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Code for Design of Steel Structures

钢 结 构 设 计 规 范

GB 50017－2003

（英 文 版）

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NOTICE

This code is written in Chinese and English. The Chinese text shall be taken as the ruling one in the event of any inconsistency between the Chinese text and the English text.

Bulletin of Ministry of Construction of the People's Republic of China

Bulletin No. 147

Bulletin of Promulgation for the National Standard "Code for Design of Steel Structures"

"Code for Design of Steel Structures" has been approved as a national standard with a serial number of GB 50017—2003, and it shall come into force upon December 1, 2003. Herein, Clauses 1.0.5, 3.1.2, 3.1.3, 3.1.4, 3.1.5, 3.2.1, 3.3.3, 3.4.1, 3.4.2, 8.1.4, 8.3.6, 8.9.3, 8.9.5, 9.1.3 are mandatory clauses, which must be enforced strictly. The original "Code for Design of Steel Structures" GBJ 17—88 shall be abolished simultaneously.

Research Institute of Standards and Norms-Ministry of Construction will organize the China Planning Press to take on publishing and distributing works of this code.

Ministry of Construction of the People's Republic of China

April 25, 2003

Preface

According to the requirement of the document Jian Biao [1997] No. 108 of the Ministry of Construction, the Beijing Central Engineering and Research Incorporation of Iron and Steel Industry, together with relevant design, education and research institutions, formed a revising-drafting group and proceeded a comprehensive revision of the "Code for Design of Steel Structures" GBJ 17—88. In the process of the work, an overall revision program was mapped out, and quite a few design codes of foreign countries have been consulted. Solicitation of opinions from all sides was carried out upon completion of the first draft. After many amendments, by putting forward successively the first draft, the draft for seeking opinions and that for reviewing, and upon the performance of tentative design projects by ten-odd participating units for comparison between the new and old codes, the final draft for approval of the "Code for Design of Steel Structures" GB 50017—2003 was completed in December 2001. The major amendments of this revision are as follows:

1. The provisions regarding "quality level of weld", originally Clause 1.0.5 in the Commentary of the former code, has been moved to the text as Clause 7.1.1 of Chapter 7 of the Code. Moreover, the classification principle and specific rule have been added.

2. According to the requirement of the document Jian Biao [1996] No. 626 "Prescription for writing standard of construction work", clauses of "Terms" have been added and compiled together with "Symbols" into Chapter 2. The contents of "Materials", Chapter 2 of the former version, are put into Chapter 3, as Section 3.3 "Material Selection".

3. According to the new National Standards of structural steel, steel grades Q235, Q345, Q390 are recommended and Q420 is added. Requirements for material quality guarantee that various steel structures shall meet are more complete than before. The condition of applicability of 0℃ notch toughness guarantee for Q235 steel has been added and the principle of using Z-direction steel and weathering steel prescribed as well. Meanwhile, the design indices of steels have been somewhat adjusted.

4. In Chapter 3, a section on "Load and calculation of load effects" has been added, emphasizing the appropriateness of using the elastic second-order analysis approach for unbraced pure frames, which considers the effect of deformation on internal forces. Amplification factor for crane transverse horizontal load in the former code has been deleted and calculation formula of transverse horizontal force caused by sway of crane has been given instead.

5. The amendments to the Section "Provisions for deformation of structures and structural members" are:

1) In the text of the Code, design principle solely is mentioned, whereas a table on

limiting values of deformation is given in the Appendix.

2) The limiting values of deformation may be suitably modified according to requirement and experience. The calculation of crane girder deflection under unfactored wheel loads of only one crane is prescribed.

6. Formulas for calculating local stability of girder webs have been significantly altered from the former code, considering no more the webs as fully elastic and perfect, but taking account of the effect of inelastic deformation and geometric imperfection. Furthermore, calculation method for taking account of web post-buckling strength is given, and the restraining factors to webs have been adjusted as well. The formulas for determining stiffener spacing according to fully elastic plate in the former code have been deleted.

7. The classification of sections of axial compression members has been enlarged to include I-and box section with component plates of thickness $t \geqslant 40$mm and the relevant φ factor of class d has been added.

8. The approach to calculating flexural-torsional buckling about the axis of symmetry of struts with mono-symmetric section has been added.

9. The method for calculating forces in lateral bracings used to reduce the unsupported length of compression members or compression flanges has been amended. Also amended is the approach to determining the out-of-plane effective length of cross-diagonals.

10. Frames are distinguished into three categories, namely unbraced pure frames, strongly braced frames and weakly braced frames, and the approach to calculating the effective lengths of these various frames has been given.

11. An approach to determining column effective length of unbraced pure frames and weakly braced frames containing leaning columns have been added.

12. The number of stress cycle, n, has been amended as follows: fatigue calculation shall be carried out when n is equal to or larger than 5×10^4 (in the former code, fatigue calculation is required only when n is equal to or larger than 10^5). Besides, minor amendments to the classification of members and connections for fatigue calculation have been adopted.

13. The limiting value of web depth-thickness ratio in T-section struts and that of beam-columns with web free edge under tension, has been amended.

14. Two sections on "beam-to-column rigid connection" and "calculation of plate elements in joints" have been added, the main contents of which are:

1) Provisions regarding column web or flange thicknees requirements in case no transverse stiffeners are provided to the column in a beam-to-column rigid joint.

2) Strength calculation of plate elements under combined tension and shear, that of truss gusset plates and relevant stability calculation method and prescription.

15. Provisions regarding plate bearing, spherical bearing and composite rubber and steel support have been replenished.

16. Prescriptions on design and detailing of inserted column base, imbedded column base and wrapped column base have been added.

17. Prescription on design and detailing requirements of large-span roof structures has been added.

18. Prescription on requirement of improving the brittle fracture resistance of structures in cold region has been added.

19. The reduction factor 0.9 to the design value of strength of steel and connection for plastic design and steel-concrete composite beams, as prescribed in the former code, has been deleted.

20. Formulas for calculating the strength of circular tube spatial nodes have been added. Method for calculating the strength of rectangular or square tube planar nodes and relevant detailing requirements have been supplemented.

21. The Chapter 11 "Light steel structures of round bars and small angles" of the former code has been deleted.

22. The following issues regarding steel-concrete composite beams have been supplemented: method for calculating the negative moment portion of continuous composite beams, calculation and detailing peculiarity of composite beams with concrete flange cast on profiled steel sheeting, design requirement for composite beams with partial shear-resisting connection and deflection calculation of composite beams.

Clauses marked with boldface letters in the code are mandatory clauses and must be enforced strictly.

The Ministry of Construction is in Charge of management and explanation of mandatory clauses in the code, while the Beijing Central Engineering and Research Incorporation of Iron and Steel Industry will be responsible for explanation of concrete contents. Users are solicited to sum up experiences in the course of Code implementation. Comments and suggestions on this Code are requested to send to the Administrative Group of the National Standard "Code for design of steel structures", Beijing Central Engineering and Research Incorporation of Iron and Steel Industry (Address: 4 Baiguang Road, Beijing, Postcode: 100053; FAX: 010-83587966).

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1 General principles

1.0.1 This Code intends to implement the technical-economic policy of the State in the design of steel structures, by using advanced technology and ensuring economy, reasonableness, safety, suitability for use and good quality of the structures.

1.0.2 This Code applies to the design of steel structures of industrial and civil buildings and allied engineering structures, among which members made of cold-formed steel shapes and their connections shall comply with the current national standard "Technical code of cold-formed thin-wall steel structures" GB 50018.

1.0.3 The design principles of this Code are based on the "Unified standard for reliability design of building structures" GB50068. Loadings and their combination values assumed in designing with this code shall comply with the current national standard "Load code for the design of building structures" GB 50009. Buildings and engineering structures in seismic region shall furthermore comply with the current national standards "Code for seismic design of buildings" GB 50011, "Seismic ground motion parameter zonation map of China" GB 18306 and "Design code for antiseismic of special structures" GB 50191.

1.0.4 In designing steel structures, designers shall consider the real situation of the project, select reasonably the material, the structural scheme and detailing measures. The requirements of strength, stability and stiffness of the structure during transportation, erection and service, as well as requirements of fire protection and corrosion resistance shall be fulfilled. Typical and standardized structures and structural members should be adopted in preference, the amount of fabrication and erection work should be reduced.

1.0.5 In the design documents of steel structures shall be indicated the design working life of the building structures, the steel grade, the category (or grade) of connection materials and mechanical properties, chemical composition and additional items of guarantee of the steel. Moreover, the weld type and the class of weld quality, the location of end planning for close fitting and its quality requirement shall also be indicated.

1.0.6 The design of steel structures with special requirements and those under special circumstances shall furthermore comply with the relevant current national codes.

2 Terms and Symbols

2.1 Terms

2.1.1 Strength

The capacity of resisting failure in member cross-section material or connection. Strength checking aims at preventing failure of structural members or connections from exceeding the material strength.

2.1.2 Load-carrying capacity

The largest internal force that a structure or member can bear without failure from strength, stability or fatigue, etc., or the largest internal force at the onset of failure mechanism in plastically analyzed structures; or the internal force generating a deformation that hinders further loading.

2.1.3 Brittle fracture

In general, the suddenly occurred brittle fracture of a steel structure subject to tensile stress without warning by plastic deformation.

2.1.4 Characteristic value of strength

The yield point (yield strength) or tensile strength of steel as specified by National Standard.

2.1.5 Design value of strength

The value obtained from division of the characteristic value of strength of steel or connection by corresponding partial factor of resistance.

2.1.6 First order elastic analysis

The elastic analysis of structure internal forces and deformation, based on the equilibrium condition of undeformed structure, taking no account of the effect of the second order deformation on infernal forces.

2.1.7 Second order elastic analysis

The elastic analysis of structure internal forces and deformation, based on the equilibrium condition of deformed structure, taking account of the effect of the second order deformation on internal forces.

2.1.8 Buckling

An abrupt large deformation, not conforming to the original configuration of members or plates subject to axial compression, bending moment or shear force, and thereby causing loss of stability.

2.1.9 Post-buckling strength of web plate

The capacity of web plates to bear further loading after buckling.

2.1.10 Normalized web slenderness

Parameter, equal to the square root of the quotient of steel yield strength in flexion, shear or compression by corresponding elastic buckling stress of web plates in flexion, shear or local compression.

2.1.11 Overall stability

Assessment of the possibility of buckling or loss of stability of structures or structural numbers as a whole under the action of external loading.

2.1.12 Effective width

That part of plate width assumed effective in checking the section strength and the stability.

2.1.13 Effective width factor

Ratio of the effective width to the actual width of a plate element.

2.1.14 Effective length

The equivalent length of a member obtained by multiplying its geometrical length within adjacent effective restraining points by a coefficient taking account of end deformation condition and loading condition. The length of welds assumed in calculation of the strength of welded connections.

2.1.15 Slenderness ratio

The ratio of member effective length to the radius of gyration of its cross-section.

2.1.16 Equivalent slenderness ratio

The slenderness ratio transforming a lattice member into solid-wed one according to the principle of equal critical force for checking the overall stability of axial compression members. The slenderness ratio transforming a flexural-torsional buckling and torsional bucking into flexural buckling.

2.1.17 Nodal bracing force

Force to be applied at the location of lateral support installed for reducing the unsupported length of a compression member (or compression flange of a member). This force acts in the direction of member buckling at the shear center of the member section.

2.1.18 Unbraced frame

Frames resisting lateral load by bending resistance of members and their connections.

2.1.19 Frame braced with strong bracing system

A frame braced with bracing system of large stiffness against lateral displacement (bracing truss, shear wall, elevator well, etc.), adequate to be regarded as frame without sidesway.

2.1.20 Frame braced with weak bracing system

A frame braced with bracing system of weak stiffness against lateral displacement, inadequate to be regarded as frame without sidesway.

2.1.21 Leaning column

A column hinged at both ends and not capable of resisting lateral load in a framed structure.

2.1.22 Panel zone of column web

The zone of column web within the beam depth at a rigid joint of frame.

2.1.23 Spherical steel bearing

A hinged or movable support transmitting force through a spheric surface allowing the structure to rotate in any direction at the support.

2.1.24 Composite rubber and steel support

A support transmitting end reaction through a composite product of rubber and thin steel plates satisfying the displacement requirement at the support.

2.1.25 Chord member

Members continuous through panel points in tubular structures, similar to chord members in regular trusses.

2.1.26 Bracing member

Members cut short and connected to the chord members at panel points in tubular structures, similar to web members in regular trusses.

2.1.27 Gap joint

Joints of tubular structures where the toes of two bracing members are distant from each other by a gap.

2.1.28 Overlap joint

Joints of tubular structures where the two bracing members are overlaping.

2.1.29 Uniplanar joint

Joints where chord member is connected to bracing members in a same plane.

2.1.30 Multiplannar joint

Tubular joints where chord member is connected to bracing members in different planes.

2.1.31 Built-up member

Members fabricated by joining more than one plate elements(or rolled shapes), such as built-up beams or columns of I-or box-section.

2.1.32 Composite steel and concrete beam

A beam composed of steel beam and concrete flange plate, acting as an integrated member by means of shear connectors.

2.2 Symbols

2.2.1 Actions and effects of actions

F——concentrated load;

H——horizontal force;

M——bending moment;

N——axial force;

P——pretension of high-strength bolts;

Q——gravity load;

R——reaction of support;

V——shear force.

2.2.2 Calculation indices

E——modulus of elasticity of steel;

E_c——modulus of elasticity of concrete;

G——shear deformation modulus of steel;

N_t^a——design value of tensile capacity of an anchor bolt;

N_t^b, N_v^b, N_c^b——design values of tensile, shear and bearing capacities of a bolt;

N_t^r, N_v^r, N_c^r——design values of tensile, shear and bearing capacities of a rivet;

N_v^c——design value of shear capacity of a connector in composite structures;

N_t^{pj}, N_c^{pj}——design values of capacities of bracing members in tension and in compression at a joint of tubular structures;

S_b——lateral sway stiffness of bracing structures (horizontal force causing a leaning angle of unity);

f——design value of tensile, compressive and bending strength of steel;

f_v——design value of shear strength of steel;

f_{ce}——design value of end bearing strength of steel;

f_{st}——design value of tensile strength of reinforcing bars;

f_y——yield strength (or yield point) of steel;

f_t^a——design value of tensile strength of an anchor bolt;

f_t^b, f_v^b, f_c^b——design values of tensile, shear and bearing strengths of bolts ;

f_t^r, f_v^r, f_c^r——design values of tensile, shear and bearing strengths of rivets;

f_t^w, f_v^w, f_c^w——design values of tensile, shear and compressive strengths of butt welds;

f_f^w——design value of tensile, shear and compressive strength of fillet welds;

f_c——design value of axial compressive strength of concrete;

Δu——lateral inter-story deflection;

$[v_Q]$——allowable deflection taking into account solely the characteristic value of variable loads;

$[v_T]$——allowable deflection taking into account the characteristic value of permanent and variable loads simultaneously;

σ——normal stress;

σ_c——local compressive stress;

σ_f——stress normal to the direction of the length of a fillet weld, calculated on its effective section;

$\Delta\sigma$——stress range or reduced stress range for fatigue calculation;

$\Delta\sigma_e$——equivalent stress range of variable amplitude fatigue;

$[\Delta\sigma]$——allowable stress range of fatigue;

$\sigma_{cr}, \sigma_{c,cr}, \tau_{cr}$——critical stresses of plate under individual action of bending stress, local compressive stress and shear stress respectively;

τ——shear stress;

τ_f——shear stress of a fillet weld along the direction of its length calculated on its effective section;

ρ——density of mass.

2.2.3 Geometric parameters

A——gross sectional area;

A_n——net sectional area;

H——column height;

H_1, H_2, H_3——heights of the upper, middle (or lower) and lower portions of stepped columns;

I——moment of inertia of gross section;

I_t——torsional moment of inertia (St. Venant torsion constant) of gross section;

I_ω——sectorial moment of inertia (warping constant) of gross section;

I_n——moment of inertia of net section;

S——first moment of gross sectional area;

W——gross section modulus;

W_n——net section modulus;

W_p——plastic gross section modulus;

W_{pn}——plastic net section modulus;

a, g——spacing; gap;

b——plate width or free outstand of plate;

b_0——flange unsupported width between webs of a box-section; width of the top surface of the concrete haunch;

b_s——outstand of stiffeners;

b_e——effective width of plate;

d——diameter;

d_e——effective diameter;

d_0——hole diameter;

e——eccentricity;

h——full height of a section (section depth); story height;

h_{c1}——thickness of concrete slab;

h_{c2}——thickness of concrete haunch;

h_e——effective thickness of fillet welds;

h_f——leg size of fillet welds;

h_w——web height(web depth);

h_0——effective web height;

i——radius of gyration of a section;

l——length or span length;

l_1——spacing of lateral supports in the compression flange of a beam; connecting length of bolted (riveted) joints in the direction of force;

l_0——effective length for flexural buckling;

l_ω——effective length for torsional buckling;

l_w——effective length of welds;

l_z——assumed distribution length of a concentrated load on the edge of effective web depth;

s——shortest distance from the root of the groove to weld surface in an incomplete penetration butt weld;

t——plate thickness; wall thickness of (tubular) chord members;

t_s——stiffener thickness;

t_w——web thickness;

α——angle;

θ——angle; angle of stress dispersal;

λ_b——normalized depth-thickness ratio in calculating girder web subject to bending moment;

λ_s——normalized depth-thickness ratio in calculating girder web subject to shear force;

λ_c——normalized depth-thickness ratio in calculating girder web subject to local compressive force;

λ——slenderness ratio;

$\lambda_0, \lambda_{yz}, \lambda_z, \lambda_{uz}$——equivalent slenderness ratio.

2.2.4 Coefficients of calculation and others

C——dimensional parameter for fatigue calculation;

K_1, K_2——ratios of linear stiffness of members;

k_s——shear buckling factor of members;

O_v——overlap ratio of bracing members in tubular joints;

n——number of bolts, rivets or connectors; number of stress cycles;

n_1——number of bolts(or rivets)on a calculated section;

n_f——number of frictional force transferring surfaces in a high-strength bolted connection;

n_v——number of shear planes of bolts or rivets;

α——coefficient of linear expansion; coefficient for calculating transverse force generated by crane sway;

α_E——modular ratio of steel to concrete;

α_e——reduction factor of girder section modulus taking account of web effective depth;

α_f——equivalent factor of underloading effect for fatigue calculation;

α_0——stress gradient factor of column web;

α_y——factor of steel strength effect;

α_l——factor for planed and closely fitted web edge;

α_{2i}——amplification coefficient for bending moment of the i-th story members due to lateral translation of a frame, taking account of second order effect;

β——ratio of outside diameter of bracing member to that of chord member; parameter for fatigue calculation;

β_b——factor of equivalent critical moment for overall stability of beams;

β_f——amplification coefficient for design value of the transverse fillet weld strength;

β_m, β_t——factors of equivalent moment for beam-column stability;

β_1——amplification coefficient of design value of strength for reduced stress;

γ——strength-yielding ratio of stud steel;

γ_0——importance factor of structures;

γ_x, γ_y——plasticity adaptation factor of cross-sections about principal axes x, y;

η——modification factor;

η_b——factor of unsymmetry of a beam section;

η_1, η_2——parameters for calculation the effective length of stepped columns;

μ——slip coefficient for friction surfaces in a high-strength bolted connection; effective length factor of columns;

μ_1, μ_2, μ_3——effective length factors for the upper, middle (or lower) and lower portions of stepped columns;

ξ——parameter for checking overall stability of beams;

ρ——effective width factor of web compressive zone;

φ——stability factor of axial compression members;

φ_b, φ'_b——overall stability factors of beams;

ψ——amplification coefficient of a concentrated load ;

ψ_n, ψ_a, ψ_d——parameters for capacity calculation of directly welded tubular joints.

3 Basic design stipulations

3.1 Design principles

3.1.1 For all calculations except fatigue calculation, the limit state design method based on probabilistic theory is adopted, using design expressions with partial safety factors.

3.1.2 Load-carrying structures shall be designed according to the following ultimate limit states and serviceability limit states:

1 The ultimate limit states include: strength failure of members and connections, fatigue failure and excessive deformation no longer suitable for carrying load, loss of stability of structures and members, formation of mechanism and overturning of the structure.

2 The serviceability limit states include: deformations affecting normal use and appearance of a structure, structural and non-structural components, vibration affecting normal use, local damage (including concrete cracks) affecting normal use or durability.

3.1.3 In the design of steel structures, different classes of safety shall be adopted according to the consequence of damage which may be caused by a structural failure.

Steel structures of industrial and civil buildings, in general, shall be taken as safety class 2, whereas for a special building structure the safety class shall be dealt with individually in conformity to its actual condition.

3.1.4 In designing a steel structure according to the ultimate limit state, the basic combination of load effects shall be considered and, if necessary, the accidental combination of load effects shall also be considered.

In designing a steel structure according to the serviceability limit state, the normal combination of load effects shall be considered, whereas for composite steel and concrete beams, the quasi-permanent combination shall also be considered.

3.1.5 In checking the strength and stability of structures or structural members and also the strength of connections, the design value of loads shall be used (i.e. the characteristic value of loads multiplied by partial safety factor for loads), whereas in checking fatigue, the characteristic value of loads shall be used.

3.1.6 For checking the strength and stability of structures subjected to direct dynamic loading, the design value of the dynamic load shall be multiplied by an dynamic factor, whereas the characteristic value of the dynamic load without dynamic factor shall be used in checking fatigue and deformation.

In the calculation of fatigue and deflection of crane girders or crane trusses together with their surge girders, the crane load shall be determined by one of the cranes of largest loading effect in the bay.

3.2 Load and calculation of load effects

3.2.1 In the design of steel structures, the characteristic value of loads, the partial safety factor for loads, the load combination coefficient, the dynamic factor of dynamic loads shall comply with the requirements of the current national standard "Load code for the design of building structures" GB 50009.

The importance factor of structures γ_0 shall comply with the current national standard "Unified standard for reliability design of building structures" GB 50068. Among others, γ_0 for structural members with design working life of 25 years shall not be less than 0.95.

Note: For members or structures supporting light roofing (purlins, roof trusses, frames, etc.), the characteristic value of uniform roof live load shall be taken as 0.3kN/m² when only one variable load is acting and that the horizontal projection of the loaded area exceeds 60m².

