricator shall clearly scribe loose base and bearing plates with lines or other suitable marks to facilitate proper alignment. Promptly after the setting of Bearing Devices, the EDRC shall check them for line and grade. The variation in elevation relative to the established grade for all Bearing Devices shall be equal to or less than plus or minus 3 mm. The final location of Bearing Devices shall be the responsibility of the Erector.

16.3.7 Grouting

Grouting shall be the responsibility of the Erector. Leveling plates and loose base and bearing plates shall be promptly grouted after they are set, checked for line and grade, and approved by EDRC. Columns with attached base plates, beams with attached bearing plates and other similar members with attached bearing devices that are temporarily supported on leveling nuts and washers, shims or other similar leveling devices, shall be promptly grouted after the Structural Steel frame or portion thereof has been plumbed.

16.3.8 Field Connection Material

16.3.8.1 The Fabricator shall provide field connection details that are consistent with the requirements of the Contract Documents and that will result in economical fabrication and erection.

16.3.8.2 When the Fabricator is responsible for erecting the Structural Steel, the Fabricator shall furnish all materials that are required for both temporary and permanent connection of the component parts of the Structural Steel frame.

16.3.8.3 When the erection of the Structural Steel is not performed by the Fabricator, the Fabricator shall furnish the following field connection material:

a- Bolts, nuts and washers of the required grade, type and size in sufficient quantity for all Structural Steel-to-Structural Steel field connections that are to be permanently bolted, including an extra 2 percent of each bolt size (diameter and length);

b- Shims that are shown as necessary for make-up of permanent structural steel-to-structural steel connections; and,

c- Backing bars and run-off tabs that are required for field welding.

16.3.8.4 The Erector shall furnish all welding electrodes, fit-up bolts and drift pins used for the erection of the Structural Steel.

16.3.9 Loose Material

Unless otherwise specified in the Contract Documents, loose Structural Steel items that are not connected to the Structural Steel frame shall be set by the Erector.

16.3.10 Temporary Support of Structural Steel Frames

16.3.10.1 The EDRD shall identify the following in the Contract Documents:

a- The lateral-load-resisting system and connecting diaphragm elements that provide for lateral strength and stability in the completed structure; and,

b- Any special erection conditions or other considerations that are required by the design concept, such as the use of shores, jacks or loads that must be adjusted as erection progresses to set or maintain camber, position within specified tolerances or pre-stress.

16.3.10.2 The EDRD shall indicate to the Erector, in the tender documents, the installation schedule for non-Structural Steel elements of the lateral-load-resisting system and connecting diaphragm elements identified in the Contract Documents.

16.3.10.3 Based upon the information provided in accordance with Sections 16.3.10.1 and 16.3.10.2, the Erector shall determine, furnish and install all temporary supports, such as temporary guys, beams, falsework, cribbing or other elements required for the erection operation. These temporary supports shall be sufficient to secure the bare Structural Steel framing or any portion thereof against loads that are likely to be encountered during erection, including those due to wind and those that result from erection operations.

16.3.10.4 All temporary supports required for the erection operation and furnished and installed by the Erector shall remain the property of the Erector and shall not be modified, moved or removed without the consent of the Erector. Temporary supports provided by the Erector shall remain in place until the portion of the Structural Steel frame that they brace is complete and the lateral-load-resisting system and connecting diaphragm elements identified by the EDRD in accordance with Section 16.3.10.1 are installed. Temporary supports required to be left in place after the completion of Structural Steel erection shall be removed when no longer needed by the EDRC and returned to the Erector.

16.3.11 Safety Protection

16.3.11.1 The Erector shall provide floor coverings, handrails, walkways and other safety protection for the Erector's personnel as required by law and applicable safety regulations. Unless otherwise specified in the Contract Documents, the Erector is permitted to remove such safety protection from areas where the erection operations are completed and approved by EDRC.

16.3.11.2 When safety protection provided by the Erector is left in an area for the use of other trades after the Structural Steel erection activity is completed, the EDRC shall:

a- Indemnify the Fabricator and/or the Erector from damages that may be incurred from the use of this protection by other trades;

b- Ensure that this protection is adequate for use by other affected trades;

c- Ensure that this protection complies with applicable safety regulations when being used by other trades; and

d- Instruct the Fabricator and/or the Erector to remove this protection when it is no longer required.

16.3.11.3 Safety protection for other trades that are not under the direct employment of the Erector shall be the responsibility of the EDRC.

16.3.11.4 When permanent steel decking is used for protective flooring and is installed by the EDRC, all such work shall be scheduled and performed in a timely manner so as not to interfere with or delay the work of the Fabricator or the Erector.