3.2.2 In checking the strength and stability of crane girders (or trusses) for heavy duty cranes and the associated surge girders, and also in checking the strength of their connections (mutual connections between crane girders or trusses, surge girders and columns), a horizontal transverse force generated by sway of cranes shall be taken into account. The characteristic value of this horizontal force acting at each wheel of the crane should be the following:

$$H_k = \alpha P_{k,max} \qquad (3.2.2)$$

where $P_{k,max}$——characteristic value of maximum wheel load from the crane;

α——coefficient, $\alpha = 0.1$ for regular cranes with flexible hook, $\alpha = 0.15$ for grab cranes and magnetic disc cranes and $\alpha = 0.2$ for cranes with rigid hook.

Note: The current national standard "Code for design of cranes" GB/T 3811 classifies cranes into A1 through A8 categories according to their working grade. Generally speaking, light duty regime in this Code corresponds to categories A1~A3; medium duty regime corresponds to categories A4 and A5; heavy duty regime corresponds to categories A6~A8, while A8 belongs to extra-heavy duty.

3.2.3 In checking roof trusses with suspended cranes and electric hoisting tackles, the number of hoisting equipment on each operation route in one bay should not be more than two for girder crane and not more than one for electric tackle.

3.2.4 In calculating working platform structures of metallurgical workshop or of similar workshops, the loading caused by repairing materials may be multiplied by a reduction factor:

0.85 for main girders;

0.75 for columns (including foundation).

3.2.5 The structure calculation model and basic assumptions shall comply with the actual behavior of members and connections as far as possible.

3.2.6 The internal forces of building structures are generally determined by elastic analysis according to structural statics; statically indetermined structures, satisfying the requirements of Chapter 9 of this Code, may adopt plastic analysis. Structures analyzed elastically are allowed to develop plastic deformation in member sections.

3.2.7 In framed structures, a rigid beam-to-column connection shall be consistent with the assumption that the angle between intersecting members remains unchanged under loading and that the joints are strong enough to bear all the most unfavorable internal forces transmitted to the ends of intersecting members. A hinged beam-to-column connection shall have adequate rotation capacity and can effectively transmit transverse shear and axial force. A semi-rigid beam-to-column connection has only limited rotational stiffness and the angle between intersecting members varies at the onset of bending moment. The moment-rotation characteristic curve of the connection must be determined beforehand in analyzing internal forces in order to account for the effect of connection deformation.

3.2.8 The internal force analysis of framed structures should comply with the following:

1 First-order elastic analysis may be used for framed structures.

2 Second-order elastic analysis should be used for those framed structures where the inequality $\frac{\sum N \cdot \Delta u}{\sum H \cdot h} > 0.1$ holds. In this situation, notional horizontal forces H_{ni} calculated with Formula(3.2.8-1), shall be applied at the column top of each storey:

$$H_{ni} = \frac{\alpha_y Q_i}{250}\sqrt{0.2 + \frac{1}{n_s}} \qquad (3.2.8\text{-}1)$$

where Q_i——design value of total gravity load at the i-th storey top;

n_s——number of stories in the frame; when $\sqrt{0.2 + 1/n_s} > 1$, take this square root equal to 1;

α_y——factor of steel strength effect, taken as 1.0 for Q235 steel; 1.1 for Q345 steel; 1.2 for Q390 steel and 1.25 for Q 420 steel.

For pure frames without bracing, the second order elastic moment at member ends may be calculated by the following approximate formula:

$$M_{\mathrm{II}} = M_{\mathrm{I b}} + \alpha_{2i} M_{\mathrm{I s}} \qquad (3.2.8\text{-}2)$$

$$\alpha_{2i} = \frac{1}{1 - \frac{\sum N \cdot \Delta u}{\sum H \cdot h}} \qquad (3.2.8\text{-}3)$$

where $M_{\mathrm{I b}}$—— moment at member ends given by first order elastic analysis assuming null sidesway of the frame;

$M_{\mathrm{I s}}$——moment at member ends due to sidesway of frame joints according to first order elastic analysis;

α_{2i}—— amplifying factor of sway moment of the i-th storey members taking account for second order effect;

ΣN—— sum of design values of column axial compression in the calculated storey;

ΣH—— sum of horizontal forces of the calculated storey and storey above generating the interstorey lateral deflection Δu;

Δu—— interstorey lateral deflection of the calculated storey according to first-order analysis, the allowable storey drift[Δu]may be used for Δu as an approximation when determining the necessity of adopting second-order analysis;

h—— height of the calculated storey.

Note: 1 The stiffness of the framed structures should be enlarged when $\alpha_{2i} > 1.33$ is obtained from formula (3.2.8-3).

2 This provision does not apply to the pitched portal frame and the like , nor to framed structures designed by plastic analysis according to Chapter 9 of this Code.

3.3 Material selection

3.3.1 The grade and quality of steel used for load-carrying structures shall be selected in taking comprehensive account of the importance, the loading characteristic, the structural type, the stress state, the connection device, the thickness of steel and the working circumstance of the structure etc..

The material of load-carrying structures may be steel of grades Q235, Q345, Q390 and Q420. Their quality shall conform respectively to the requirements of the current national standards "Carbon structural steels" GB/T 700 and "High strength low-alloy structural steels" GB/T 1591. Steel of other grades , if adopted for use, shall furthermore conform to the requirements of the relevant standards.

3.3.2 Grade Q235 rimmed steel shall not be used for the following load-carrying structures and members.

1 Welded structures

1) Structures subject to direct dynamic or vibrational load requiring fatigue check.

2) Structures subject to direct dynamic or vibrational load but not requiring fatigue check and important structures subject to bending and tension under static load, in case their working temperature is lower than -20℃.

3) All kinds of load-carrying structure whose working temperature is equal to or lower than -30℃.

2 Non-welded structures: structures subject to direct dynamic load requiring fatigue check, whose working temperature is equal to or lower than -20℃.

3.3.3 Steel for load-carrying structures shall be guaranteed for meeting the requirements of tensile strength, percentage of elongation, yield strength (or yield point) and also of proper sulfur and phosphorus contents. For welded structures, the steel shall also be

guaranteed for proper carbon content.

The steel for welded and important non-welded load-carrying structures shall also be guaranteed for passing the cold-bending test.

3.3.4 Steel used for welded structures requiring fatigue check shall be guaranteed for meeting the requirement of notch toughness at normal temperature. But, in case the working temperature is not higher than 0℃ but higher than −20℃, Q235 and Q345 steel shall be guaranteed for meeting the requirement of notch toughness at the temperature 0℃, whereas steels Q390 and Q420 shall be guaranteed for meeting the requirement of notch toughness at the temperature of −20℃. In case the working temperature is not higher than −20℃, the notch toughness test shall be performed at −20℃ for Q235 and Q345 and at −40℃ for Q390 and Q420 steels.

Steel used for non-welded structures, requiring fatigue check shall also be guaranteed for the requirement of notch toughness at normal temperature. In case the working temperature is not higher than −20℃, the notch toughness test shall be performed at 0℃ for Q235 and Q345 steels and at −20℃ for Q390 and Q420 steels.

Note: crane girders for crane of medium duty with capacity not less than 50t shall meet the notch toughness requirement identical to members requiring fatigue checking.

3.3.5 The steel quality for cast steel parts shall conform to the current national standard, "Carbon steel castings for general engineering purpose" GB/T 11352.

3.3.6 When welded load-carrying structures adopt Z-direction steel for preventing lamellar tearing, the steel quality shall conform to the current national standards "Steel plate with through-thickness characteristics" GB/T 5313.

3.3.7 Load-carrying structures, exposed to atmospheric circumstance and requiring specific corrosion protection, or exposed to corrosive gaseous and solid medium, should use weathering steel whose quality shall conform to the requirements of the current national standard "Atmospheric corrosion resisting steel for weld structures" GB/T 4172.

3.3.8 The connection material of steel structures shall comply with the following requirements:

1 The electrodes used for manual welding shall meet the requirements of the current national standard, "Carbon steel covered electrodes" GB/T 5117, or "Low-alloy steel covered electrodes" GB/T 5118. The type of selected electrodes shall match the base metal in mechanical properties. For structures subject to direct dynamic or vibrational load and requiring fatigue check, low-hydrogen electrodes should be used.

2 The wire and flux used for automatic or semi-automatic welding shall match the base metal in mechanical properties and meet the requirements of the relevant current national standards.

3 Ordinary bolts shall meet the requirements of the current national standard "Hexagon head bolts -product grade C" GB/T 5780 and "Hexagon head bolts" GB/T 5782.

4 High-strength bolts shall meet the requirements of the current national standards "High strength bolts with large hexagon head for steel structures" GB/T 1228, "High strength large hexagon nuts for steel structures"GB/T 1229, "High strength plain washers for steel structures "GB/T 1230, "Specification of high strength bolts with large hexagon head, large hexagon nuts, plain washers for steel structures"GB/T 1231 or "Sets of torshear type high strength bolt, hexagon nut and plain washer for steel structures"GB/T 3632, "Technical requirement for set of torshear type high strength bolt, hexagon nut and plain washers for steel structures"GB/T 3633.

5 The material of cheese head stud connector shall meet the requirements of the current national standard "Cheese head studs for arc stud welding "GB/T 10433.

6 Rivets shall be made of BL2 or BL3 steel specified in current national standard "Hot-rolled carbon steel bars for standard parts"GB/T 715.

7 Anchor bolts may be made of Q235 steel or Q345 steel specified respectively in the current national standards "Carbon structural steel"GB/T 700 or "High strength low-alloy structure steel"GB/T 1591.

3.4 Design indices

3.4.1 The design value of steel strength shall be taken from Table 3.4.1-1 according to the steel thickness or diameter. The design value of strength of cast steel parts shall be taken from Table 3.4.1-2. The design value of connection strength shall be taken from Tables 3.4.1-3 through 3.4.1-5.

Table 3.4.1-1 Design value of steel strength (N/mm^2)

Steel		Tension, compression and bending	Shear	End bearing (planed and closely fitted)
Grade	Thickness or diameter (mm)	f	f_v	f_{ce}
Q235	≤16	215	125	325
	>16~40	205	120	
	>40~60	200	115	
	>60~100	190	110	
Q345	≤16	310	180	400
	>16~35	295	170	
	>35~50	265	155	
	>50~100	250	145	
Q390	≤16	350	205	415
	>16~35	335	190	
	>35~50	315	180	
	>50~100	295	170	
Q420	≤16	380	220	440
	>16~35	360	210	
	>35~50	340	195	
	>50~100	325	185	

Note: Thickness in this table denotes the steel thickness at the calculation location, for members subject to axial force, it is the thickness of the thicker plate element of the section.

Table 3.4.1-2 Design value of cast steel strength (N/mm^2)

Steel grade	Tension, compression and bending f	Shear f_v	End bearing (planed and closely fitted) f_{ce}
ZG200-400	155	90	260
ZG230-450	180	105	290
ZG270-500	210	120	325
ZG310-570	240	140	370

Table 3.4.1-3 Design value of weld strength (N/mm^2)

Method of welding and type of electrode	Member material		Butt weld				Fillet weld
	Steel grade	Thickness or diameter (mm)	Compression f_c^w	Tension, f_t^w, for weld quality of class		Shear f_v^w	Tension, compression and shear f_f^w
				1 and 2	3		
Automatic, semi-automatic welding and manual welding with E43 type electrode	Q235	≤16	215	215	185	125	160
		>16~40	205	205	175	120	
		>40~60	200	200	170	115	
		>60~100	190	190	160	110	
Automatic, and semi-automatic welding and manual welding with E50 type electrode	Q345	≤16	310	310	265	180	200
		>16~35	295	295	250	170	
		>35~50	265	265	225	155	
		>50~100	250	250	210	145	
Automatic, and semi-automatic welding and manual welding with E55 type electrode	Q390	≤16	350	350	300	205	220
		>16~35	335	335	285	190	
		>35~50	315	315	270	180	
		>50~100	295	295	250	170	
	Q420	≤16	380	380	320	220	220
		>16~35	360	360	305	210	
		>35~50	340	340	290	195	
		>50~100	325	325	275	185	

Note: 1 The electrode wire and flux used for auromatic and semi-automatic welding shall be guaranteed that the mechanical properties of the deposited metal is not lower than the stipulations of the current national standards "Carbon steel electrodes and fluxes for submerged arc welding" GB/T 5293 and "Fluxes for the submerged arc welding of low alloy steel" GB/T 12470.

2 The weld quality class shall comply with the stipulations of the current national standard "Code for acceptance of construction quality of steel structures" GB 50205. For butt welds of steel components thinner than 8mm ultrasonic flaw detector shall not be used to determine the weld quality class.

3 For butt welds subject to flexion, take f_c^w as the design value of strength in compression zone and f_t^w in tension zone.

4 "Thickness" in this table denotes the steel thickness at the location of calculation. For members in axial tension and axial compression it is the thickness of the thicker plate element of the section.

Table 3.4.1-4 Design value of bolted connection strength (N/mm^2)

Property grade of bolts, steel grade of anchor bolts and members		Ordinary bolts						Anchor bolts	High strength bolts in bearing type joint		
		Grade C			Grade A, B						
		Tension f_t^b	Shear f_v^b	Bearing f_c^b	Tension f_t^b	Shear f_v^b	Bearing f_c^b	Tension f_t^a	Tension f_t^b	Shear f_v^b	Bearing f_c^b
Ordinary bolts	4.6, 4.8	170	140	—	—	—	—	—	—	—	—
	5.6	—	—	—	210	190	—	—	—	—	—
	8.8	—	—	—	400	320	—	—	—	—	—
Anchor bolts	Q235	—	—	—	—	—	—	140	—	—	—
	Q345	—	—	—	—	—	—	180	—	—	—
High strength bolts in bearing type joint	8.8	—	—	—	—	—	—	—	400	250	—
	10.9	—	—	—	—	—	—	—	500	310	—
Members	Q235	—	—	305	—	—	405	—	—	—	470
	Q345	—	—	385	—	—	510	—	—	—	590
	Q390	—	—	400	—	—	530	—	—	—	615
	Q420	—	—	425	—	—	560	—	—	—	655

Note: 1 Grade A bolts are used for bolts with $d \leqslant 24$mm and $l \leqslant 10d$ or $l \leqslant 150$mm (take the lesser value); grade B bolts are used for bolts with either $d > 24$mm or $l > 10\ d$ or $l > 150$mm (take the lesser value). d is the nominal diameter. l is the nominal length of bolt shank.

2 The precision and the surface roughness of holes of grade A, B bolts and the tolerance and surface roughness of holes of grade C bolts shall meet the requirements of the current national standard "Code for acceptance of construction quality of steel structures" GB 50205.

Table 3.4.1-5 Design value of riveted connection strength (N/mm^2)

Steel grade of rivet and member		Tension (breaking of rivet head) f_t^r	Shear f_v^r		Bearing f_c^r	
			Class Ⅰ holes	Class Ⅱ holes	Class Ⅰ holes	Class Ⅱ holes
Rivet	BL2 or BL3	120	185	155	—	—
Member	Q235	—	—	—	450	365
	Q345	—	—	—	565	460
	Q390	—	—	—	590	480

Note: 1 Holes made by the following processes belong to class Ⅰ:

1) Holes drilled to the design diameter on assembled members;

2) Holes drilled to the design diameter separately on individual elements and members by using drilling template;

3) Holes drilled or punched to a smaller diameter on individual elements and reamed afterwards to the design diameter on assembled member.

2 Holes punched or drilled without template to the design diameter on individual elements belong to class Ⅱ.

3.4.2 The design value of strength specified in Clause 3.4.1 shall be multiplied by a relevant reduction factor in the following situations of member and connection calculation:

1 Single angle connected by one leg

1) **For checking member and connection strength as axially loaded, multiply by 0.85;**

2) **For checking stability as an axial compression member:**

Equal leg angles, multiply by $0.6 + 0.0015\lambda$, but not larger than 1.0; Unequal leg angles connected by short leg, multiply by $0.5 + 0.0025\lambda$, but not larger than 1.0; Unequal leg angles connected by long leg, multiply by 0.7; where λ is the slenderness ratio, which shall be determined by the least radius of gyration for a single angle compression member without intermediate connection. Assume $\lambda = 20$ when $\lambda < 20$.

2 **Butt weld performed by welding from one side without backing plate, multiply by 0.85;**

3 **Welded and riveted erection connections made high above the ground in unfavorable conditions, multiply by 0.9;**

4 **Countersunk and semicountersunk riveted connection, multiply by 0.8.**

Note: When several of these situations occur simultaneously, the relevant reduction factors shall be multiplied successively.

3.4.3 The indices of physical properties of rolled and cast steel shall be taken acording to Table 3.4.3.

Table 3.4.3 Indices of physical properties of rolled and cast steel

Modulus of elasticity E(N/mm^2)	Shear deformation modulus G(N/mm^2)	Coefficient of linear expansion α(per℃)	Density ρ (kg/m^3)
206×10^3	79×10^3	12×10^{-6}	7850

3.5 Provisions for deformation of structures and structural members

3.5.1 In order not to impair the serviceability, nor to affect the appearance of structures and structural members, their deformation (deflection or lateral drift) shall comply with the relevant limiting values in designing. The allowable values of deformation, as a general rule, are specified in Appendix A of this Code. The values therein may be suitably modified in consideration of practical experiences or to meet a specific demand, provided the serviceability is not impaired nor the appearance affected.

3.5.2 Reduction of sectional area by bolt (or rivet) holes may not be taken into account in the deformation calculation of steel structures and members.

3.5.3 In order to improve the appearance and the service condition, members subject to transverse forces may be given a predetermined camber, whose magnitude shall be set according to practical need and usually taken as the deflection caused by the unfactored dead load plus one half unfactored live load. In the case of solely improving the appearance, the member deflection shall be taken as that calculated from the unfactored dead and live load and minus the camber.

4 Calculation of flexural members

4.1 Strength

4.1.1 The bending strength of solid web members bent in their principal planes shall be checked as follows (for members taking account of web post-buckling strength see Clause 4.4.1 of this Code):

$$\frac{M_x}{\gamma_x W_{nx}} + \frac{M_y}{\gamma_y W_{ny}} \leqslant f \tag{4.1.1}$$

where M_x, M_y——bending moments about x-and y-axes at a common section (for I-section, x-axis is the strong axis and y is the weak axis);

W_{nx}, W_{ny}——net section moduli about x-and y-axes;

γ_x, γ_y——plasticity adaptation factors, $\gamma_x = 1.05$, $\gamma_y = 1.20$ for I-section, $\gamma_x, \gamma_y = 1.05$ for box section, see Table 5.2.1 for other sections;

f——design value of bending strength of steel.

When the ratio of the free outstand of the compression flange to its thickness is larger than $13\sqrt{235/f_y}$, but not exceeding $15\sqrt{235/f_y}$, γ_x shall be taken as 1.0. f_y is the yield strength of the material indicated by the steel grade.

For beams requiring fatigue checking, $\gamma_x = \gamma_y = 1.0$ should be used.

4.1.2 The shear strength of solid web members bent in their principal plane shall be checked by the following formula (for members taking account of web post-buckling strength, see Clause 4.4.1 of this Code):

$$\tau = \frac{VS}{It_w} \leqslant f_v \tag{4.1.2}$$

where V——shear force in the calculated section along the plane of web;

S——first moment about neutral axis of that part of the gross section above the location where shear stress is calculated;

I——moment of inertia of gross section;

t_w——web thickness;

f_v——design value of shear strength of steel.

4.1.3 When a concentrated load is acting along the web plane on the upper flange of the beam, and that no bearing stiffener is provided at the loading location, the local compressive stress of the web at the upper edge of its effective depth shall be computed as follows:

$$\sigma_c = \frac{\psi F}{t_w l_z} \leqslant f \tag{4.1.3-1}$$

where F——concentrated load, taking into account the dynamic factor in case of dynamic loading;

ψ——amplification coefficient of the concentrated load, $\psi = 1.35$ for heavy duty crane girder; $\psi = 1.0$ for other beams and girders;

l_z——assumed distribution length of the concentrated load on the upper edge of the effective web depth taken as:

$$l_z = a + 5h_y + 2h_R \tag{4.1.3-2}$$

a——bearing length of the concentrated load along the beam span, taken as 50mm for wheel loading on rail;

h_y——distance from the top of girders or beams to the upper edge of the effective web depth;

h_R——depth of the rail, $h_R = 0$ for beams without rail on top;

f——design value of compressive strength of steel.

At the beam support, when no bearing stiffener is provided, the local compressive stress in the web at its lower edge of effective depth shall also be checked by Formula (4.1.3-1), with ψ take as 1.0. The distribution length of the end reaction shall be determined with reference to Formula(4.1.3-2) and according to the dimensions of the support.

Note: The effective web depth h_0 is:

For rolled beams: the distance between the web toes of the fillets joining the web with the upper and lower flanges;

For welded girders: the depth of the web;

For riveted (or high-strength bolted) girders: the distance between the nearest gauge lines of rivets (or high-strength bolts) connecting the web with the upper and lower flanges (see Fig. 4.3.2).

4.1.4 In case comparatively large normal stress σ, shear stress τ, and local compressive stress σ_c(or comparatively large σ and τ)exist simultaneously at the edge of the effective web depth of build-up girders, e.g. at the intermediate support of a continuous girder or at a section where the flange changes its dimensions, the reduced stress shall be checked by the following expression:

$$\sqrt{\sigma^2 + \sigma_c^2 - \sigma\sigma_c + 3\tau^2} \leqslant \beta_1 f \tag{4.1.4-1}$$

where σ, τ, σ_c——normal stress, shear stress and local compressive stress occurring simultaneously at a same point on the edge of effective web depth. τ and σ_c are calculated by Formulae (4.1.2) and (4.1.3-1) respectively, while σ is determined as follows:

$$\sigma = \frac{M}{I_n} y_1 \tag{4.1.4-2}$$

σ and σ_c are taken as positive while being tensile and negative while compressive;

I_n——moment of inertia of the net beam section;

y_1——distance from the calculated point to the neutral axis of the beam section;

β_1——amplification coefficient of design value of strength for reduced stress, $\beta_1 = 1.2$ when σ and σ_c are of different signs, $\beta_1 = 1.1$ when σ and σ_c are of the same sign or when $\sigma_c = 0$.

4.2 Overall stability

4.2.1 Calculation of the overall stability of the beams may not be needed when one of the following situations takes place:

1 A rigid decking (reinforced concrete slab or steel plate) is securely connected to the compression flange of the beam and capable of preventing its lateral deflection;

2 The ratio of the unsupported length, l_1, of the compression flange of a simply supported rolled H- or uniform I-section beam to its flange width, b_1, does not exceed the values given in Table 4.2.1.

Table 4.2.1 Maximum l_1/b_1 values of simply supported rolled H- or uniform I-section beams to avoid checking for overall stability

Steel grade	Beams without intermediate lateral support, load acting at		Beams with intermediate lateral support, load acting anywhere
	the upper flange	the lower flange	
Q235	13.0	20.0	16.0
Q345	10.5	16.5	13.0
Q390	10.0	15.5	12.5
Q420	9.5	15.0	12.0

Note: The maximum l_1/b_1 values of beams made of steel grade other than those shown in Table 4.2.1 shall be that of Q235 steel multiplied by $\sqrt{235/f_y}$.

For beams devoid of lateral support within the span, l_1 is the span length; for those provided with lateral supports within the span, l_1 is the distance between these supports (beam bearings are considered as supports).

4.2.2 Except for the situations specified in Clause 4.2.1, members bent in their principal plane of largest rigidity shall be checked for overall stability as follows:

$$\frac{M_x}{\varphi_b W_x} \leqslant f \tag{4.2.2}$$

where M_x——maximum bending moment about the strong axis;

W_x——gross section modulus of the beam with respect to compression fibers;

φ_b——overall stability factor determined according to Appendix B.

4.2.3 Except for the situations specified in Clause 4.2.1, H-and I-section members bent in their two principal planes shall be checked for overall stability as follows:

$$\frac{M_x}{\varphi_b W_x} + \frac{M_y}{\gamma_y W_y} \leqslant f \tag{4.2.3}$$

where W_x, W_y——gross section moduli about x-and y-axes with respect to compression fibers;

φ_b——overall stability factor for members bent about the strong axis, same as in Clause 4.2.2.

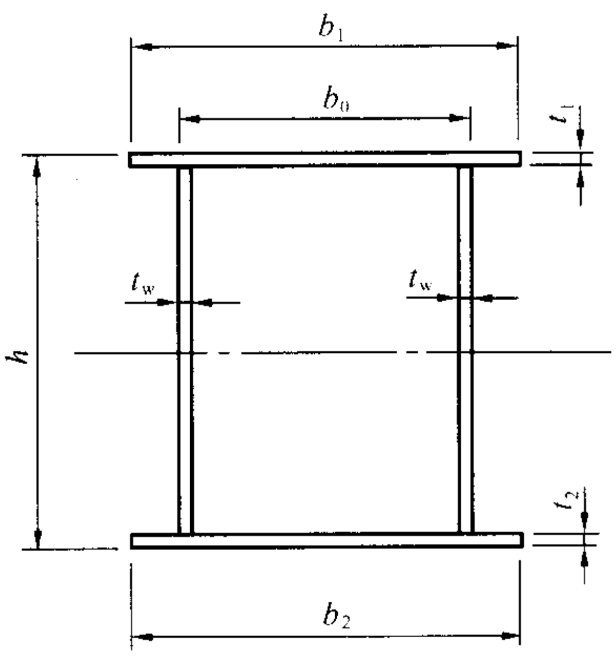


Fig. 4.2.4 Box-section

4.2.4 Simply supported box section beams not conforming to the first situation specified in Clause 4.2.1 shall have their cross section dimension (Fig. 4.2.4) meeting the relationships $h/b_0 \leqslant 6$ and $l_1/b_0 \leqslant 95(235/f_y)$.

Simply supported box section beams fulfilling the above requirement may not be checked for overall stability.

4.2.5 Detailing measures shall be taken to prevent twisting of the section at beam end supports.

4.2.6 Lateral bracings for reducing the unsupported length of the compression flange of a beam shall have their axial force determined in accordance to Clause 5.1.7 by considering the compression flange as an axially loaded compression member.

4.3 Local stability

4.3.1 Built-up girders subject to static or indirect dynamic loading should take account of web post-buckling strength with their bending and shear capacities checked in accordance with section 4.4; crane girders and similar nembers subject to direct dynamic loading, or other girders not taking account of post-buckling strength, shall be provided with stiffeners in accordance with Clause 4.3.2. In case $h_0/t_w > 80\sqrt{235/f_y}$, web stability shall be checked according to the requirements of Clauses 4.3.3 through 4.3.5.

In checking the web stability of light and medium duty crane girders, the design value of crane wheel load may be multiplied by a reduction factor 0.9.

4.3.2 Stiffeners shall be provided for webs of built-up girders in accordance with the following provisions (Fig. 4.3.2):

1 When $h_0/t_w \leqslant 80\sqrt{235/f_y}$, transverse stiffeners shall be provided for girders with local comressive stress($\sigma_c \neq 0$) in accordance with detailing requirements, but may not be provided for girders without local compressive stress($\sigma_c = 0$).

2 Transverse stiffeners shall be provided in case $h_0/t_w > 80\sqrt{235/f_y}$, among which, when $h_0/t_w > 170\sqrt{235/f_y}$ (twisting of compression flange is restrained, such as connected with rigid slab, surge plate or welded-on rail) or $h_0/t_w > 150\sqrt{235/f_y}$ (twisting of compression flange not restrained), or demanded by calculation, longitudinal stiffeners shall be added in the compression zone of large flexural stress panels. For girders with considerable local compressive stress, additional short stiffeners should also be provided if necessary.

h_0/t_w shall in no case exceed 250.

In the above, h_0 is the effective web depth (for monosymmetric girders, h_0 shall be taken as twice the height of compression zone h_c in judging whether longitudinal stiffeners are necessary), t_w is the web thickness.

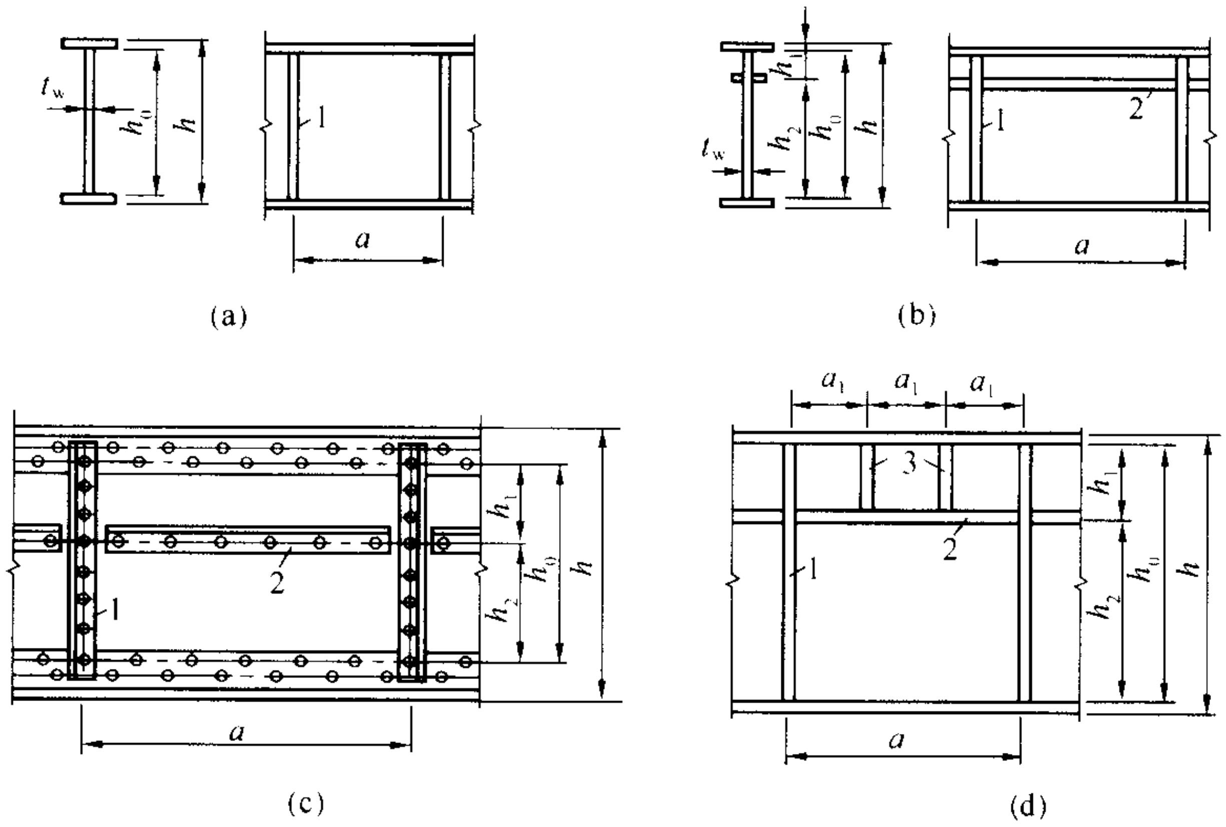


Fig. 4.3.2 Layout of stiffeners

1—transverse stiffeners; 2—longitudinal stiffeners; 3—short stiffeners

3 Bearing stiffeners shall be provided at girder supports and anywhere a fixed and comparatively large concentrated load is applied on the upper flange.

4.3.3 Panels of girder webs provided solely with transverse stiffeners (Fig. 4.3.2a) shall be checked for local stability by the following expression:

$$\left(\frac{\sigma}{\sigma_{cr}}\right)^2 + \left(\frac{\tau}{\tau_{cr}}\right)^2 + \frac{\sigma_c}{\sigma_{c,cr}} \leqslant 1 \tag{4.3.3-1}$$

where σ——bending compressive stress at the edge of effective depth of the web caused by the average bending moment in the calculated web panel;

τ——mean shear stress of the web caused by the average shear force in the calculated web panel, $\tau = V/(h_w t_w)$, h_w being the web depth.

σ_c——local compressive stress at the edge of effective depth of the web, calculated with Formula (4.1.3-1), but taking $\psi = 1.0$;

σ_{cr}, τ_{cr}, $\sigma_{c,cr}$——critical value of bending-, shear- and local compressive stress, acting individually and calculated as follows:

1) σ_{cr} is calculated with the following formulae:

When $\lambda_b \leqslant 0.85$

$$\sigma_{cr} = f \tag{4.3.3-2a}$$

When $0.85<\lambda_b\leqslant 1.25$

$$\sigma_{cr} = [1 - 0.75(\lambda_b - 0.85)]f \tag{4.3.3-2b}$$

When $\lambda_b>1.25$

$$\sigma_{cr} = 1.1f/\lambda_b^2 \tag{4.3.3-2c}$$

where λ_b——normalized depth-thickness ratio for calculation of webs subject to flexion;

When twisting of the girder compression flange is restrained:

$$\lambda_b = \frac{2h_c/t_w}{177}\sqrt{\frac{f_y}{235}} \tag{4.3.3-2d}$$

When twisting of the girder compression flange is not restrained:

$$\lambda_b = \frac{2h_c/t_w}{153}\sqrt{\frac{f_y}{235}} \tag{4.3.3-2e}$$

where h_c——the height of bending compression zone of girder web, $2h_c = h_0$ for doubly symmetric section.

2) τ_{cr} is calculated with the following formulae:

When $\lambda_s\leqslant 0.8$

$$\tau_{cr} = f_v \tag{4.3.3-3a}$$

When $0.8<\lambda_s\leqslant 1.2$

$$\tau_{cr} = [1 - 0.59(\lambda_s - 0.8)]f_v \tag{4.3.3-3b}$$

When $\lambda_s>1.2$

$$\tau_{cr} = 1.1f_v/\lambda_s^2 \tag{4.3.3-3c}$$

where λ_s——normalized depth-thickness ratio for calculation of webs subject to shear.

When $a/h_0\leqslant 1.0$

$$\lambda_s = \frac{h_0/t_w}{41\sqrt{4 + 5.34(h_0/a)^2}}\sqrt{\frac{f_y}{235}} \tag{4.3.3-3d}$$

When $a/h_0 > 1.0$

$$\lambda_s = \frac{h_0/t_w}{41\sqrt{5.34 + 4(h_0/a)^2}}\sqrt{\frac{f_y}{235}} \tag{4.3.3-3e}$$

3) $\sigma_{c,cr}$ is calculated with the following formulae:

When $\lambda_c\leqslant 0.9$

$$\sigma_{c,cr} = f \tag{4.3.3-4a}$$

When $0.9<\lambda_c\leqslant 1.2$

$$\sigma_{c,cr} = [1 - 0.79(\lambda_c - 0.9)]f \tag{4.3.3-4b}$$

When $\lambda_c>1.2$

$$\sigma_{c,cr} = 1.1f/\lambda_c^2 \tag{4.3.3-4c}$$

where λ_c——normalized depth-thickness ratio for calculation of webs under localized compression.

When $0.5\leqslant a/h_0\leqslant 1.5$

$$\lambda_c = \frac{h_0/t_w}{28\sqrt{10.9 + 13.4(1.83 - a/h_0)^3}}\sqrt{\frac{f_y}{235}} \qquad (4.3.3\text{-}4d)$$

When $1.5 < a/h_0 \leqslant 2.0$

$$\lambda_c = \frac{h_0/t_w}{28\sqrt{18.9 - 5a/h_0}}\sqrt{\frac{f_y}{235}} \qquad (4.3.3\text{-}4e)$$

4.3.4 Webs strengthened simultaneously with transverse and longitudinal stiffeners (Fig.4.3.2b,c) shall be checked for local stability by the following expressions:

1 Panels between compression flange and longitudinal stiffener

$$\frac{\sigma}{\sigma_{cr1}} + \left(\frac{\tau}{\tau_{cr1}}\right)^2 + \left(\frac{\sigma_c}{\sigma_{c,cr1}}\right)^2 \leqslant 1.0 \qquad (4.3.4\text{-}1)$$

where σ_{cr1}, τ_{cr1}, $\sigma_{c,cr1}$ are calculated as follows:

1) σ_{cr1} is calculated with formulae (4.3.3-2), but λ_b thereof is replaced by λ_{b1}.

When twisting of the girder compression flange is restrained:

$$\lambda_{b1} = \frac{h_1/t_w}{75}\sqrt{\frac{f_y}{235}} \qquad (4.3.4\text{-}2a)$$

When twisting of the girder compression flange is not restrained:

$$\lambda_{b1} = \frac{h_1/t_w}{64}\sqrt{\frac{f_y}{235}} \qquad (4.3.4\text{-}2b)$$

where h_1——distance from the longitudinal stiffener to the compressive edge of the effective web depth.

2) τ_{cr1} is given by Formulae (4.3.3-3), but replacing h_0 thereof by h_1.

3) $\sigma_{c,cr1}$ is given by Formulae (4.3.3-2), but replacing λ_b thereof by λ_{c1}.

When twisting of the girder compression flange is restrained:

$$\lambda_{c1} = \frac{h_1/t_w}{56}\sqrt{\frac{f_y}{235}} \qquad (4.3.4\text{-}3a)$$

When twisting of the girder compression flange is not restrained:

$$\lambda_{c1} = \frac{h_1/t_w}{40}\sqrt{\frac{f_y}{235}} \qquad (4.3.4\text{-}3b)$$

2 Panels between tension flange and the longitudinal stiffeners:

$$\left(\frac{\sigma_2}{\sigma_{cr2}}\right)^2 + \left(\frac{\tau}{\tau_{cr2}}\right)^2 + \frac{\sigma_{c2}}{\sigma_{c,cr2}} \leqslant 1.0 \qquad (4.3.4\text{-}4)$$

where σ_2——web bending compressive stress at the location of the longitudinal stiffener caused by the average bending moment in the calculated panel;

σ_{c2}——transverse compressive stress of the web at the location of longitudinal stiffener, taken as $0.3\,\sigma_c$.

1) σ_{cr2} is given by formulae (4.3.3-2), but replacing λ_b thereof by λ_{b2}.

$$\lambda_{b2} = \frac{h_2/t_w}{194}\sqrt{\frac{f_y}{235}} \tag{4.3.4-5}$$

2) τ_{cr2} is given by Formulae (4.3.3-3), but replacing h_0 thereof by $h_2(h_2 = h_0 - h_1)$.

3) $\sigma_{c,cr2}$ is given by Formulae(4.3.3-4), but replacing h_0 thereof by h_2, take $a/h_2 = 2$ when $a/h_2 > 2$.

4.3.5 Local stability of panels provided with short stiffeners between compression flange and longitudinal stiffeners (Fig.4.3.2d) is checked by Formula(4.3.4-1), where σ_{cr1} is calculated in accordance with paragraph 1) in section 1 of Clause 4.3.4; τ_{cr1} is given by Formulae (4.3.3-3), but replacing h_0 and a thereof by h_1 and a_1 respectively (a_1 is the spacing of short stiffeners); $\sigma_{c,cr1}$ is given by Formulae (4.3.3-2), but replacing λ_b thereof by λ_{c1} given by the following

When twisting of the compression flange is restrained:

$$\lambda_{c1} = \frac{a_1/t_w}{87}\sqrt{\frac{f_y}{235}} \tag{4.3.5a}$$

When twisting of the compression flange is not restrained:

$$\lambda_{c1} = \frac{a_1/t_w}{73}\sqrt{\frac{f_y}{235}} \tag{4.3.5b}$$

For panels with $a_1/h_1 > 1.2$, the right side of Eqs (4.3.5) shall be multiplied by $1/(0.4 + 0.5a_1/h_1)^{\frac{1}{2}}$.

4.3.6 Stiffeners should preferably be placed in pairs on each side of the web. Stiffeners on one side of the web are also allowed except for bearing stiffeners and stiffeners of heavy duty crane girders.

The minimum spacing of the transverse stiffeners is $0.5h_0$, the maximum spacing is $2h_0$ ($2.5h_0$ may be used for girders without local compressive stress when $h_0/t_w \leqslant 100$). Longitudinal stiffeners shall be located within a distance $h_c/2.5 \sim h_c/2$ from the compressive edge of the web effective depth.

Transverse stiffeners in pairs, made of flats, shall satisfy the following requirements:

outstanding width:

$$b_s \geqslant \frac{h_0}{30} + 40 \quad (\text{mm}) \tag{4.3.6-1}$$

thickness

$$t_s \geqslant \frac{b_s}{15} \tag{4.3.6-2}$$

The outstanding width of transverse stiffeners on one side shall be larger than 1.2 times that obtained from Formula (4.3.6-1). Its thickness shall not be less than 1/15 of the outstanding width.

For webs strengthened simultaneously by transverse and longitudinal stiffeners, the dimensions of the transverse stiffener shall not only meet the above requirements but its moment of inertia, I_z, shall also conform to the following requirement:

$$I_z \geqslant 3h_0 t_w^3 \tag{4.3.6-3}$$

The moment of inertia of the longitudinal stiffener shall satisfy the following requirement:

$$I_y \geqslant 1.5h_0 t_w^3, \text{when} \frac{a}{h_0} \leqslant 0.85 \tag{4.3.6-4a}$$

$$I_y \geqslant \left(2.5 - 0.45\frac{a}{h_0}\right)\left(\frac{a}{h_0}\right)^2 h_0 t_w^3, \text{when} \frac{a}{h_0} > 0.85 \tag{4.3.6-4b}$$

The minimum spacing of short stiffeners is $0.75h_1$. Their outstanding width shall be 0.7~1.0 times that of the transverse stiffeners, their thickness shall not be less than 1/15 of the outstanding width.

Note: 1 Stiffeners made of structural shapes (H-and I-section, channel, angle with leg tip welded to the web) shall have a moment of inertia not less than that of the flat stiffeners.

2 The moment of inertia of stiffeners in pairs shall be computed about the center line of the web.

3 The moment of inertia of stiffeners on one side shall be computed about the edge line of the web where the stiffener is connected.

4.3.7 The bearing stiffeners of girders shall be checked for buckling resistance about the web axis as an axially loaded strut subjected to end reaction or fixed concentrated load. The cross-section of this strut shall comprise that of the stiffener plus a portion of the web $15t_w \sqrt{235/f_y}$ in width on each side of the stiffener. The effective length of the strut shall be taken equal to h_0.

End bearing strength shall be checked in accordance with the end reaction or concentrated fixed load acting thereon when the end of a bearing stiffener is planed and closely fitted (extended flanged stiffener shall furthermore meet the requirement of Clause 8.4.12); weld strength shall be checked according to force transfer when the end is welded.

Weld connecting bearing stiffener to girder web shall be calculated according to the requirement of force transmission.

4.3.8 The ratio of the free outstands of a compression flange, b, to its thickness, t, shall satisfy the following requirement:

$$\frac{b}{t} \leqslant 13\sqrt{\frac{235}{f_y}} \tag{4.3.8-1}$$

b/t may be enlarged to $15\sqrt{235/f_y}$, if the bending strength of the girder is computed with $\gamma_x = 1.0$.

The ratio of the compression flange width, b_0, of a box girder between two web plates to its thickness, t, shall satisfy the following requirement:

$$\frac{b_0}{t} \leqslant 40\sqrt{\frac{235}{f_y}} \tag{4.3.8-2}$$

When longitudinal stiffeners are provided for the compression flange of the box girder, b_0 in Formula (4.3.8-2) shall be the flange unsupported width between web and longitudinal stiffener.

Note: The free outstand b of the flange shall be taken as follows: the distance from the face of the web to the flange tip for welded members; the distance from the toe of the fillet to the flange tip for rolled members.

4.4 Calculation of built-up girder with webs taking account of post-buckling strength

4.4.1 I-section welded girders, whose web has only bearing stiffeners (or has intermediate transverse stiffeners as well), shall be checked for their bending and shear capacity as follows when taking account of post-buckling strength:

$$\left(\frac{V}{0.5V_u}-1\right)^2+\frac{M-M_f}{M_{eu}-M_f}\leqslant 1 \tag{4.4.1-1}$$

$$M_f=\left(A_{f1}\frac{h_1^2}{h_2}+A_{f2}h_2\right)f \tag{4.4.1-2}$$

where M、V——design value of bending moment and shear force occurring simultaneously at a same section of girder, take $V=0.5V_u$ when $V<0.5V_u$ and take $M=M_f$ when $M<M_f$;

M_f——design value of bending moment that the two flanges of the girder are capable to bear;

A_{f1}, h_1——cross-section area of the larger flange and the distance from its centroid to the neutral axis of the girder;

A_{f2}, h_2——cross-section area of the smaller flange and the distance from its centroid to the neutral axis of the girder;

M_{eu}, V_u——design value of bending capacity and shear capacity of the girder respectively.