16.3.11.5 Unless the interaction and safety of activities of others, such as construction by others or the storage of materials that belong to others, are coordinated with the work of the Erector by the EDRC, such activities shall not be permitted until the erection of the Structural Steel frame or portion thereof is completed by the Erector and accepted by the EDRC.

16.3.12 Structural Steel Frame Tolerances

The accumulation of the mill tolerances and fabrication tolerances shall not cause the erection tolerances to be exceeded.

16.3.13 Erection Tolerances

Erection tolerances shall be defined relative to member working points and working lines, which shall be defined as follows:

a- For members other than horizontal members, the member work point shall be the actual center of the member at each end of the shipping piece.

b- For horizontal members, the working point shall be the actual centerline of the top flange or top surface at each end.

c- The member working line shall be the straight line that connects the member working points.

The substitution of other working points is permitted for ease of reference, provided they are based upon the above definitions. The tolerances on Structural Steel erection shall be in accordance with the requirements in Chapter 15 of this Code.

16.3.14 Correction of Errors

The correction of minor misfits by moderate amounts of reaming, grinding, welding or cutting, and the drawing of elements into line with drift pins, shall be considered to be normal erection operations. Errors that cannot be corrected using the foregoing means, or that require major changes in member or connection configuration, shall be promptly reported to the EDRD and EDRC and the Fabricator by the Erector, to enable the responsible entity to either correct the error or approve the most efficient and economical method of correction to be used by others.

16.3.15 Cuts, Alterations and Holes for other Trades

Neither the Fabricator nor the Erector shall cut, drill or otherwise alter their work, nor the work of other trades, to accommodate other trades, unless such work is clearly specified in the Contract Documents. When such work is so specified, the EDRD and EDRC shall furnish complete information as to materials, size, location and number of alterations in a timely manner so as not to delay the preparation of Shop and Erection Drawings.

16.3.16 Handling and Storage

The Erector shall take reasonable care in the proper handling and storage of the Structural Steel during erection operations to avoid the accumulation of excess dirt and foreign matter. The Erector shall be responsible for the removal from the Structural Steel of dust, dirt or other foreign matter that may accumulate during erection as the result of job-site conditions or exposure of the elements.

16.3.17 Field Painting

The Erector is neither responsible to paint field bolt heads and nuts or field welds, nor to touch up abrasions of the shop coat, nor to perform any other field painting.

16.3.18 Final Cleaning Up

Upon the completion of erection and before final acceptance, the Erector shall remove all of the Erector's falsework, rubbish and temporary buildings.

16.4 QUALITY ASSURANCE

16.4.1 General

16.4.1.1 The Fabricator shall maintain a quality assurance program to ensure that the work is performed in accordance with the requirements of this Code and the Contract Documents.

16.4.1.2 The Erector shall maintain a quality assurance program to ensure that the work is performed in accordance with the requirements of this Code and the Contract Documents. The Erector shall bear the costs of performing the erection of the Structural Steel, and shall provide all necessary equipment, material, personnel and management for the scope, magnitude and required quality of each project.

16.4.2 Inspection of Mill Material

Certified mill test reports shall constitute sufficient evidence that the mill product satisfies material order requirements. The Fabricator shall make a visual inspection of material that is received from the mill, but need not perform any material tests unless the EDRD specifies in the Contract Documents that additional testing is to be performed.

16.4.3 Non-Destructive Testing

When non-destructive testing is required, the process, extent, technique and standards of acceptance shall be clearly specified in the Contract Documents.

16.4.4 Surface Preparation and Shop Painting Inspection

Inspection of surface preparation and shop painting shall be planned for the acceptance of each operation as the Fabricator completes it. Inspection of the paint system, including material and thickness, shall be made promptly upon completion of the paint application. When wet-film thickness is to be inspected, it shall be measured during the application.

16.4.5 Independent Inspection

When inspection by personnel other than those of the Fabricator and/or Erector is specified in the Contract Documents, the requirements in Sections 16.4.5.1 through 16.4.5.6 below shall be met.

16.4.5.1 The Fabricator and/or the Erector shall provide the Inspector with access to all places where the work is being performed. A minimum of 24 hours notification shall be given prior to the commencement of work.

16.4.5.2 Inspection of shop work by the Inspector shall be performed in the Fabricator's shop to the fullest extent possible. Such inspections shall be timely, in-sequence and performed in such a manner as will not disrupt fabrication operations and will permit the repair of non-conforming work prior to any required painting while the material is still in-process in the fabrication shop.

16.4.5.3 Inspection of field work shall be promptly completed without delaying the progress or correction of the work.

16.4.5.4 Rejection of material or workmanship that is not in conformance with the Contract Documents shall be permitted at any time during the progress of the work.