1 M_{eu} shall be determined from the following:

$$M_{eu}=\gamma_x\alpha_e W_x f \tag{4.4.1-3}$$

$$\alpha_e=1-\frac{(1-\rho)h_c^3t_w}{2I_x} \tag{4.4.1-4}$$

where α_e——reduction factor of section modulus of the girder taking account of the effective height of the web;

I_x——moment inertia about x-axis of girder assuming the whole section effective;

h_c——the height of compressive zone of the web assuming the whole section effective;

γ_x——plasticity adaptation factor of the girder section;

ρ——effective height factor of web compressive zone.

When $\lambda_b \leqslant 0.85$

$$\rho = 1.0 \qquad (4.4.1\text{-}5a)$$

When $0.85 < \lambda_b \leqslant 1.25$

$$\rho = 1 - 0.82(\lambda_b - 0.85) \qquad (4.4.1\text{-}5b)$$

When $\lambda_b > 1.25$

$$\rho = \frac{1}{\lambda_b}\left(1 - \frac{0.2}{\lambda_b}\right) \qquad (4.4.1\text{-}5c)$$

where λ_b—— normalized depth-thickness ratio for calculation of webs subject to flexion, determined by Eqs(4.3.3-2d) and (4.3.3-2e).

2 V_u shall be determined by the following:

When $\lambda_s \leqslant 0.8$

$$V_u = h_w t_w f_v \qquad (4.4.1\text{-}6a)$$

When $0.8 < \lambda_s \leqslant 1.2$

$$V_u = h_w t_w f_v[1 - 0.5(\lambda_s - 0.8)] \qquad (4.4.1\text{-}6b)$$

When $\lambda_s > 1.2$

$$V_u = h_w t_w f_v / \lambda_s^{1.2} \qquad (4.4.1\text{-}6c)$$

where λ_s—— normalized depth-thickness ratio for calculation of webs subject to shear, determined by Eqs(4.3.3-3d) and (4.3.3-3e).

When stiffeners are provided only at the supports of the built-up girder, take $h_0/a = 0$in the Eq (4.3.3-3e).

4.4.2 If the requirement of Eq (4.4.1-1) can not be satisfied by providing stiffeners solely at the supports, intermediate transverse stiffeners shall be added in pair on the two side of the web. Intermediate transverse stiffeners including those subject to concentrate compression on top, besides conforming to the requirement of Eqs(4.3.6-1)and (4.3.6-2), shall be checked as axially loaded strut for buckling resistance out of the web plane with reference to Clause 4.3.7. The axial compression shall be given by the following:

$$N_s = V_u - \tau_{cr} h_w t_w + F \qquad (4.4.2\text{-}1)$$

where V_u——as calculated by Eq(4.4.1-6);

h_w——depth of the web plate;

τ_{cr}——as calculated by Eq(4.3.3-3);

F—— concentrated compression acting at the top of intermediate bearing stiffeners.

When the web panel adjoining the support takes account of the post-buckling strength, that is when $\lambda_s > 0.8$, the stiffener at the support is subject to the horizontal component H of the tension field in addition to the end reaction of the girder and shall be checked for strength and stability out of the web plane as a beam-column.

$$H = (V_u - \tau_{cr} h_w t_w)\sqrt{1 + (a/h_0)^2} \qquad (4.4.2\text{-}2)$$

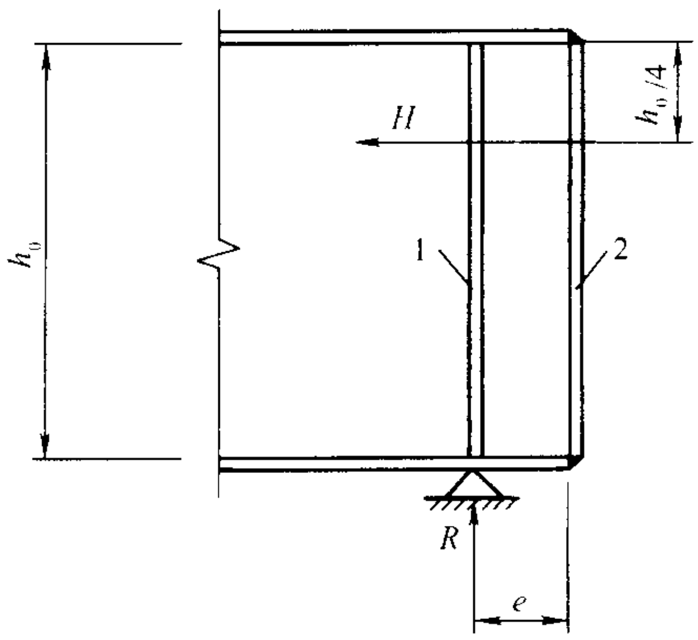


Fig.4.4.2 Twin stiffeners end post of girders

a is equal to the stiffener spacing of the end panel when intermediate transverse stiffeners are provided. a is equal to the distance from the girder support to the section of zero shear when no intermediate stiffeners are provided.

The point of application of H is situated at a distance equal to $h_0/4$ from the upper edge of web plate effective depth. The cross-section and the effective length of this beam-column are the same as common end stiffeners. When twin stiffeners end post is used as shown in Fig.4.4.2, the following simplified method may be adopted: stiffener 1 is checked as an axially loaded strut subject to end reaction R, and the cross-sectional area of the end covering stiffener 2 shall not be less than the following:

$$A_c = \frac{3h_0 H}{16ef} \tag{4.4.2-3}$$

Note: 1 Depth-thickness ratio of the web plate shall not exceed 250.

2 For girders taking account of web post-buckling strength, intermediate transverse stiffeners may be provided in conforming to detailing requirement.

3 Webs with sparse stiffeners ($a > 2.5h_0$) and those without intermediate transverse stiffeners, may take $H = 0$ in case Formula (4.3.3-1) is satisfied.

5 Calculation of axially loaded members and members subjected to combined axial load and bending

5.1 Axially loaded members

5.1.1 The strength of members subject to axial tension or compression, except at high strength bolted friction-type connections, shall be checked as follows:

$$\sigma = \frac{N}{A_n} \leqslant f \tag{5.1.1-1}$$

where N——axial tension or compression;

A_n——net sectional area.

The strength of member at a high-strength bolted friction-type connection shall be checked by the following formulae:

$$\sigma = \left(1 - 0.5\frac{n_1}{n}\right)\frac{N}{A_n} \leqslant f \tag{5.1.1-2}$$

and

$$\sigma = \frac{N}{A} \leqslant f \tag{5.1.1-3}$$

where n——number of high-strength bolts of one end of the member at a joint or a splice;

n_1——number of high-strength bolts on the calculated section (outermost line of bolts);

A——gross sectional area of the member.

5.1.2 The stability of axial compression solid web members shall be checked as follows:

$$\frac{N}{\varphi A} \leqslant f \tag{5.1.2-1}$$

where φ——stability factor of axial compression members (lesser factor about the two principal axes), which shall be taken from Appendix C in accordance with the slenderness ratio, the yield strength and the classification of sections in Tables 5.1.2-1 and 5.1.2-2.

Table 5.1.2-1 Classification of sections of axial compression members (thickness of plate elements $t < 40$mm)

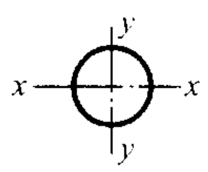
Shape of section	About x-axis	About y-axis
 Rolled	Class a	Class a

Table 5.1.2-1(Continued)

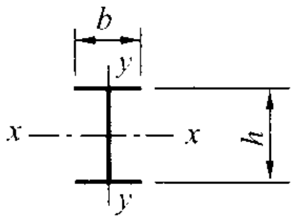
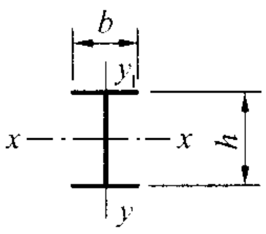
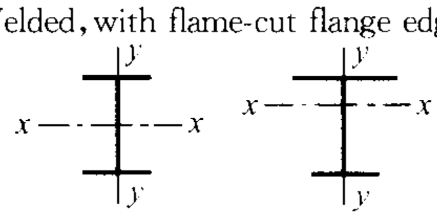
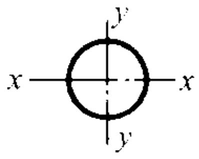
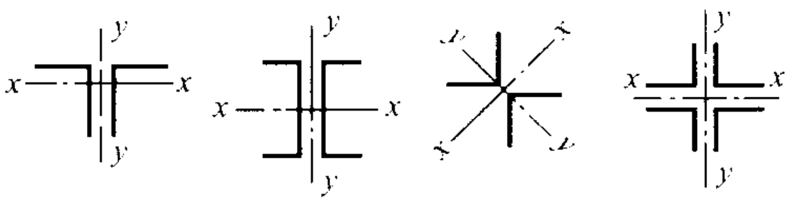
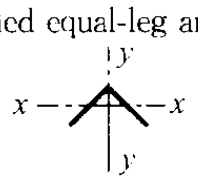
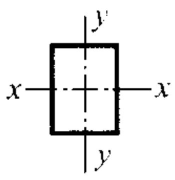
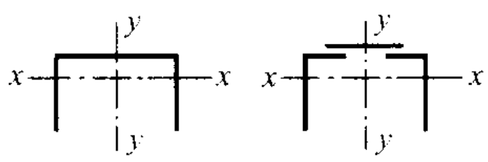
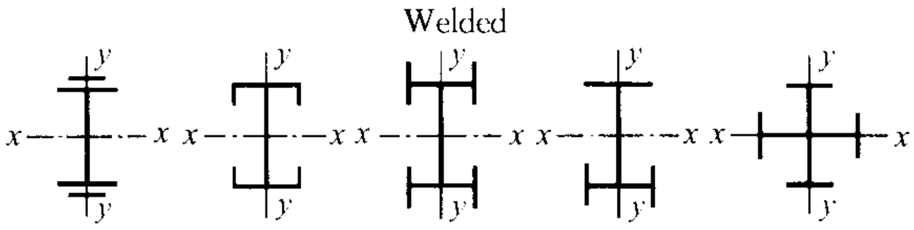
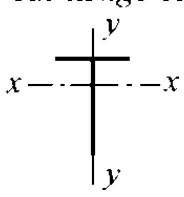
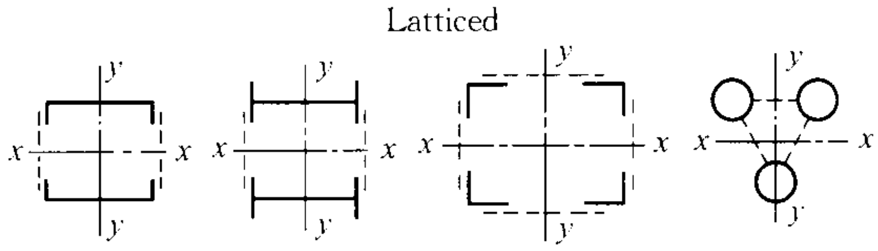
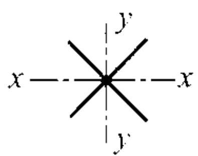
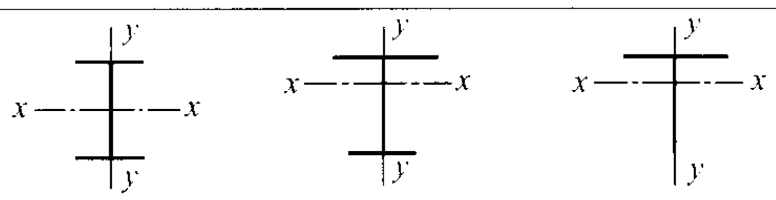
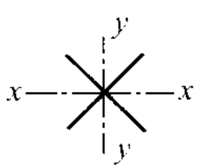
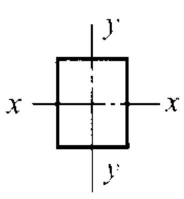
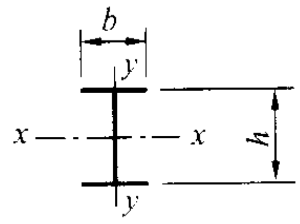
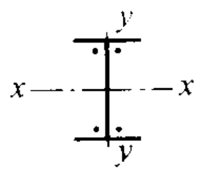
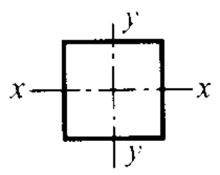
<table>
<tr><th>Shape of section</th><th>About x-axis</th><th>About y-axis</th></tr>
<tr><td> Rolled, $b/h \leqslant 0.8$</td><td>Class a</td><td>Class b</td></tr>
<tr><td>Rolled, $b/h > 0.8$

Welded, with flame-cut flange edges

Welded
</td><td rowspan="6">Class b</td><td rowspan="6">Class b</td></tr>
<tr><td>Rolled

Rolled equal-leg angle
</td></tr>
<tr><td>Rolled, welded (width-thickness ratio of plate elements>20)

Rolled or welded
</td></tr>
<tr><td>Welded

Rolled section and welded section with flame-cut flange edges
</td></tr>
<tr><td>Latticed

Welded, with flame-cut plate edges
</td></tr>
<tr><td></td></tr>
<tr><td> Welded, with rolled or sheared flange edges</td><td>Class b</td><td>Class c</td></tr>
<tr><td> Welded, with plate edges rolled or sheared
 Welded, with wall width-thickness-ratio $\leqslant 20$</td><td>Class c</td><td>Class c</td></tr>
</table>

Table 5.1.2-2 Classification of sections of axial compression members (thickness of plate elements $t \geqslant 40$mm)

Shape of section			About x-axis	About y-axis
	Rolled I- or H-section	$t<80$mm	Class b	Class c
		$t \geqslant 80$mm	Class c	Class d
	Welded I-section	Flame-cut flanges	Class b	Class b
		Rolled or sheared flanges	Class c	Class d
	Welded box section	Wall width-thickness ratio>20	Class b	Class b
		Wall width-thickness ratio$\leqslant 20$	Class c	Class c

The slenderness ratio of members, λ, shall be determined according to the following:

1 For members of doubly symmetric or polar-symmetric section:

$$\lambda_x = l_{0x}/i_x \quad \lambda_y = l_{0y}/i_y \tag{5.1.2-2}$$

where l_{0x}, l_{0y}——member effective length about the principal axis x and y respectively;

i_x, i_y——radius of gyration of member section about principal axis x and y respectively.

For members of doubly symmetric cruciform section, λ_x or λ_y shall not be less than 5.07 b/t (b/t being width-thickness ratio of the outstand).

2 For members of monosymmetric section, λ_x shall be calculated by (5.1.2-2) for checking the buckling resistance about the non-symmetric axis and the following equivalent slenderness ratio shall be used in lieu of λ_y for checking the buckling resistance about the axis of symmetry:

$$\lambda_{yz} = \frac{1}{\sqrt{2}}\left[(\lambda_y^2 + \lambda_z^2) + \sqrt{(\lambda_y^2 + \lambda_z^2)^2 - 4(1 - e_0^2/i_0^2)\lambda_y^2\lambda_z^2}\right]^{1/2} \tag{5.1.2-3}$$

$$\lambda_z^2 = i_0^2 A/(I_t/25.7 + I_\omega/l_\omega^2) \tag{5.1.2-4}$$

$$i_0^2 = e_0^2 + i_x^2 + i_y^2$$

where e_0——distance between centroid and shear centre of the section;

i_0——polar radius of gyration about the shear centre of the section;

λ_y——member slenderness about the axis of symmetry;

λ_z——equivalent slenderness ratio for torsional buckling;

I_t——torsional moment of inertia of the gross section;

I_ω——sectorial moment of inertia of the gross section, may be taken as $I_\omega=0$ for T-sections (rolled, welded with plates or composed of two angles), cruciform and angle sections;

A——area of the gross section;

l_ω——effective length for torsional buckling, take $l_\omega = l_{0y}$ for hinged-hinged members with end sections free to warp or fixed-fixed members with end warping fully restrained.

3 For single angle section and double angle compound T-section, λ_{yz} about the axis of symmetry may be determined by the following simplified approach:

1) Single equal-leg angle section (Fig. 5.1.2a):

When $b/t \leqslant 0.54 l_{0y}/b$

$$\lambda_{yz} = \lambda_y\left(1 + \frac{0.85b^4}{l_{0y}^2 t^2}\right) \tag{5.1.2-5a}$$

When $b/t > 0.54 l_{0y}/b$

$$\lambda_{yz} = 4.78\frac{b}{t}\left(1 + \frac{l_{0y}^2 t^2}{13.5b^4}\right) \tag{5.1.2-5b}$$

where b, t——angle leg width and thickness respectively.

2) Double equal-leg angle section (Fig. 5.1.2b):

When $b/t \leqslant 0.58 l_{0y}/b$

$$\lambda_{yz} = \lambda_y\left(1 + \frac{0.475b^4}{l_{0y}^2 t^2}\right) \tag{5.1.2-6a}$$

When $b/t > 0.58 l_{0y}/b$

$$\lambda_{yz} = 3.9\frac{b}{t}\left(1 + \frac{l_{0y}^2 t^2}{18.6b^4}\right) \tag{5.1.2-6b}$$

3) Double unequal-leg angle section with short leg outstanding (Fig. 5.1.2c):

When $b_2/t \leqslant 0.48 l_{0y}/b_2$

$$\lambda_{yz} = \lambda_y\left(1 + \frac{1.09b_2^4}{l_{0y}^2 t^2}\right) \tag{5.1.2-7a}$$

When $b_2/t > 0.48 l_{0y}/b_2$

$$\lambda_{yz} = 5.1\frac{b_2}{t}\left(1 + \frac{l_{0y}^2 t^2}{17.4b_2^4}\right) \tag{5.1.2-7b}$$

4) Double unequal-leg angle section with long leg outstanding (Fig. 5.1.2d):

Take approximately $\lambda_{yz} = \lambda_y$ when $b_1/t \leqslant 0.56\ l_{0y}/b_1$. If not the case, take

$$\lambda_{yz} = 3.7\frac{b_1}{t}\left(1 + \frac{l_{0y}^2 t^2}{52.7b_1^4}\right)$$

4 Axially compressed members of monosymmetric section buckling about any axis other than the non-symmetric principal axis, shall be checked as flexural-torsional buckling

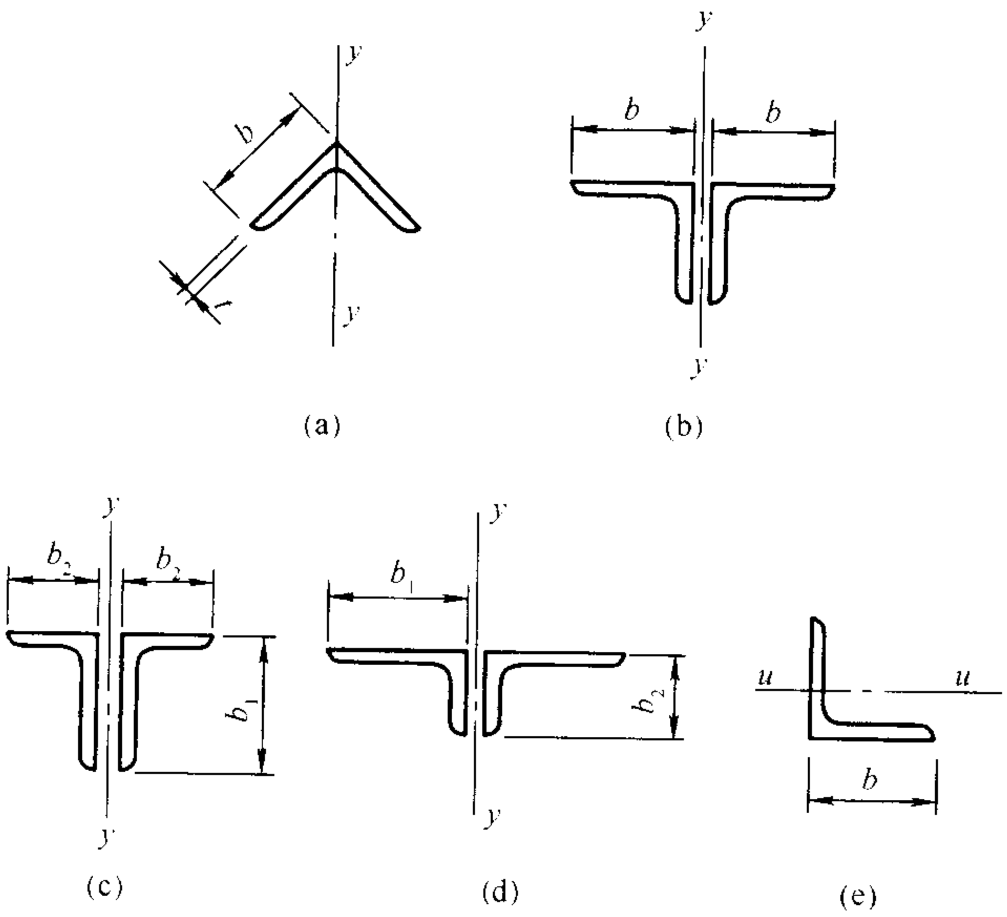


Fig.5.1.2 Single angle section and double angle compound T-section

b—width of equal-leg angle; b_1—longer leg width of unequal-leg angle;

b_2—shorter leg width of unequal-leg angle

problem. For single angle members buckling about an axis parallel to one leg (u-axis in Fig.5.1.2e), the equivalent slenderness ratio λ_{uz} may be calculated by the following, and the φ factor determined as a class b section:

When $b/t \leqslant 0.69 l_{0u}/b$

$$\lambda_{uz} = \lambda_u\left(1 + \frac{0.25b^4}{l_{0u}^2 t^2}\right) \tag{5.1.2-8a}$$

When $b/t > 0.69 l_{0u}/b$

$$\lambda_{uz} = 5.4\frac{b}{t} \tag{5.1.2-8b}$$

where $\lambda_u = l_{0u}/i_u$; l_{0u} is the effective length of the member about the u-axis, i_u is the radius of gyration of the member about the u-axis.

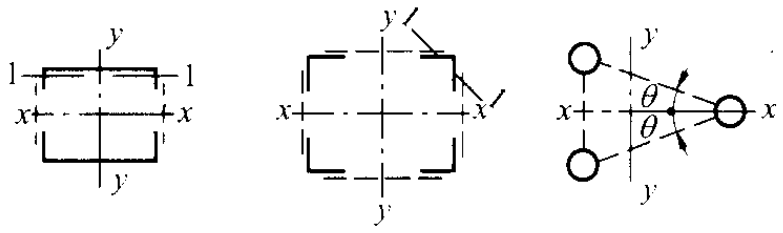
Note : 1 sections having neither axis of symmerty nor pole of symmetry are not suited for axially compressed member (except single unequal leg angles connected by one leg).

2 Torsional effect may not be taken into account for single angle axially compressed members connected by one leg, as long as the design value of strength is multiplied by the reduction factor according to Clause 3.4.2.

3 Channels used as component of laced or battened members, need not take account of the torsional effect when calculating stability about their axis of symmetry (y-axis), and their factor φ_y may be obtained directly by λ_y.

5.1.3 The stability of laced or battened members in axial compression shall still be checked by Formula(5.1.2-1), but using the equivalent slenderness ratio for buckling about the open web axis (x-axis in Fig.5.1.3a and x-and y-axis in Fig.5.1.3b,c).

The equivalent slenderness ratio shall be computed as follows:



(a) (b) (c)

Fig. 5.1.3 Section of laced or battened members

1 Members built-up of two components (Fig. 5.1.3a):

For battened members

$$\lambda_{0x} = \sqrt{\lambda_x^2 + \lambda_1^2} \qquad (5.1.3\text{-}1)$$

For laced members

$$\lambda_{0x} = \sqrt{\lambda_x^2 + 27\frac{A}{A_{1x}}} \qquad (5.1.3\text{-}2)$$

where λ_x——slenderness ratio of the whole member about the x-axis;

λ_1—— slenderness ratio of the components about their weak axis 1-1, the unsupported length taken equal to the clear spacing of battens for welded members, and equal to the distance between end bolts of adjacent battens for bolted members;

A_{1x}——sum of gross sectional area of the lacings perpendicular to the x-axis in a section of the member.

2 Members built-up of four components (Fig. 5.1.3b):

For battened members

$$\lambda_{0x} = \sqrt{\lambda_x^2 + \lambda_1^2} \qquad (5.1.3\text{-}3)$$

$$\lambda_{0y} = \sqrt{\lambda_y^2 + \lambda_1^2} \qquad (5.1.3\text{-}4)$$

For laced members

$$\lambda_{0x} = \sqrt{\lambda_x^2 + \frac{40A}{A_{1x}}} \qquad (5.1.3\text{-}5)$$

$$\lambda_{0y} = \sqrt{\lambda_y^2 + \frac{40A}{A_{1y}}} \qquad (5.1.3\text{-}6)$$

where λ_x——slenderness ratio of the whole member about the y-axis;

A_{1y}——sum of gross sectional area of the lacings perpendicular to the y-axis in a section of the member.

3 Laced members with three components (Fig. 5.1.3c):

$$\lambda_{0x} = \sqrt{\lambda_x^2 + \frac{42A}{A_1(1.5 - \cos^2\theta)}} \qquad (5.1.3\text{-}7)$$

$$\lambda_{0y} = \sqrt{\lambda_y^2 + \frac{42A}{A_1\cos^2\theta}} \qquad (5.1.3\text{-}8)$$

where A_1——sum of gross sectional area of all the lacings in a section of the member;

θ——angle between a lacing plane and the x-axis in a section of the member.

Note: 1 The linear stiffness of the battens shall conform to the requirements of Clause 8.4.1.

2 The angle between the lacing and the member axis shall be within the range of 40°~70°.

5.1.4 The slenderness ratio of the components, λ_1, of laced members in axial compression shall not exceed 0.7 times λ_{max}, which is the larger of slenderness ratios of the members in two directions (the equivalent slenderness ratio shall be taken for open-web axis); λ_1 of battened members in axial compression shall not exceed 40 nor 0.5 λ_{max} (λ_{max} is taken equal to 50 in case it is less than 50).

5.1.5 Members composed of twin angles or twin channels separated by fillers may be considered as solid ones in calculation provided that the spacing of the fillers is less than:

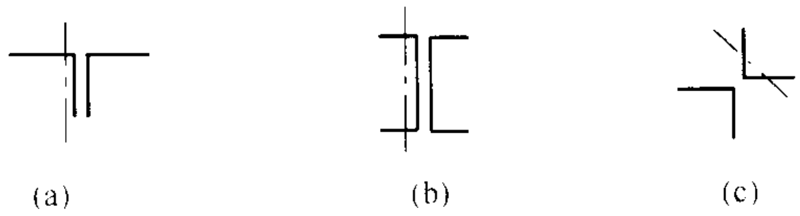


Fig.5.1.5 Axis for computing i

$40i$ for compression members

$80i$ for tension members

where i is the radius of gyration of one component and shall be taken as follows:

1 i is taken about the centroidal axis parallel to the fillers when the twin angles or channels are combined as shown in Fig.5.1.5a,b.

2 i is taken about the weak axis when the two angles are combined in a cruciform section as shown in Fig.5.1.5c.

The number of fillers of a compression member between two adjacent laterally supported points shall not be less than 2.

5.1.6 The shear force of axial compression members shall be computed according to the following:

$$V = \frac{Af}{85}\sqrt{\frac{f_y}{235}} \qquad (5.1.6)$$

The shear force V may be considered as constant along the member length.

The shear force V shall be distributed to relevant lacing or batten planes (also to planes of solid plate element if there exists any) for laced or battened members in axial compression.

5.1.7 Bracing bars, used for reducing the unsupported length of axial compression members, and whose axis passes by the shear centre of the braced members, shall be proportioned to carry an axial force in the direction of buckling by the following:

1 Single brace supporting a single column of height l

For mid-height bracing

$$F_{bl} = N/60 \qquad (5.1.7\text{-}1a)$$

For bracing distant αl from column end ($0<\alpha<1$)

$$F_{bl} = \frac{N}{240\alpha(1-\alpha)} \qquad (5.1.7\text{-}1b)$$

where N—maximum axial compression of the braced member.

2 For multiple equidistant braces supporting a single column of height l (the bracing spacing may be unequal, but the difference between any spacing and the average value does not exceed 20%), the force in each of the m-braces is:

$$F_{\mathrm{bm}} = N/[30(m+1)] \tag{5.1.7-2}$$

3 For mid-height brace supporting a row of columns:

$$F_{\mathrm{bn}} = \frac{\sum N_i}{60}(0.6 + 0.4/n) \tag{5.1.7-3}$$

where n——number of braced columns in the row;

$\sum N_i$——sum of axial compressions acting simultaneously on the braced columns.

4 The brace forces given above may not be additive to the forces in the bracing generated by the effect of other actions on the structure.

5.2 Members subjected to combined axial load and bending

5.2.1 The strength of members subjected to combined axial load and moments acting in principal planes shall be checked as follows:

$$\frac{N}{A_{\mathrm{n}}} \pm \frac{M_{\mathrm{x}}}{\gamma_{\mathrm{x}} W_{\mathrm{nx}}} \pm \frac{M_{\mathrm{y}}}{\gamma_{\mathrm{y}} W_{\mathrm{ny}}} \leqslant f \tag{5.2.1}$$

where γ_{x}, γ_{y}——plasticity adaptation factors relevant to section modulus and given in Table 5.2.1.

Table 5.2.1 Plasticity adaptation factors of cross-sections γ_{x}, γ_{y}

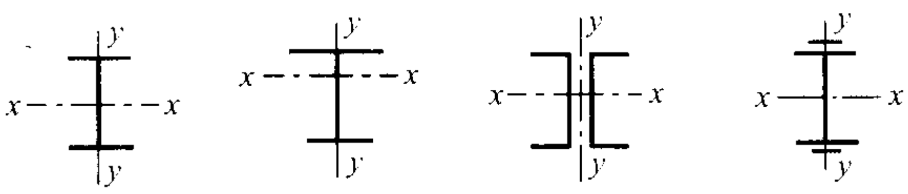
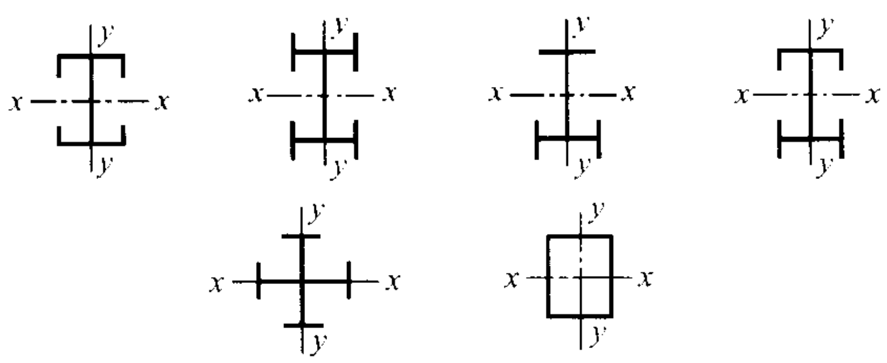
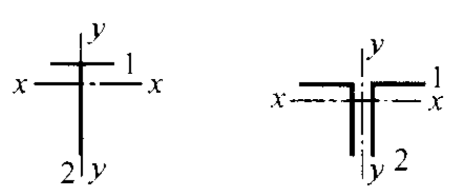
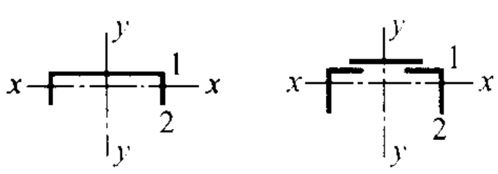
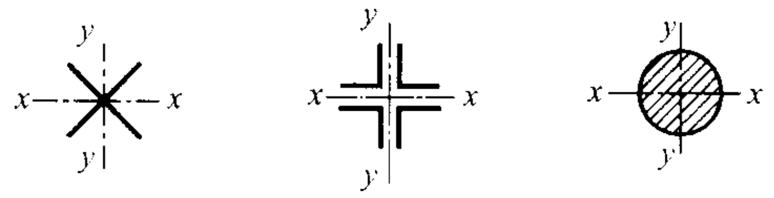
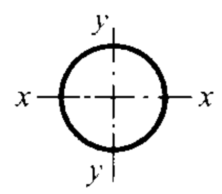
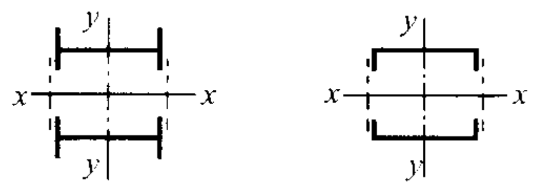
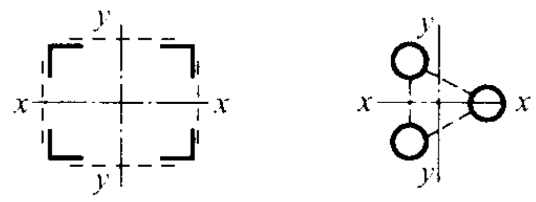
Item No.	Cross-sectional shapes	γ_{x}	γ_{y}
1		1.05	1.2
2			1.05
3		$\gamma_{\mathrm{x1}} = 1.05$ $\gamma_{\mathrm{x2}} = 1.2$	1.2
4			1.05

Table 5.2.1(Continued)

<table>
<tr><th>Item No.</th><th>Cross-sectional shapes</th><th>γ_x</th><th>γ_y</th></tr>
<tr><td>5</td><td></td><td>1.2</td><td>1.2</td></tr>
<tr><td>6</td><td></td><td>1.15</td><td>1.15</td></tr>
<tr><td>7</td><td></td><td rowspan="2">1.0</td><td>1.05</td></tr>
<tr><td>8</td><td></td><td>1.0</td></tr>
</table>

When the ratio of free outstand of the compression flange of a beam-column to its thickness is larger than 13 $\sqrt{135/f_y}$, but not exceeding 15 $\sqrt{235/f_y}$, γ_x shall be taken as 1.0.

For members, subject to axial load and bending, requiring fatigue checking, γ_x and γ_y should be taken as 1.0.

5.2.2 Solid web beam-columns bent in their plane of symmetric axis (about x-axis) shall have their stability checked as follows:

1 In-plane stability:

$$\frac{N}{\varphi_x A} + \frac{\beta_{mx} M_x}{\gamma_x W_{1x}\left(1 - 0.8\dfrac{N}{N'_{Ex}}\right)} \leqslant f \tag{5.2.2-1}$$

where N——axial compression in the calculated portion of the member;

N'_{Ex}——parameter, $N'_{Ex} = \pi^2 EA/(1.1\lambda_x^2)$;

φ_x——stability factor of axial compression members buckling in the plane of bending;

M_x——maximum moment in the calculated portion of the member;

W_{1x}——gross section modulus referred to the more compressed fiber in the plane of bending;

β_{mx}——factor of equivalent moment, taken as follows:

1) For columns of frames and for members supported at the two ends:

①In the case of no transverse load: $\beta_{mx} = 0.65 + 0.35M_2/M_1$, where M_1 and M_2 are end moments taken as of same sign for members bent in single curvature (without inflexion point) and of different signs for members bent in reverse curvatures (with inflexion point), $|M_1| \geqslant |M_2|$;

② In the case of having end moments combined with transverse load: $\beta_{mx} = 1.0$ for members bent in single curvature and $\beta_{mx} = 0.85$ for members bent in reverse curvatures;

③In the case of having transverse loads and no end moments: $\beta_{mx} = 1.0$.

2) For cantilevers, columns of pure frame not taking account of 2nd order effect in stress analysis and columns of frame with weak bracings, $\beta_{mx} = 1.0$.

Beam-columns of singly symmetric sections shown in Item No 3 and 4 of Table 5.2.1 shall be checked by the following formula in addition to Formula (5.2.2-1) in case the bending moment in the plane of symmetric axis causes compression on the flange:

$$\left| \frac{N}{A} - \frac{\beta_{mx} M_x}{\gamma_x W_{2x}\left(1 - 1.25 \frac{N}{N'_{Ex}}\right)} \right| \leqslant f \qquad (5.2.2\text{-}2)$$

where W_{2x}——gross section modulus referred to the flangeless edge.

2 Out-of-plane stability:

$$\frac{N}{\varphi_y A} + \eta \frac{\beta_{tx} M_x}{\varphi_b W_{1x}} \leqslant f \qquad (5.2.2\text{-}3)$$

where φ_y——stability factor of axial compression members buckling out of the plane of M_x, determined in accordance with Clause 5.1.2;

φ_b—— overall stability factor of beams under uniform bending, determined in accordance with Appendix B, among which, Clause B.5 may be used For I-(H-) and T-section noncantilever members; for box section, use $\varphi_b = 1.0$;

M_x——maximum moment in the calculated member portion;

η——factor of section effect, taken as $\eta = 0.7$ for box section and $\eta = 1.0$ for others;

β_{tx}——factor of equivalent moment, taken as follows:

1) For members with lateral supports, β_{tx} shall be determined according to loading and internal force situation in the member portion between two adjacent supporting points as follows:

①In the case of no transverse load within the calculated portion: $\beta_{tx} = 0.65 + 0.35M_2/M_1$, where M_1 and M_2 are end moments in the plane of bending, taken as of same sign for member portions bent in a single curvature and of different signs for member portions bent in reverse curvatures; $|M_1| \geqslant |M_2|$;

②In the case of having end moments combined with transverse loads within the calculated portion: $\beta_{tx} = 1.0$ for member portions bent in single curvature, $\beta_{tx} =$

0.85 for those bent in reverse curvatures;

③In the case of having transverse loads and no end moment within the calculated portion: $\beta_{tx}=1.0$.

2) For members acting as cantilevers out of the plane of bending $\beta_{tx}=1.0$.

5.2.3 Laced or battened beam-columns bent about the open web axis (x-axis) shall be checked for in-plane stability by the following formula:

$$\frac{N}{\varphi_x A}+\frac{\beta_{mx}M_x}{W_{1x}\left(1-\varphi_x\frac{N}{N'_{Ex}}\right)}\leqslant f \qquad (5.2.3)$$

where $W_{1x}=\frac{I_x}{y_0}$, I_x being the moment of inertia of the gross area about the x-axis, y_0 being the distance from the x-axis to the axis of the more compressed component or to the outside face of web of this component, whichever is larger; φ_x and N'_{Ex} shall be determined using the equivalent slenderness ratio.

The overall out-of-plane stability of the member may not be checked in this case, but the stability of components shall be checked. The axial force of these components shall be determined as in the chords of trusses. For battened columns, bending of the components due to shear force shall be taken into account.

5.2.4 Laced or battened beam-columns bent about the solid web axis shall have their in-plane and out-of-plane stability checked in the same way as solid web members, but the equivalent slenderness ratios shall be used for out-of-plane overall stability calculation and φ_b taken as 1.0.

5.2.5 Doubly symmetrical I-(H-) and box (closed) section beam-columns bent in two principal planes, shall be checked for stability by the following formulae:

$$\frac{N}{\varphi_x A}+\frac{\beta_{mx}M_x}{\gamma_x W_x\left(1-0.8\frac{N}{N'_{Ex}}\right)}+\eta\frac{\beta_{ty}M_y}{\varphi_{by}W_y}\leqslant f \qquad (5.2.5\text{-}1)$$

$$\frac{N}{\varphi_y A}+\eta\frac{\beta_{tx}M_x}{\varphi_{bx}W_x}+\frac{\beta_{my}M_y}{\gamma_y W_y\left(1-0.8\frac{N}{N'_{Ey}}\right)}\leqslant f \qquad (5.2.5\text{-}2)$$

where φ_x, φ_y——stability factors of axial compression members about the strong axis x-x and the weak axis y-y;

φ_{bx}, φ_{by}——overall stability factors of beams under uniform bending: for I-(H-) section non-cantilever members, φ_{bx} may be determined in accordance with Section B.5 of Appendix B, φ_{by} may be taken as 1.0; for box section, $\varphi_{bx}=\varphi_{by}=1.0$;

M_x, M_y——maximum bending moment about the strong and the weak axes in the calculated member portion;

N'_{Ex}, N'_{Ey}——parameters, $N'_{Ex}=\pi^2EA/(1.1\lambda_x^2)$, $N'_{Ey}=\pi^2EA/(1.1\lambda_y^2)$;

W_x, W_y——gross section moduli about the strong and the weak axes;

β_{mx}, β_{my}——factors of equivalent moment used for calculation of in-plane stability in accordance with Clause 5.2.2;

β_{tx}, β_{ty}——factors of equivalent moment used for calculation of out-of-plane stability in accordance with Clause 5.2.2.

5.2.6 The stability of laced (or battened) beam-columns with two components bent in two principal planes shall be checked as follows:

1 Overall stability:

$$\frac{N}{\varphi_x A} + \frac{\beta_{mx} M_x}{W_{1x}\left(1 - \varphi_x \frac{N}{N'_{Ex}}\right)} + \frac{\beta_{ty} M_y}{W_{1y}} \leqslant f \tag{5.2.6-1}$$

where W_{1y}——gross section modulus referred to the more compressed fiber under the action of M_y.

2 Component stability:

The axial force of a component due to N and M_x shall be determined as in the chord member of a truss. M_y shall be distributed to two components according to Formulae(5.2.6-2) and (5.2.6-3) (see Fig. 5.2.6). The stability of the components shall then be checked according to Clause 5.2.2.

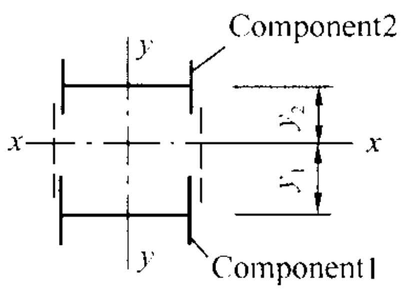


Fig. 5.2.6 Section of laced or battened member

Component 1:

$$M_{y1} = \frac{I_1/y_1}{I_1/y_1 + I_2/y_2} M_y \tag{5.2.6-2}$$

Component 2:

$$M_{y2} = \frac{I_2/y_2}{I_1/y_1 + I_2/y_2} M_y \tag{5.2.6-3}$$

where I_1, I_2——moment of inertia about the y-axis of components 1 and 2;

y_1, y_2——distances from x-x axis to the centroidal axis of components 1 and 2.

5.2.7 The lacing bars or battens of beam-columns shall be checked by using the actual shear force of the member or the shear force given by Formula(5.1.6), whichever is larger.

5.2.8 Bracing bars for reducing the lateral unsupported length of a beam-column out of the plane of bending shall have their axial force determined according to Clause 5.1.7 by considering the compression flange (for solid web members) or the compression component (for laced or battened members) as an axial compression member.

5.3 Effective length and allowable slenderness ratio

5.3.1 The effective length l_0 of chord members and single system web members (connected to chord members through gusset plate) of trusses shall be taken as those given in Table 5.3.1.

Table 5.3.1 Effective length l_0 of truss chords and single system web members

Item No.	Direction of bending	Chord members	Web members	
			End posts	Other web members
1	In truss plane	l	l	$0.8l$
2	Out of truss plane	l_1	l	l
3	In skew plane	—	l	$0.9l$

Note: 1 l—geometric length of the member (distance between panel points); l_1—distance between laterally supported points of chord members.

2 The skew plane refers to the plane oblique to the truss plane. It is applicable to web members with single angle and double angle cruciform section whose principal planes are non-coincident with truss plane.

3 For web members without gusset plate, the effective length in any plane shall be their geomeric length (excepting tubular structures).