16.4.5.5 The Fabricator and/or the Erector shall be informed of deficiencies that are noted by the Inspector promptly after the inspection. Copies of all reports prepared by the Inspector shall be promptly given to the Fabricator and/or the Erector. The necessary corrective work shall be performed in a timely manner.

16.4.5.6 The Inspector shall not suggest, direct, or approve the Fabricator and/or Erector to deviate from the Contract Documents or the approved Shop and Erection Drawings, or approve such deviation, without the written approval of the EDRD and EDRC.

16.5 CONTRACTS

16.5.1 Types of Contracts

16.5.1.1 For contracts that stipulate a lump sum price, the work that is required to be performed by the Fabricator and/or the Erector shall be completely defined in the Contract Documents.

16.5.1.2 For contracts that stipulate a price per ton, the scope of work that is required to be performed by the Fabricator and/or the Erector, the type of materials, the character of fabrication and the conditions of erection shall be based upon the Contract Documents, which shall be representative of the work to be performed.

16.5.1.3 For contracts that stipulate a price per item, the work that is required to be performed by the Fabricator and/or the Erector shall be based upon the quantity and the character of the items described in the Contract Documents.

16.5.1.4 For contracts that stipulate unit prices for various categories of Structural Steel, the scope of work required to be performed by the Fabricator and the Erector shall be based upon the quantity, character and complexity of the items in each category as described in the Contract Documents, and shall also be representative of the work to be performed in each category.

16.5.2 Calculation of Weights

Unless otherwise specified in the contract, for contracts stipulating a price per ton for fabricated Structural Steel that is delivered and/or erected, the quantities of materials for payment shall be determined by the calculation of the gross weight of materials as shown in the Shop Drawings.

16.5.2.1 The unit weight of steel shall be taken as 7850 kg/m^3. The unit weight of other materials shall be in accordance with the manufacturer's published data for the specific product.

16.5.2.2 The weights of Standard Structural shapes, plates and bars shall be calculated on the basis of Shop Drawings that show the actual quantities and dimensions of material to be fabricated, as follows:

a- The weights of all Standard Structural shapes shall be calculated using the nominal weight per meter and the detailed overall length.

b- The weights of plates and bars shall be calculated using the detailed overall rectangular dimensions.

c- When parts can be economically cut in multiples from material of larger dimensions, the weight shall be calculated on the basis of the theoretical rectangular dimensions of the material from which the parts are cut.

d- When parts are cut from Standard Structural shapes, leaving a non-standard section that is not useable on the same contract, the weight shall be calculated using the nominal weight per meter and the detailed overall length of the Standard Structural shapes from which the parts are cut.

e- Deductions shall not be made for material that is removed for cuts, copes, clips, blocks, drilling, punching, boring, slot milling, planing or weld joint preparation

16.5.2.3 The weights of shop or field weld metal and protective coatings shall not be included in the calculated weight for the purposes of payment.

16.5.3 Revisions to the Contract Documents

Revisions to the Contract Documents shall be confirmed by variation or change order or extra work order. Unless otherwise noted, the issuance of a revision to the Contract Documents shall constitute authorization by the Employer that the revision is released for construction. The contract price and schedule shall be adjusted in accordance with Sections 16.5.4 and 16.5.5.

16.5.4 Contract Price Adjustment

16.5.4.1 When the scope of work and responsibilities of the Fabricator and/or the Erector are changed from those previously established in the Contract Documents, an appropriate modification of the contract price shall be made. In computing the contract price adjustment, the Fabricator and the Erector shall consider the quantity of work that is added or deleted, the modifications in the character of the work and the status of material ordering, detailing, fabrication and erection operations.

16.5.4.2 Requests for contract price adjustments shall be presented by the Fabricator and/or the Erector in a timely manner and shall be accompanied by a description of the change that is sufficient to permit evaluation and timely approval by the Employer.

16.5.4.3 Price-per-ton and price-per-item contracts shall provide for additions or deletions to the quantity of work that are made prior to the time the work is released for construction. When changes are made to the character of the work at any time, or when additions and/or deletions are made to the quantity of the work after it is released for detailing, fabrication or erection, the contract price shall be equitably adjusted.

16.5.5 Scheduling

Design Drawings will be released for construction, if such Design Drawings are not available at the time of bidding, and/or when the job site, foundations, piers and abutments will be ready, free from obstructions and accessible to the Erector, so that erection can start at the designated time and continue without interference or delay caused by the EDRC or other trades.

16.5.5.2 The Fabricator and/or the Erector shall advise the Employer's EDRC and EDRD, in a timely manner, of the effect any revision has on the contract schedule.