If the distance between laterally supported points of compression chord of a truss is equal to twice the panel length (Fig. 5.3.1) and the two panels have unequal compressive forces, the effective length for out-of-plane buckling shall be taken as (but not less than $0.5l_1$):

$$l_0 = l_1\left(0.75 + 0.25\frac{N_2}{N_1}\right) \quad (5.3.1)$$

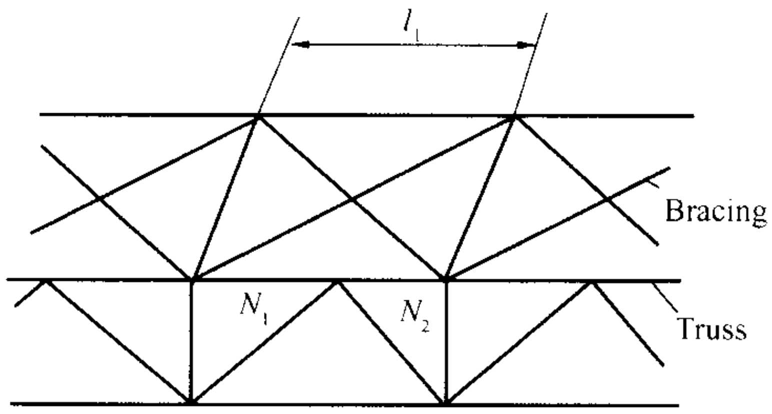


Fig. 5.3.1 Chord member of truss with unequal axial compression between laterally supported points

where N_1——the larger compressive force, taken as positive;

N_2—— the smaller compressive force or tensile force, taken as positive for compression and negative for tension.

The main compression diagonals in subdivided web member system and the verticals in K-web member system shall also have their out-of-plane effective length determined by Formula (5.3.1)(this length shall be l_1 for main tension diagonals), while their in-plane effective length shall be taken as distance between adjacent joints.

5.3.2 The effective length of cross diagonals interconnected at the intersection point shall be taken as the distance from a panel point to the crossing point for in-plane buckling, whereas for out-of-plane buckling, in the case of diagonals of equal length, it shall be taken as prescribed in the following:

1 Diagonal in compression.

1) In case the intersecting diagonal, is also in compression and that both diagonals, identical in section, are not interrupted at the crossing point:

$$l_0 = l\sqrt{\frac{1}{2}\left(1 + \frac{N_0}{N}\right)}$$

2) In case the intersecting diagonal is also in compression, but interrupted and spliced by a gusset plate at the crossing point:

$$l_0 = l\sqrt{1 + \frac{\pi^2}{12} \cdot \frac{N_0}{N}}$$

3) In case the intersecting diagonal is in tension and that both diagonals, identical in section, are not interrupted at the crossing point:

$$l_0 = l \cdot \sqrt{\frac{1}{2}\left(1 - \frac{3}{4} \cdot \frac{N_0}{N}\right)} \geqslant 0.5l$$

4) In case the intersecing diagonal is in tension, but interrupted and spliced by a gusset plate at the crossing point:

$$l_0 = l \cdot \sqrt{1 - \frac{3}{4} \cdot \frac{N_0}{N}} \geqslant 0.5l$$

In case this tension diagonal is continuous but the compression diagonal is interrupted and spliced by a gusset plate, and that $N_0 \geqslant N$ or the tension diagonal has its out-of-plane rigidity $EI_y \geqslant \frac{3N_0 l^2}{4\pi^2}\left(\frac{N}{N_0} - 1\right)$, take $l_0 = 0.5l$ for the compression diagonal.

In the above formulae, l denotes distance between panel points of the truss (the crossing point is not counted in); N and N_0 denote respectively the force in the diagonal under consideration, and that in the intersecting diagonal, both in absolute value. When both diagonals are in compression, and have identical section, take $N_0 \leqslant N$.

2 Diagonal in tension, l_0 shall be taken equal to l.

The effective length of single angle cross diagonals in skew plane shall be taken as the distance between a panel point and the crossing point.

5.3.3 The in-plane effective length of uniform section columns of single-or multiple-story frames shall be taken as the product of the column length (story height) by an effective length factor μ. Frames are classified as pure frame without bracing and braced frame. The latter is subdivided into strongly braced and weakly braced ones according to the stiffness against lateral translation.

1 Pure frames.

1) In case internal forces are computed by first-order elastic analysis, the column effective length factor μ shall be determined in accordance with Table D-2 in Appendix D as a frame with sidesway.

2) In case internal forces are computed by second-order elastic analysis, and that notional horizontal forces H_{ni} determined by Eq(3.2.8-1) are applied at the top of each story, take $\mu = 1.0$.

2 Braced frames.

1) In case the sway stiffness S_b (horizontal force that causes a leaning angle of unity) of the bracing system (bracing truss, shear wall, elevator shaft) satisfies the requirement of the Formula(5.3.3-1), the frame is recognized as strongly braced, and the effective length factor μ of its columns shall be detemined in accordance with Table D-1 in Appendix D as a frame without sidesway.

$$S_b \geqslant 3(1.2\sum N_{bi} - \sum N_{0i}) \qquad (5.3.3\text{-}1)$$

where $\sum N_{bi}$, $\sum N_{0i}$——sum of the load carrying capacity of all the columns in the i-th story calculated by making use of effective length factor of frames without sidesway and with sidesway respectively.

2) In case the sway stiffness S_b of the bracing system does not satisfy the requirement of the Formula (5. 3. 3-1), the frame is recognized as weakly braced and the relevant stability factor φ of axial compression member shall be determined according to Eq (5.3.3-2).

$$\varphi = \varphi_0 + (\varphi_1 - \varphi_0)\frac{S_b}{3(1.2\sum N_{bi} - \sum N_{0i})} \qquad (5.3.3\text{-}2)$$

where φ_1, φ_0——the stability factor of axially loaded columns corresponding to column effective length factor of frames without and with sidesway respectively from Tables in Appendix D.

5.3.4 The in-plane effective length of a stepped column rigidly restrained at the base of a single-story mill building shall be determined as follows:

1 Single-stepped column:

1) Effective length factor μ_2 for the lower portion of the column:

For column pin-connected to the girder, μ_2 shall be the figures taken from Table D-3 (single stepped column with free upper end) in Appendix D, multiplied by an appropriate reduction factor given in Table 5.3.4. For columns rigidly connected to the girder, μ_2 shall be the figures taken from Table D-4 (single-stepped column with translation-free and rotation-fixed upper end) in Appendix D, multiplied by an appropriate reduction factor given in Table 5.3.4.

Table 5.3.4 Reduction factor for the effective length of stepped columns in single story mill building

<table>
<tr><th colspan="4">Type of the mill building</th><th rowspan="2">Reduction factor</th></tr>
<tr><th>Number of bays</th><th>Number of columns in a longitudinal row within a section between expansion joints of the building</th><th>Roofing</th><th>Longitudinal horizontal bracing on the two sides of the roof system</th></tr>
<tr><td rowspan="4">Single</td><td>≤6</td><td>—</td><td>—</td><td rowspan="2">0.9</td></tr>
<tr><td rowspan="3">>6</td><td rowspan="2">Other than large-size precast R. C. roofing slab</td><td>not provided</td></tr>
<tr><td>provided</td><td rowspan="3">0.8</td></tr>
<tr><td>Large-size precast R.C. roofing slab</td><td>—</td></tr>
<tr><td rowspan="3">Multiple</td><td rowspan="3">—</td><td rowspan="2">Other than large-size precast R. C. roofing slab</td><td>not provided</td></tr>
<tr><td>provided</td><td rowspan="2">0.7</td></tr>
<tr><td>Large-size precast R.C. roofing slab</td><td>—</td></tr>
</table>

Note: The reduction factor 0.9 may be used for outdoor structures with girder (e.g. drop ball mill, etc.).

2) Effective length factor μ_1 for the upper portion of the column shall be calculated by the following formula:

$$\mu_1 = \frac{\mu_2}{\eta_1} \tag{5.3.4-1}$$

where η_1 is a parameter calculated by the formula shown in Table D-3 or D-4 in Appendix D.

2 Double-stepped column:

1) Effective length factor μ_3 of the lower portion of the column:

For columns pin-connected to the girder, μ_3 shall be the figures taken from Table D-5 (double-stepped column with free upper end), in Appendix D, multiplied by an appropriate reduction factor given in Table 5.3.4.

For columns rigidly connected to the girder, μ_3 shall be the figures taken from Table D-6 (double-stepped column with translation-free and rotation-fixed upper end), in Appendix D, multiplied by an appropriate reduction factor given in Table 5.3.4.

2) Effective length factors μ_1 and μ_2 for the upper and middle portions shall be calculated by the following formulae respectively:

$$\mu_1 = \frac{\mu_3}{\eta_1} \tag{5.3.4-2}$$

$$\mu_2 = \frac{\mu_3}{\eta_2} \tag{5.3.4-3}$$

where η_1 and η_2——parameters calculated by formulae shown in Tables D-5 and D-6 in Appendix D.

Note: For the effective length of tapered columns, refer to the current national standard "Technical code of cold-formed thin-wall steel structures" GB 50018.

5.3.5 In calculating the momemt of inertia of laced or battened columns of a frame and that of lattice girders(trusses), the influence of the variation of section depth of columns or girders and that of the deformation of lacing bars or batten plates(or web members) shall be taken into account.

5.3.6 Special consideration to be taken in determining the effective length factor of frame columns under the following situations:

1 The column slenderness ratio factor of pure frames and weakly braced frames adjoined with leaning columns (columns hinged at both ends) shall be multiplied by an amplification factor η:

$$\eta = \sqrt{1 + \frac{\sum(N_l/H_l)}{\sum(N_f/H_f)}} \tag{5.3.6}$$

where $\sum(N_f/H_f)$——summation of the ratio of the factored axial compression to column height of all the frame columns;

$\sum(N_l/H_l)$——summation of the ratio of factored axial compression to column height of all the leaning columns.

The effective length of leaning columns is equal to their geometric length.

2 When other columns in the same story of the calculated column, or columns continuous with it in the upper and lower story, have potential in load-carrying capacity, the effective length factor of the column may be reduced by taking into account the restraining effect of these adjacent columns; whereas the effective length factor of the latter should be enlarged.

3 For columns joining beams by semi-rigid connections, the effective length shall be determined by taking the behavior of the connections into consideration.

5.3.7 The effective length of frame columns in the longitudinal direction of the building (out of the frame plane) shall be taken as the distance between supporting points restraining the out-of-plane displacement.

5.3.8 The slenderness ratio of compression members should not exceed the allowable values given in Table 5.3.8.

Table 5.3.8 Allowable values of slenderness ratio for compression members

Item No.	Nomenclature of members	Allowable values
1	Columns, members of trusses and monitors. Lacing of columns, column bracings beneath crane girders or crane trusses	150
2	Bracings(except column bracings beneath crane girders or crane trusses). Members used to reduce the slenderness ratio of compression members	200

Note: 1 A slenderness ratio of 200 may be allowed for compression web members in trusses(including space trusses) when they are stressed to or under 50% of their capacities.

2 For single angle compression members, the least radius of gyration shall be used for calculation of slenderness ratio, but the radius of gyration about the axis parallel to the leg may be used for calculation of the out-of-plane slendemess ratio in the case of cross diagonals interconnected at the crossing point.

3 For trusses with span length equal to or larger than 60m, the allowable slenderness ratio should be taken as 100 for compression chords and end posts, and 150(when subjected to static or indirect dynamic load), or 120 (when subjected to direct dynamic load) for other web members in compression.

4 In case member section is governed by allowable slenderness ratio, the effect of twisting may be neglected in calculating the slendeness ratio.

5.3.9 The slenderness ratio of tension members should not exceed the allowable values given in Table 5.3.9.

Table 5.3.9 Allowable values of slenderness ratio for tension members

Item No.	Nomenclature of members	Structures subject to static or indirect dynamic loading		Structures subject to direct dynamic loading
		Common buildings	Mill buildings with heavy duty crane	
1	Members of Trusses	350	250	250

Table 5.3.9(Continued)

Item No.	Nomenclature of members	Structures subject to static or indirect dynamic loading		Structures subject to direct dynamic loading
		Common buildings	Mill buildings with heavy duty crane	
2	Column bracings beneath crane girders or crane trusses	300	200	—
3	Other tension members, bracings and ties (except pretensioned round bars)	400	350	—

Note: 1 For structures subject to static loading, slenderness ratio of tension members may be checked only in vertical planes.

2 For structures subject to direct or indirect dynamic loading, the slenderness ratio of a single angle tension member is calculated similarly to Note 2 of Table 5.3.8.

3 The slenderness ratio of bottom chord of crane trusses for medium and heavy duty cranes should not exceed 200.

4 In mill buildings equipped with soaking pit cranes and stripper cranes or rigid claw cranes, the slenderness ratio of the bracings(except Item No. 2 in the Table)should not exceed 300.

5 When tension members change into compression ones under the combined action of dead and wind loads, their slenderness ratio should not exceed 250.

6 For trusses with span length equal to or larger than 60m, the slenderness ratio of tension chords and tension web members should not exceed 300 (when subjected to static or indirect dynamic loading) or 250 (when subjected to direct dynamic loading).

5.4 Local stability of compression members

5.4.1 The ratio of free outstand, b, of a flange to its thickness, t, in compression members shall conform to the following requirements:

1 Axial compression members:

$$\frac{b}{t} \leqslant (10 + 0.1\lambda)\sqrt{\frac{235}{f_y}} \tag{5.4.1-1}$$

where λ——the larger of the slenderness ratios of the member in two directions, taken as 30 when $\lambda < 30$, and as 100 when $\lambda > 100$.

2 Beam-columns:

$$\frac{b}{t} \leqslant 13\sqrt{\frac{235}{f_y}} \tag{5.4.1-2}$$

b/t may be enlarged to $15\sqrt{235/f_y}$ in case $\gamma_x = 1.0$ is used for strength and stability checking.

Note: The free outstand b of the flange shall be taken as follows: the distance from the face of the web to the flange tip for welded members; the distance from the toe of the fillet to the flange tip for rolled members.

5.4.2 The ratio of effective web depth, h_0, to thickness, t_w, in I-section compression members shall conform to the following requirements:

1 Axial compression members:

$$\frac{h_0}{t_w} \leqslant (25 + 0.5\lambda)\sqrt{\frac{235}{f_y}} \tag{5.4.2-1}$$

where λ——the larger of the slenderness ratios of the member in two directions, taken as 30 when $\lambda < 30$, and as 100 when $\lambda > 100$.

2 Beam-columns:

$$\frac{h_0}{t_w} \leqslant (16\alpha_0 + 0.5\lambda + 25)\sqrt{\frac{235}{f_y}}, \text{when} \quad 0 \leqslant \alpha_0 \leqslant 1.6 \tag{5.4.2-2}$$

$$\frac{h_0}{t_w} \leqslant (48\alpha_0 + 0.5\lambda - 26.2)\sqrt{\frac{235}{f_y}}, \text{when} \quad 1.6 < \alpha_0 \leqslant 2 \tag{5.4.2-3}$$

$$\alpha_0 = \frac{\sigma_{max} - \sigma_{min}}{\sigma_{max}}$$

where σ_{max}——maximum compressive stress on the edge of effective web depth, not taking account of the stability factor of the member, nor the plasticity adaptation factor of the section;

σ_{min}——corresponding stress on the other edge of effective web depth, taken as positive for compression and negative for tension;

λ——slenderness ratio in the plane of bending, taken as 30 when $\lambda < 30$ and as 100 when $\lambda > 100$.

5.4.3 The width-to-thickness ratio of the compression flanges in box section compression members shall conform to the requirement of Clause 4.3.8, whereas the ratio of the effective web depth, h_0, to thickness, t_w, shall conform to the following requirements:

1 For axial compression members:

$$\frac{h_0}{t_w} \leqslant 40\sqrt{\frac{235}{f_y}} \tag{5.4.3}$$

2 For beam-columns, the ratio h_0/t_w shall not exceed 0.8 times the right-hand side of Formula(5.4.2-2)or(5.4.2-3), but not less than 40 $\sqrt{235/f_y}$.

5.4.4 The depth-to-thickness ratio of the web in T-section compression members shall not exceed the following values:

1 Axial compression members and beam-columns in which bending moment causes tension on the free edge of the web:

For hot-rolled cut-T section: $(15 + 0.2\lambda)\sqrt{235/f_y}$

For welded T-section: $(13 + 0.17\lambda)\sqrt{235/f_y}$

2 Beam-columns, in which bending moment causes compression on the free edge of the web:

$$15\sqrt{235/f_y}, \text{when } \alpha_0 \leqslant 1.0$$

$$18\sqrt{235/f_y}, \text{when } \alpha_0 > 1.0$$

λ and α_0 are taken according to Clauses 5.4.1 and 5.4.2 respectively.

5.4.5 The ratio of outside diameter to wall thickness of circular tubes subject to compression shall not exceed $100(235/f_y)$.

5.4.6 When the web depth-to-thickness ratio of H-, I- and box-section compression members does not conform to the requirements of Clause 5.4.2 or 5.4.3, the web may be strengthened with longitudinal stiffeners, or otherwise only a width of $20t_w \sqrt{235/f_y}$ on each side of the web should be considered effective in calculation of the strength and stability of members (but the whole section is effective for determining the stability factor φ).

When longitudlinal stiffeners are provided, the portion of the web between the more compressed flange and the longitudinal stiffener shall have a depth-to-thickness ratio conforming to the requirement of Clause 5.4.2 or 5.4.3.

Longitudinal stiffeners should be placed in pair on the two sides of the web, with an outstanding width not less than $10t_w$ on each side, and thickness not less than $0.75t_w$.

6 Fatigue calculation

6.1 General stipulations

6.1.1 For steel structural members and their connections subject directly to repeated fluctuations of dynamic loading, fatigue calculation shall be carried out when the number of stress cycles, n, is equal to or greater than 5×10^4.

6.1.2 The provisions in this Chapter are not applicable to the fatigue calculation of structural members and their connections under special conditions (e.g. members subject to surface temperatures above 150℃, members in corrosive environments of sea water, residual stress relieved through post-weld heat treatment, low cycle-high strain fatigue, etc.).

6.1.3 The allowable stress range method is used in fatigue calculation, with stresses calculated in accordance with elastic state. The allowable stress ranges are determined in accordance with the detail categories of members and connections as well as the number of stress cycles. For the region of a member, where no tensile stress occurs in the stress cycles, fatigue calculation may not be needed.

6.2 Fatigue calculation

6.2.1 For constant amplitude fatigue (the stress range in all stress cycles remains constant) calculation shall be carried out by the following formula:

$$\Delta\sigma \leqslant [\Delta\sigma] \tag{6.2.1-1}$$

where $\Delta\sigma$——stress range for welded region, $\Delta\sigma = \sigma_{max} - \sigma_{min}$; reduced stress range for non-weld regions, $\Delta\sigma = \sigma_{max} - 0.7\sigma_{min}$;

σ_{max}——maximun tensile stress (taken as positive) of each stress cycle in the region considered;

σ_{min}——minimum tensile stress (taken as positive) of compressive stress (taken as negative) of each stress cycle in the region considered;

$[\Delta\sigma]$——allowable stress range (N/mm^2) of constant amplitude fatigue, and shall be calculated by the following formula:

$$[\Delta\sigma] = \left(\frac{C}{n}\right)^{\frac{1}{\beta}} \tag{6.2.1-2}$$

n——number of stress cycles;

C, β——parameters taken from Table 6.2.1 according to the detail category of members and connections in Appendix E.

Table 6.2.1 Parameters C, β

Detail category of members and connections	1	2	3	4	5	6	7	8
C	1940×10^{12}	861×10^{12}	3.26×10^{12}	2.18×10^{12}	1.47×10^{12}	0.96×10^{12}	0.65×10^{12}	0.41×10^{12}
β	4	4	3	3	3	3	3	3

Note: Formula(6.2.1-1) is also valid for shear stress.

6.2.2 For variable amplitude fatigue(the stress range in stress cycles varies at random), if the design stress spectrum composed of the frequency distribution of various loadings, the level of stress ranges and the sum of the frequency distributions during the service life of the structure could be predicted, then it may be reduced to equivalent constant amplitude fatigue, and calculated by the following formula:

$$\Delta\sigma_e \leqslant [\Delta\sigma] \tag{6.2.2-1}$$

where $\Delta\sigma_e$——equivalent stress range of variable amplitude fatigue, determined by the following formula:

$$\Delta\sigma_e = \left[\frac{\sum n_i(\Delta\sigma_i)^\beta}{\sum n_i}\right]^{1/\beta} \tag{6.2.2-2}$$

$\sum n_i$——expected service life of the structure expressed in number of stress cycles;

n_i——number of stress cycles with stress range level reaching $\Delta\sigma_i$ within the expected life.

6.2.3 The fatigue of crane girders for heavy duty cranes and crane trusses for both heavy and medium duty cranes may be regarded as constant amplitude fatigue and calculated by the following formula:

$$\alpha_f\Delta\sigma \leqslant [\Delta\sigma]_{2\times10^6} \tag{6.2.3}$$

where α_f——equivalent factor of underloading effect, adopted in accordance with Table 6.2.3-1;

$[\Delta\sigma]_{2\times10^6}$——allowable stress range corresponding to the number of cycles $n = 2\times10^6$, adopted in accordance with Table 6.2.3-2.

Table 6.2.3-1 Equivalent factor of underloading effect for crane girder and crane truss, α_f

Crane category	α_f
Heavy duty crane with rigid hook(e.g., soaking pit crane)	1.0
Heavy duty crane with flexible hook	0.8
Medium duty crane	0.5

Table 6.2.3-2 Allowable stress range(N/mm²) corresponding to number of cycles $n = 2\times10^6$

Detail category of members and connections	1	2	3	4	5	6	7	8
$[\Delta\sigma]_{2\times10^6}$	176	144	118	103	90	78	69	59

Note: The allowable stress range given in the Table is calculated by Formula(6.2.1-2).

7 Calculation of connections

7.1 Welded connection

7.1.1 The quality level of welds shall be specified according to the structural importance, loading character, weld types, service conditions and stress states, and shall be selected by the following principles:

1 For butt welds subject to fatigue loading, full penetration shall be specified, and the weld quality level shall be:

1) For butt weld, or combined butt and fillet weld in T joint, with loading perpendicular to the weld length, class 1 weld shall be specified when loaded in tension, and class 2 weld shall be specified when loaded in compression;

2) For butt weld with loading parallel to the weld length, class 2 weld shall be specified.

2 For butt welds without fatigue loading, if the weld resistance is required to be equal to that of the parent metal, full penetration is required and the quality level shall not be lower than class 2 for welds in tension, and class 2 should be specified for welds in compression.

3 Full penetration shall be specified for the T joint between top flange and web of a crane girder, and for welds between top chord and gusset plate of a crane truss to carry heavy-duty crane or medium duty crane with lifting capacity $Q \geqslant 50$t. Normally combined butt and fillet welds in T joint shall be used and quality level shall not be lower than class 2.

4 For T joint with fillet weld or combined partial penetration butt and fillet weld, and for lap joint with fillet weld, the weld quality level shall be specified as follows:

1) For structures subject to dynamic and fatigue loading, and for crane girder to carry medium duty crane with lifting capacity $Q \geqslant 50$t, weld quality by visual inspection shall be class 2;

2) For other structures, weld quality by visual inspection may be class 3.

7.1.2 Capacity of butt weld, or combined butt and fillet weld in T joint shall be calculated as follows:

1 In butt joints and T joints, the design capacity of butt weld or combined butt and fillet weld when subject to axial tension or compression perpendicular to the weld, the following formula shall be satisfied:

$$\sigma = \frac{N}{l_w t} \leqslant f_t^w \quad \text{or} \quad f_c^w \tag{7.1.2-1}$$

where N——design value of axial tensile or compressive force;

l_w——effective weld length;

t——thickness of thinner plate in butt joint, or the web thickness in T joints;

f_t^w, f_c^w——design values of tensile and compressive strength of butt welds.

2 For the butt weld or combined butt and fillet weld in T joint subject to both bending moment and shear force, the normal stress and shear stress shall be calculated individually in butt joints and T joints. Where both the comparatively large normal stress and shear stress coexist(e.g. at the end of the transverse butt weld of a beam web splice), the reduced stress shall satisfy the following formula:

$$\sqrt{\sigma^2 + 3\tau^2} \leqslant 1.1 f_t^w \qquad (7.1.2\text{-}2)$$

Notes: 1 When plates spliced with oblique butt weld, the weld capacity is adequate if θ, the angle between the applied force and weld, satisfies $\tan\theta \leqslant 1.5$;

2 When run-on and run-off tabs are not applied, $2t$ reduction shall be considered when calculating the effective weld length of a butt weld or combined butt and fillet weld in T joint.

7.1.3 The strength of right-angle fillet welds shall be calculated as follows:

1 Under the action of tensile, compressive or shear force passing through the centroid of the welds:

For the end-fillet weld(the force perpendicular to the weld length):

$$\sigma_f = \frac{N}{h_e l_w} \leqslant \beta_f f_f^w \qquad (7.1.3\text{-}1)$$

For the side-fillet weld(the force parallel to the weld length):

$$\tau_f = \frac{N}{h_e l_w} \leqslant f_f^w \qquad (7.1.3\text{-}2)$$

2 Under the combined actions of various forces, at the location of combined σ_f and τ_f:

$$\sqrt{\left(\frac{\sigma_f}{\beta_f}\right)^2 + \tau_f^2} \leqslant f_f^w \qquad (7.1.3\text{-}3)$$

where σ_f——stress on the effective area ($h_e l_w$) of the weld, perpendicular to the weld length;

τ_f——shear stress on the effective area of the weld, parallel to the weld length;

h_e——throat thickness and taken as $0.7h_f$ for right-angle fillet weld, where h_f is the weld leg size(Fig. 7.1.3);

l_w——effective weld length, taken as actual weld length minus $2h_f$ for each weld;

β_f——enlargement coefficient for design value of end fillet strength, $\beta_f = 1.22$ for welds subject to static loading or dynamic loading indirectly, or $\beta_f = 1.0$ for welds subject to dynamic loading directly.

7.1.4 In T joints, when the angle α between the fusion faces of a oblique-angle fillet weld is more than 60° and less than 135°(Fig. 7.1.4), the capacity of the fillet weld shall satisfy Eq. (7.1.3-1) to (7.1.3-3), but using $\beta_f = 1.0$, and the throat thickness is taken as $h_e = h_f \cos\frac{\alpha}{2}\Big/2$ (when root gap b, b_1 or $b_2 \leqslant 1.5$mm), or $h_e = \left[h_f - \frac{b(\text{or } b_1, b_2)}{\sin\alpha}\right]\cos\frac{\alpha}{2}$ (when

b, b_1 or b_2 > 1.5mm but ≤ 5mm).

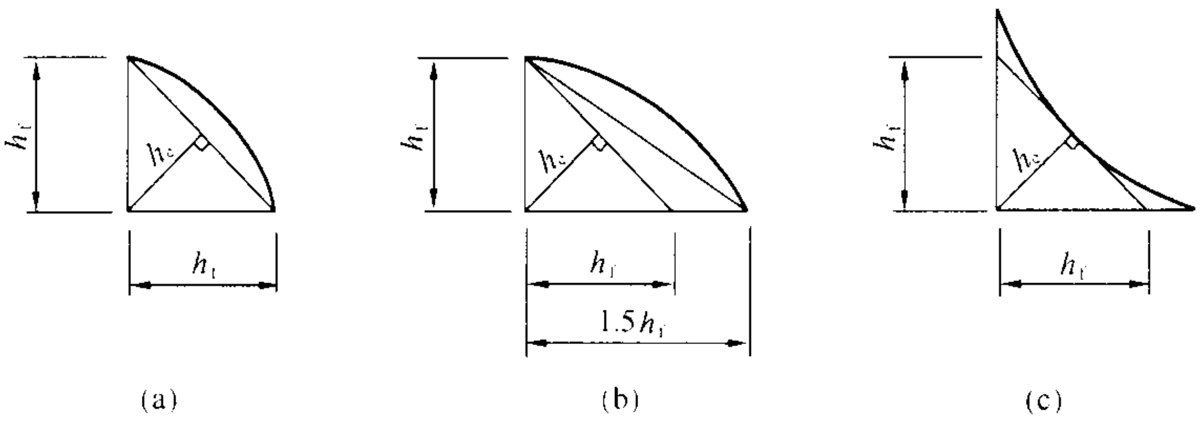


Fig. 7.1.3 Profile of right-angle fillet welds

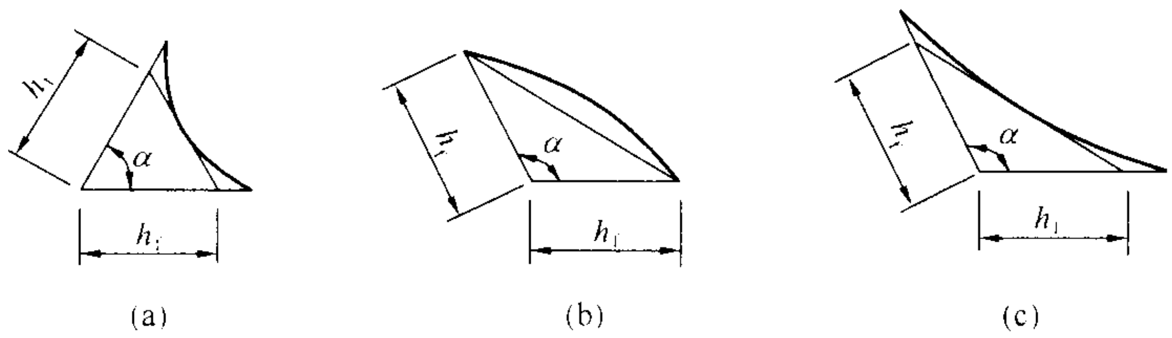


Fig. 7.1.4-1 Profile of oblique-angle fillet welds in T joint

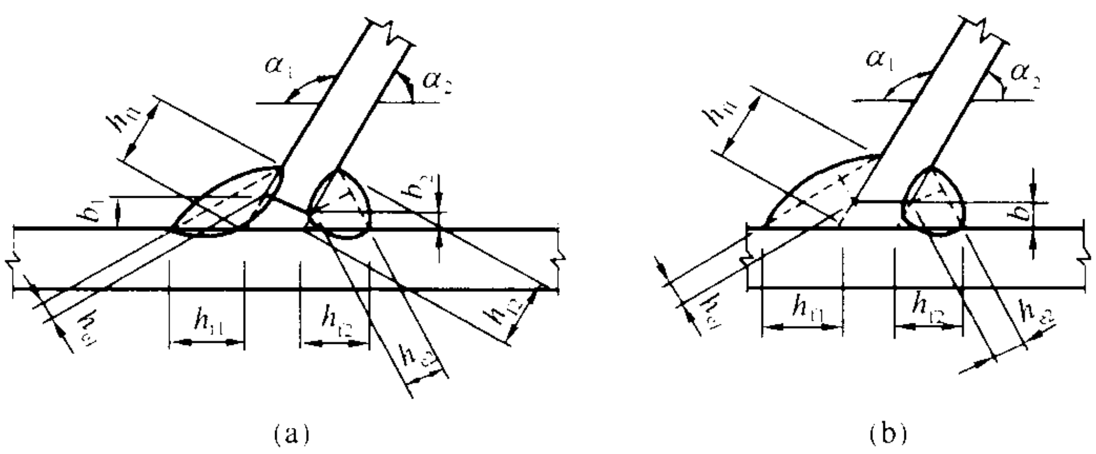


Fig. 7.1.4-2 Details of root gap and weld section in T joint

7.1.5 The strength of partially penetrated butt weld (Fig. 7.1.5a, b, d, e) and combined butt and fillet weld in T joints (Fig. 7.1.5c) shall satisfy Eq. (7.1.3-1) to (7.1.3-3). $\beta_f = 1.22$ when the applied compressive force is perpendicular to the weld length, otherwise $\beta_f = 1.0$. The throat thickness shall be determined as:

For V groove (Fig. 7.1.5a):

when $\alpha \geqslant 60°$, $h_e = s$;

when $\alpha < 60°$, $h_e = 0.75s$.

For single bevel or K groove (Fig. 7.1.5b, c), $h_e = s - 3$ when $\alpha = 45° \pm 5°$.

For U or J groove (Fig. 7.1.5d, e), $h_e = s$.

where s —— groove depth, and is the shortest distance from the groove root to the weld surface (neglecting the weld reinforcement);

α —— angle between the fusion faces of V, single bevel or K groove.

When the weld leg size on the fusion line is equal or approximate to the shortest distance s(Fig. 7. 1. 5b, c, e), the design value of shear strength shall be taken as 0.9 times the design value of strength for fillet weld.

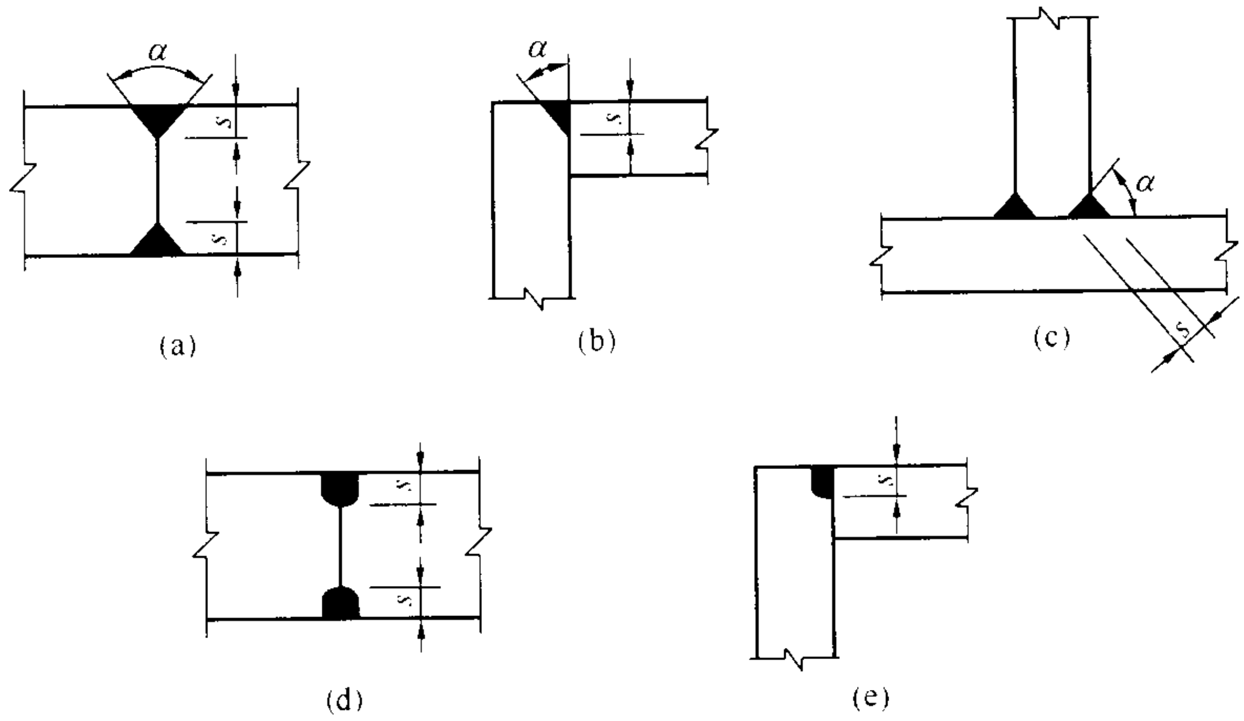


Fig. 7. 1. 5 Profile of partially penetrated butt welds, combined butt and fillet welds

7.2 Fastener(bolted and riveted) connection

7.2.1 The capacity of ordinary bolts, anchor bolts and rivets should be determined as follows:

1 In joints with ordinary bolts or rivets subject to shear force, the design capacity per ordinary bolt or rivet shall be taken as the lesser design value of shear capacity and bearing capacity.

The desing value of shear capacity:

For ordinary bolts:

$$N_v^b = n_v \frac{\pi d^2}{4} f_v^b \tag{7.2.1-1}$$

For rivets:

$$N_v^r = n_v \frac{\pi d_0^2}{4} f_v^r \tag{7.2.1-2}$$

The design value of bearing capacity:

For ordinary bolts:

$$N_c^b = d \sum t \cdot f_c^b \tag{7.2.1-3}$$

For rivets:

$$N_c^r = d_0 \sum t \cdot f_c^r \tag{7.2.1-4}$$

where n_v——number of shear plane;

d——bolt shank diameter;

d_0——rivet hole diameter;

$\sum t$——lesser sum of thickness of the connected plies bearing in the same direction;

f_v^b, f_c^b——design values of shear and bearing strength of the ordinary bolt respectively;

f^r_v, f^r_c——design values of shear and bearing strength of the rivet respectively.

2 In joints with ordinary bolts, anchor bolts or rivets subject to tension in shank axis direction, the design value of tension capacity per ordinary bolt, anchor bolt or rivet shall be determined as:

For ordinary bolts:

$$N^b_t = \frac{\pi d_e^2}{4} f^b_t \tag{7.2.1-5}$$

For anchor bolts:

$$N^a_t = \frac{\pi d_e^2}{4} f^a_t \tag{7.2.1-6}$$

For rivets:

$$N^r_t = \frac{\pi d_0^2}{4} f^r_t \tag{7.2.1-7}$$

where d_e——effective diameter of ordinary bolt or anchor bolt at threaded section;

f^b_t, f^a_t, f^r_t——design values of tensile strength of the ordinary bolt, anchor bolt or rivet respectively.

3 The ordinary bolts or rivets subject to combined action of shear and tension in the shank axis direction shall satisfy the following formulae:

For ordinary bolts:

$$\sqrt{\left(\frac{N_v}{N^b_v}\right)^2 + \left(\frac{N_t}{N^b_t}\right)^2} \leqslant 1 \tag{7.2.1-8}$$

$$N_v \leqslant N^b_c \tag{7.2.1-9}$$

For rivets:

$$\sqrt{\left(\frac{N_v}{N^r_v}\right)^2 + \left(\frac{N_t}{N^r_t}\right)^2} \leqslant 1 \tag{7.2.1-10}$$

$$N_v \leqslant N^r_c \tag{7.2.1-11}$$

where N_v, N_t——design values of shear and tensile force carried by an ordinary bolt or rivet respectively;

N^b_v, N^b_t, N^b_c——design values of shear, tensile and bearing capacity of an ordinary bolt respectively;

N^r_v, N^r_t, N^r_c——design values of shear, tensile and bearing capacity of a rivet respectively.

7.2.2 High strength bolt in friction type joint shall be calculated as follows:

1 The design capacity of a high strength bolt in shear connection shall be determined by:

$$N^b_v = 0.9 n_f \mu P \tag{7.2.2-1}$$

where n_f——number of load transmitting friction interface;

μ——slip coefficient at friction interface taken from Table 7.2.2-1;

P——pretension of a high strength bolt taken from Table 7.2.2-2.

Table 7.2.2-1 Slip coefficient μ at friction interface

Faying surface treatment	Steel grade of connected parts		
	Q235	Q345 and Q390	Q420
Blast-cleaning	0.45	0.50	0.50
Inorganic zinc paint coated after blast-cleaning	0.35	0.40	0.40
Rusted after blast-cleaning	0.45	0.50	0.50
Hand-cleaned with wire brush or untreated as rolled clean surfaces	0.30	0.35	0.40

Table 7.2.2-2 High strength bolt pretension P(kN)

Bolt property grade	Nominal bolt diameter(mm)					
	M16	M20	M22	M24	M27	M30
8.8	80	125	150	175	230	280
10.9	100	155	190	225	290	355

2 The design capacity of a high strength bolt subject to tension in bolt shank direction shall be taken as $N_t^b = 0.8P$.

3 The high strength bolt in friction type joint subject to combined shear in friction interface and external tension action (in the direction of bolt shank axis)shall satisfy:

$$\frac{N_v}{N_v^b} + \frac{N_t}{N_t^b} \leqslant 1 \tag{7.2.2-2}$$

where N_v, N_t——design values of shear and tensile force carried by a high strength bolt;

N_v^b, N_t^b——design values of shear and tension capacity of a high strength bolt respectively.

7.2.3 High strength bolt in bearing type joint shall be calculated as follows:

1 The pretension P applied to a high strength bolt in bearing-type joint shall be the same as that in friction-type joint. The faying surface shall be free of grease or rust.

High strength bolt in bearing type joint shall not be applied to the structures directly subject to dynamic loading.

2 The design value of shear capacity of a high strength bolt in bearing-type joint is calculated as that of an ordinary bolt. When the threaded section is in shearing, the design value of shear capacity shall be calculated based on the effective area of the threaded section.

3 When a high strength bolt in bearing-type joint is subjected to tension in the direction of shank axis, the design value of tension capacity is determined as that of an ordinary bolt.

4 The high strength bolt in bearing type joint subject to combined shear and tension in the direction of shank axis shall satisfy the following formulae:

$$\sqrt{\left(\frac{N_v}{N_v^b}\right)^2 + \left(\frac{N_t}{N_t^b}\right)^2} \leqslant 1 \tag{7.2.3-1}$$

$$N_v \leqslant N_c^b / 1.2 \tag{7.2.3-2}$$

where N_v, N_t——design values of shear and tensile force carried by a high strength bolt;

N_v^b, N_t^b, N_t^c——design values of shear, tension and bearing capacity of a high strength bolt.

7.2.4 At the end connection of a member or at one side of member splice, when the connecting length l_1 in the direction of the force transfer is longer than $15d_0$, the design capacity of a bolt or a rivet shall be reduced through multiplying a reduction factor $\left(1.1 - \frac{l_1}{150d_0}\right)$. In case of l_1 longer than $60d_0$, the reduction factor should be taken as 0.7, where d_0 is the bolt hole diameter.

7.2.5 Under the following circumstances, the number of bolts and rivets shall be increased:

1 When the members are connected through filler or packing plates, the number of bolts (except high strength bolts in friction-type joint) or rivets shall be 10% more than the calculated values.

2 For a lap joint or a joint with single splice plate transferring axial force, the number of bolts (except high strength bolts in friction-type joint) or rivets shall be 10% more than the calculated values to allow for the effects of bending caused by eccentricity.

3 At the end joint of a member where a lug angle is used to connect the outstands of angle or channel sections in order to reduce the joint length, the number of bolts or rivets allocated on one of the two legs of the lug angle shall be 50% more than the calculated values.

4 When the total grip thickness of riveted plates exceeds 5 times the rivet hole diameter, the number of rivets shall be increased by 1% for each additional 2mm grip (minimum 1 rivet), but the total grip should not exceed 7 times the rivet hole diameter.

7.2.6 The self-tapping screws, blind rivets and cartridge-fired pins for connecting thin-walled plates shall satisfy the stipulation specified in relevant standards.

7.3 Flange connection of built-up I-section (girder)

7.3.1 For fillet welds connecting the flange of a built-up I section to its web on both sides, the following formula shall be satisfied:

$$\frac{1}{2h_e}\sqrt{\left(\frac{VS_f}{I}\right)^2 + \left(\frac{\psi F}{\beta_f l_z}\right)^2} \leqslant f_f^w \tag{7.3.1}$$

where S_f——first moment of gross section of the flange about the neutral axis of the girder;

I——moment of inertia of the gross cross section of the girder.

F, ψ and l_z in Eq. (7.3.1) shall be determined in accordance with Clause 4.1.3 and β_f is defined in Clause 7.1.3.

Note: 1 When a fixed concentrated load is applied on the top flange of the girder, a bearing stiffener should be provided with its end tightly fitted to the top flange, so as to take $F = 0$;

2 When the flange and web are connected by combined butt and fillet weld with full penetration, the resistance may not be calculated.

7.3.2 For rivets (or high strength bolts in friction type joints) connecting the flange of a built-up I section to its web, the following formula shall be satisfied:

$$a\sqrt{\left(\frac{VS_f}{I}\right)^2+\left(\frac{\alpha_1\psi F}{l_z}\right)^2}\leqslant n_1 N_{min}^r \text{ or } n_1 N_v^b \tag{7.3.2}$$

where a——spacing of rivets(or bolts)on flange;

α_1——a parameter, $\alpha_1=0.4$ when load F is acted on the top flange and the web edge tightly fitted to the top flange, otherwise $\alpha_1=1$;

n_1——number of rivets(or bolts)on the cross section;

N_{min}^r——lesser design value of shear capacity and bearing capacity of a rivet;

N_v^b——design value of shear capacity of a high strength bolt in friction-type joint.