16.5.5.3 If the fabrication or erection is significantly delayed due to revisions of the requirements of the contract, or for other reasons that are the responsibility of others, the Fabricator and/or Erector shall be compensated for the additional time and/or costs incurred (if any).

16.5.6 Terms of Payment

Terms of payment for the contract shall be as stated in the Contract Documents.

CHAPTER 17

INSPECTION AND MAINTENANCE OF STEEL STRUCTURES

17.1 GENERAL

Steel structures are subject to gradual deterioration due to corrosion from environmental effect, impact, and fatigue damage from dynamic loads, that require periodic maintenance throughout their service life.

The damage likely to get worse or to expose the safety of the structure to danger, should be repaired as quickly as possible in all cases.

17.2 INSPECTION

The inspection of steel structures may be classified as periodic inspection and special or detailed inspection.

17.2.1 Periodic Inspection

This kind of inspection should be made at frequent, scheduled intervals depending on the condition and age of the structure. Generally, this inspection is made every two to three years and covers the following points:

a- General condition of paint on the entire steel structure.

b- Condition of the parts of the frame work with which the construction design allows water to rest in contact for prolonged periods or parts which may undergo the aggressive action of outside agents as smoke or vapours.

c- The state of rivets, bolts, welds in floor beams' connections, and also those in splices of main girders or connecting web members to upper and lower chord member in trusses.

d- Condition of the gusset plates where pitches and edge distances of bolts may be found to exceed allowable maximum values.

17.2.2 Special or Detailed Inspection

Steel structures shall undergo detailed inspection at least once every 4-6 years. This inspection shall cover the following points:

a- Location and number of bolts that are loose.

b- Welds on lateral bracing and cross frames, stiffeners and other welded details must be examined.

c- Condition of members as to loss of section due to corrosion, noting exact location and extent of such action. Measurement of remaining section if members are badly corroded and paying attention to loss of metal in girders, beam flanges, webs, as well as parts of lateral bracing system.

d- Permanent deflection shall be measured for girders more than 15.0 m span, and compared with previous values to ensure that there is no creep.

17.2.3 Inspection Sketch for Identification of Members

Typical sketches are to be prepared by inspector to show correct identity and location of parts or members described in their inspection report. Photographs shall be used to show critical conditions and to amplify the value of the report.

17.2.4 Files of Structures

There must be a file for each structure containing the following:

a- Type and origin of materials and tests carried out before and during construction.

b- Type of foundation and soil investigation report.

c- Detailed as built drawings of all parts of the structure.

d- The calculation notes.

e- The priced bill of quantities used.

f- Results of tests and comparison with theoretical calculations.

g- Possible incidences taking place during construction.

h- Maintenance carried out especially dates when the structure was repaired or strengthened.

i- Repairs or alterations made during services.

j- Reports of inspections carried out.

k- Photographic documents on the construction phases as those, which may concern different defects or damage.

17.3 MAINTENANCE OF STEEL STRUCTURES

17.3.1 Maintenance of Structural Elements

17.3.1.1 Normal maintenance must include periodical cleaning of all exposed surfaces by compressed air.

17.3.1.2 Parts which are exposed to direct attack by smoke or the projection of aggressive products as salts, solvents.. etc., should be protected.

17.3.1.3 Holes or cracks in the substructure can be maintained by appropriate grouting or proper use of epoxy or epoxy mortar.

17.3.1.4 It is necessary to ensure that neither water can exist on roof of the structure, nor it be allowed to accumulate in any member of the structure. Drainage holes with reasonable diameters must be provided to this effect.

17.3.1.5 Painting

a- For structures not greatly exposed to corrosion, the life of well applied paint is at least (8-10) years. Intermediate maintenance operations may be resorted to, for parts of the structure which are severely exposed to rust or for which this period would be harmful.

b- Where the paint is to be maintained on steel surfaces, the steel shall be prepared and painted with the recommendations of the relevant Egyptian Standard Specification.

c- Structures where paint is worn off before the 8-10 years period, shall undergo special inspections to decide if the time between two successive general painting operations should be reduced or if it would be necessary to apply special paint.

17.3.1.6 Bolting

During the detailed inspection of structures with high strength bolted connections, the torque of the tightening of these bolts should be checked by a calibrated wrench.

17.3.1.7 Cracks in Old Structures

It is not recommended to weld a crack in a bolted girder, since, with time this shall cause other cracks, however welding may be used in cases where bolting is not possible. The cracked part can be replaced by a new part or the crack can be covered with a bolted cover plate. The crack propagation may be stopped by drilling 15 mm holes at either end of the crack. Besides, the splice plate ends must be in low stress range areas.