Notes: when a fixed concentrated load is applied on the top fiange of the girder, a bearing stiffener should be provided with its end tightiy fitted to the top flange, so as to take $F=0$.

7.4 Beam-to-column rigid connection

7.4.1 In case of an I-section beam connected to a H-section column flange, if the beam flange is connected with full penetration T-butt weld and beam web is connected with weld or high strength bolt in friction-type joint, column web transverse stiffeners may not be provided if the following requirements are satisfied:

1 Where the beam flange is in compression, the column web thickness t_w shall simultaneously satisfy:

$$t_w\geqslant\frac{A_{fc}f_b}{b_e f_c} \tag{7.4.1-1}$$

$$t_w\geqslant\frac{h_c}{30}\sqrt{\frac{f_{yc}}{235}} \tag{7.4.1-2}$$

where A_{fc}——sectional area of beam flange in compression;

f_c——design value of tensile or compressive strength of column steel;

f_b——design value of tensile or compressive strength of beam steel;

b_e——assumed stress distribution length at the edge of the column effective web depth where a concentrated compressive force is applied perpendicularly to the column flange, referring to Eq. (4.1.3-2);

h_c——depth of column web;

f_{yc}——yielding strength of column steel.

2 Where the beam flange is in tension, the column flange thickness t_c shall satisfy:

$$t_c\geqslant 0.4\sqrt{A_{ft}f_b/f_c} \tag{7.4.1-3}$$

where A_{ft}——sectional area of beam flange in tension.

7.4.2 The column joint panel enclosed by the column flanges and web transverse stiffeners shall be calculated as follows:

1 The shear capacity shall satisfy:

$$\frac{M_{b1}+M_{b2}}{V_p}\leqslant\frac{4}{3}f_v \tag{7.4.2-1}$$

where M_{b1}, M_{b2}——design values of beam bending moment at the two sides of the joint;

V_p—— volume of joint panel, $V_p = h_b h_c t_w$ for I-or H-column and $V_p = 1.8 h_b h_c t_w$ for box column;

t_w——column web thickness;

h_b——beam web depth.

In case of joint panel not satisfying Eq. (7.4.2-1), the web thickness of a H-column or built-up I-column should be thickened. The thickened region shall cover not less than 150mm above the beam top flange and below the beam bottom flange. For hot-rolled H- or I-column, the column web may be also strengthened by welding supplementary plates. The supplementary plates may not extend beyond the web transverse stiffeners, or extend 150mm beyond the web stiffeners. The fillet welds connecting the supplementary plates and web stiffeners shall be designed to transfer the shear force shared by the supplementary plates, the throat thickness shall not be less than 5mm, In case of supplementary plates extending beyond the web stiffeners, the web stiffeners shall be welded to the supplementary plates only. The welds between the supplementary plates and web stiffeners shall be designed to transfer the shear force from the web stiffeners. The thickness of the supplementary plate and its weld shall be designed according to the acting force. The edge of the supplementary plate shall be connected to the column flange by fillet weld. Additional plug welds shall be provided to secure the integral junction of supplementary plates to the column web. The spacing between the centers of plugs shall not exceed 21 $\sqrt{235/f_y}$ times the thickness of the thinner part joined. For light structure, the diagonal stiffeners may be applied to reinforce the column web.

2 The web thickness t_w shall satisfy:

$$t_w \geqslant \frac{h_c + h_b}{90} \tag{7.4.2-2}$$

7.4.3 The column web transverse stiffeners applied at the beam-to-column joints shall satisfy the following:

1 The transverse stiffeners shall be able to resist and transmit the acting force transferred from the beam flange. The thickness of stiffeners shall be 0.5~1.0 times the thickness of beam flange. The width shall satisfy the requirements on force transmitting, detailing, and limiting width-to-thickness ratio.

2 The transverse stiffeners shall be located in alignment with the central axis of the beam flange and welded to the column flange with full penetration T-butt weld. When a beam is perpendicularly connected to the H- or I-column web and rigid connections are required, the web transverse stiffeners should be welded to the column web with full penetration T-butt weld.

3 The diaphragm stiffeners of a box column should be welded to the column flange with full penetration T-butt weld. In case electric arc welding is inaccessible, the consumable nozzle electroslag welding may be applied.

4 When diagonal stiffeners are applied to increase the shear capacity of the joint panel, the stiffener and its connection shall be designed to carry the supplementary shear beyond the shear capacity of column web.

7.5 Calculation of connected plates at joints

7.5.1 The connected plate subject to tension and shear action at the joint shall satisfy the following formula:

$$\frac{N}{\sum(\eta_i A_i)} \leqslant f \tag{7.5.1-1}$$

$$\eta_i = \frac{1}{\sqrt{1 + 2\cos^2 \alpha_i}} \tag{7.5.1-2}$$

where N——tensile force acting on the connected plate;

A_i——area of ith failure section, $A_i = tl_i$, taking net sectional area for bolted (or riveted) connection;

t——thickness of connected plate;

l_i——length of ith failure line, taking the length of most critical failure line on the plate (Fig. 7.5.1);

η_i——equivalent coefficient for tension and shear action on the ith failure section;

α_i——angle between the axis of applied force and the ith failure line.

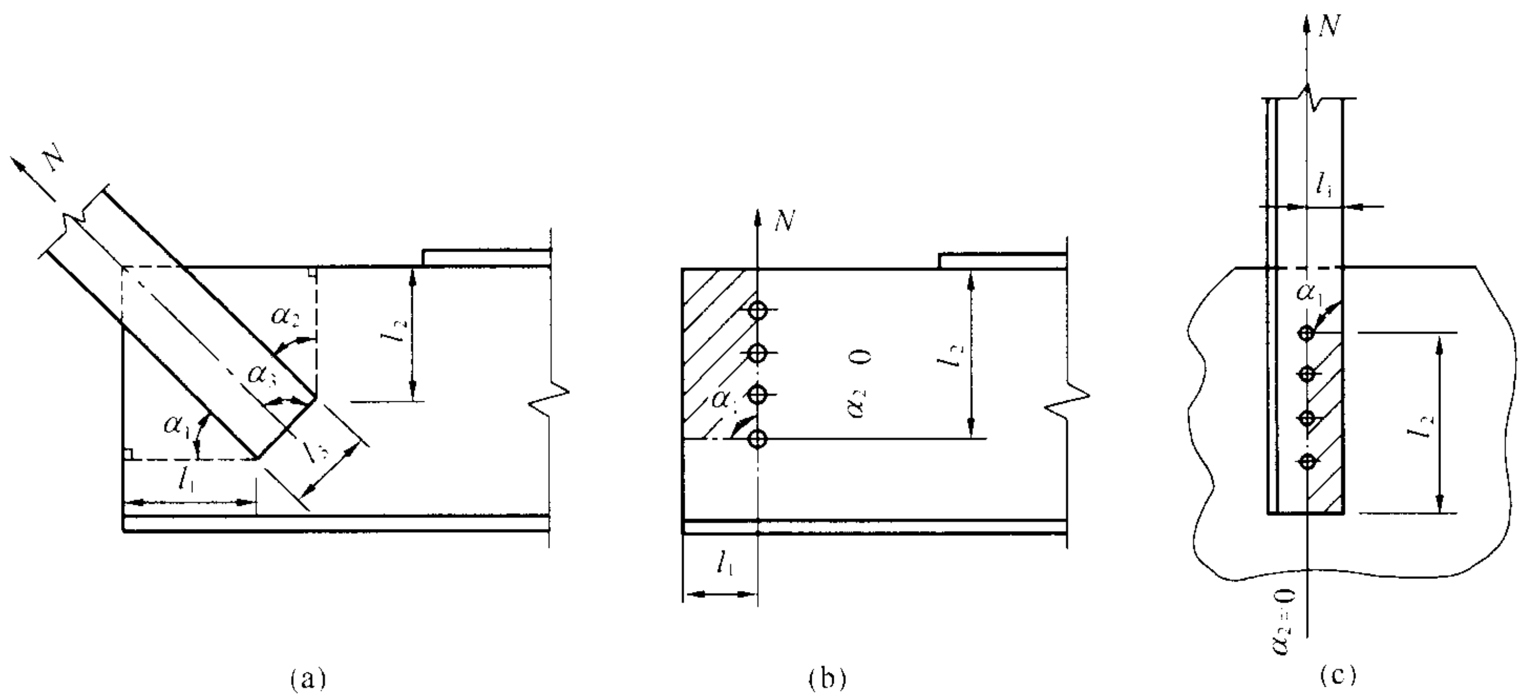


Fig. 7.5.1 Tension and shear tearing of plates

(a) Welded joint; (b) Bolted (riveted) joint; (c) Bolted (riveted) joint

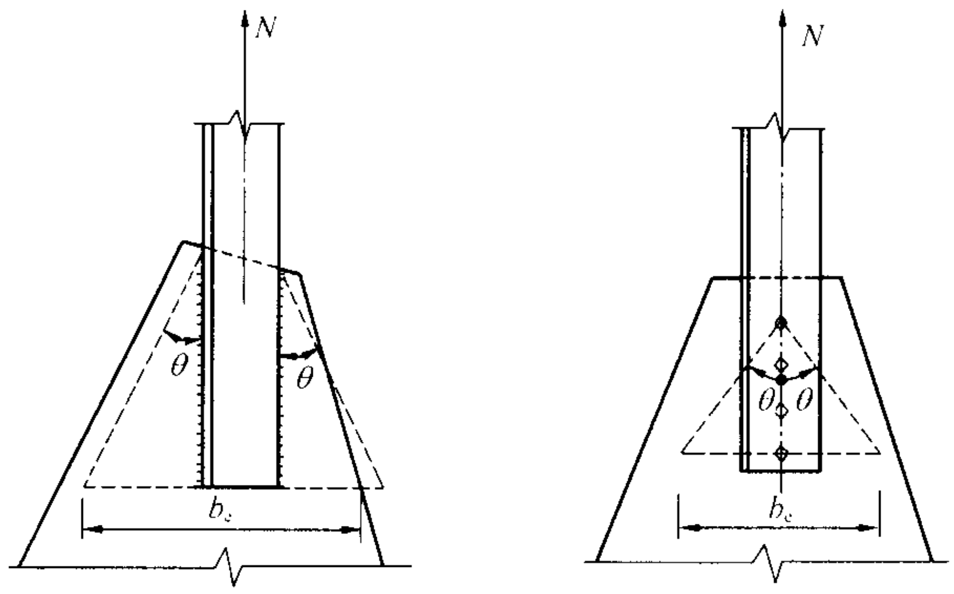
7.5.2 The capacity of truss gusset plate (except members of hot-rolled and welded T-section) can also be calculated using the effective width besides Eq. (7.5.1-1) as follows:

$$\sigma = \frac{N}{b_e t} \leqslant f \tag{7.5.2}$$

where b_e——effective width (Fig. 7.5.2) of plate, reduction for bolt hole shall be made for bolted (or riveted) connection.

7.5.3 The stability of the gusset plate at a truss joint subject to compression from a diagonal member should satisfy the following requirements:

1 The stability of a gusset plate connected to both diagonal and vertical web members may not be calculated if $c/t \leqslant 15\sqrt{235/f_y}$ (where c is the clear distance from the mid point of the end of connecting leg of the compressive diagonal member to the chord member along the diagonal axis). Otherwise, the stability shall be calculated in accordance with Appendix F. Nevertheless, in any circumstance, c/t shall not exceed $22\sqrt{235/f_y}$.



(a) (b)

Fig. 7.5.2 Effective width of plate

Note: θ is the load dispersal angle, taking $\theta = 30°$.

2 For a gusset plate connected to diagonal members only, its stability capacity can be taken as $0.8b_e tf$ if $c/t \leqslant 10\sqrt{235/f_y}$. Otherwise, the stability shall be checked in accordance with Appendix F. Nevertheless, in any circumstance, c/t shall not exceed $17.5\sqrt{235/f_y}$.

7.5.4 The gusset plate shall also satisfy the following requirements in addition to the calculation in Clauses 7.5.1～7.5.3:

1 The angle between the axis of a diagonal member and the edge of gusset plate shall not be less than 15°;

2 The angle between the axis of a diagonal member and the chord member shall be between 30°～60°;

3 The ratio of free edge length l_f of a gusset plate to its thickness t shall not be larger than $60\sqrt{235/f_y}$, otherwise an edge stiffener shall be provided.

7.6 Supports (bearings)

7.6.1 At the end of a beam or truss supported on masonry or concrete (Fig. 8.4.12a), the area of base plate shall be adequately large to transmit the end reaction. The thickness of base plate shall be determined according to the bending moment generated by the reaction.

7.6.2 The cylindrical surface in curved support (Fig. 7.6.2a) and roller support (Fig. 7.6.2b) are linearly contacted with the flat plate. The reaction R shall satisfy the following formula:

$$R \leqslant 40ndlf^2/E \tag{7.6.2}$$

where d——roller diameter of a roller support, or twice the radius r of contact curve in a curved support;

n——number of rollers, and $n = 1$ for curved support;

l——length of the contact line between the flat plate and the roller or the curved surface.

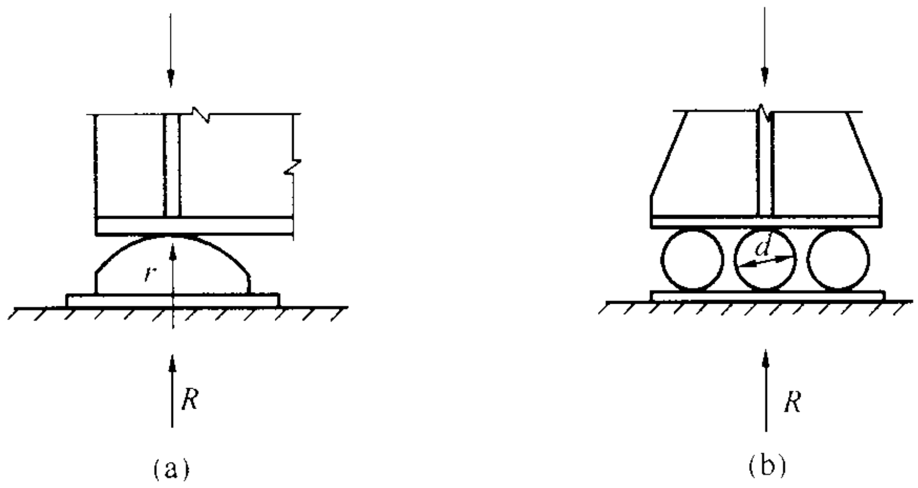


Fig. 7.6.2 Curved support and roller support

(a) Curved support; (b) Roller support

7.6.3 The bearing stress on the cylindrical pin in a hinged pin support (Fig. 7.6.3), where the two cylindrical surfaces have the same radius with a central angle of the free contact surface $\theta \geqslant 90°$, shall be determined by the following formula:

$$\sigma = \frac{2R}{dl} \leqslant f \tag{7.6.3}$$

where d——diameter of the pin;

l——longitudinal length of the contact surface of the pin.

7.6.4 For large span or complex structures, the spherical support or curved support in two directions may be applied to accommodate the displacement and rotation at the supports.

7.6.5 In case of applying composite rubber and steel support to accommodate the end displacement, proper elastomeric materials shall be selected according to the conditions of the project designed and the product properties. The aging of the elastomeric material and possibility of its replacement shall be considered in design.

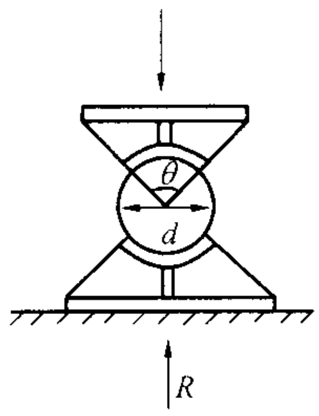


Fig. 7.6.3 Hinged pin support

7.6.6 When the end surface of an axially compressed column or a beam-column is planed, the maximum compressive force will be directly transmitted by the end surface. The connecting welds or bolts shall be determined to resist 15% of the maximum compressive force or the maximum shear force whichever is larger. When a tensile zone occurs on the cross section of a beam-column, the connection shall be designed to resist the maximum tensile force.

8 Detailing requirements

8.1 General stipulations

8.1.1 The steel structures shall be detailed so as to facilitate the fabrication, transportation, erection and maintenance, to make the loads transmitted explicitly, to reduce stress concentration and to avoid multi-axial tensile stress state. For open web structures predominated by wind loading, efforts shall be made to reduce the wind loading area.

8.1.2 For load carrying members and connections in steel structures, the following materials should not be used: plates with thickness less than 4mm, pipes with wall thickness less than 3mm, hot-rolled angles of size less than L45×4 or L56×36×4 for welded structures, hot-rolled angles of size less than L50×5 for bolted or riveted structures.

8.1.3 Whether it is necessary to take special measures such as preheating or post-weld heat treatment for welded structures shall depend upon the combined factors of steel property, plate thickness, welding process, ambient temperature during welding, as well as structural performance requirement etc.. Specifications shall be defined in the design documents.

8.1.4 Reliable bracing system shall be provided in a structure according to its type, composition and loading condition. An independent and spatial stable bracing system shall be provided in each temperature zone (distance between expansion joints) or bays constructed by stages.

8.1.5 Generally, the effect of temperature stress and deformation may not be considered when the length of temperature zone (distance between expansion joints) for single story building or open air structures does not exceed the values defined in Table 8.1.5.

Table 8.1.5 Length of temperature zone(m)

Structural conditions	Longitudinal length of temperature zone (perpendicular to the direction of truss of frame span)	Transverse length of temperature zone (parallel to the direction of truss of frame span)	
		Rigid joint at top of column	Hinged joint at top of column
Buildings with heating and buildings in regions not requiring heating	220	120	150
Hot workshop and buildings without heating in regions requiring heating	180	100	125
Open air structures	120	—	—

Note: 1 When the columns in an industrial building are made of material other than steel, expansion joints shall be provided in accordance with the relevant codes. For cladding structures, expansion joints may be provided separately according to the specific circumstance and referring to the relevant codes.

2 The column bracings in buildings without overhead travelling crane, as well as the column bracings below the crane girders or crane trusses in buildings with overhead travelling crane, should be arranged symmetrically in the middle of a temperature zone. When column bracings are not arranged symmetrically, the distance from the middle of the above mentioned bracings(or from the middle between two sets of bracings in case of two sets of bracing are to be arranged) to the ends of temperature zone should not be longer than 60% of the limit defined in Table 8.1.5.

3 The values in the Table can be increased or reduced if reliable measures or adequate evidences are justified.

8.2 Welded connection

8.2.1 The weld metal shall match the parent metal. When steels of different strength are to be connected, the weld materials matching the low strength steel may be used.

8.2.2 The weld dimensions shall not be increased arbitrarily in design. The three dimensional intersection of welds as well as weld concentration at one location shall be avoided. Weld shall be arranged as symmetrically as possible about the centroid of the connected members.

For corner welding joint with plate thickness beyond 20mm, details that not liable to cause lamellar tearing by welding contraction shall be used.

Note: When steel plates spliced with butt welds, the longitudinal and transverse butt welds may intersect to form cross or T-junctions. In case of T-junction, the spacing between intersecting points shall not be less than 200mm.

8.2.3 The type of grooves for butt welds should be specified on the basis of the plate thickness and construction conditions, and comply with the requirements of the current national standards.

8.2.4 At the splice joints with butt welds, when the plates to be welded are different in width or their thickness differs more than 4mm, a transition slope not steeper than 1:2.5 shall be prepared on one side or both sides of the width or thickness (Fig. 8.2.4). When the plate thickness is different, the weld groove shall be prepared based on the thinner parts according to the requirements in Clause 8.2.3.

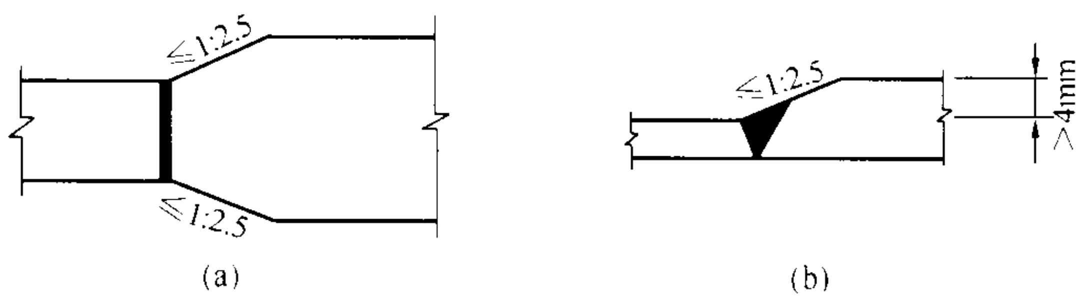


Fig. 8.2.4 Splice of steel plates with different width or thickness

(a) Different width; (b) Different thickness

Note: For the structures directly subjected to dynamic and fatigue loading, the transition slope shall not be steeper than 1:4.

8.2.5 Where partial penetration butt welds are used, the type and size of grooves shall be specified on the structural design drawings, the effective thickness h_e (mm) of the weld shall not be less than $1.5\sqrt{t}$, where t (mm) is the thickness of thicker part welded.

In the structures directly subjected to dynamic loading, partial penetration butt welds should not be used for the welds perpendicular to the applied force.

8.2.6 The angle α between the two legs of a fillet weld is generally 90° (right angle fillet weld). The oblique fillet welds with angle $\alpha > 135°$ or $\alpha < 60°$ should not be used as load-carrying welds (except in tubular structures).

8.2.7 The size of the fillet welds shall comply with the following requirements:

1 The leg size of fillet welds h_f(mm)shall not be less than $1.5\sqrt{t}$, where t(mm)is the thickness of thicker part welded (or thinner part welded when low hydrogen and alkaline electrodes are used). However, for automatic welding, the minimum leg size may be reduced by 1mm. For single sided fillet welds in T-joint, it shall be increased by 1mm. If the thickness of welded part is equal to or less than 4mm, the minimum leg size shall be the same as the thickness of welded parts.

2 The leg size of a fillet weld should not be larger than 1.2 times the thickness of the thinner part welded(except in tubular structures), but the maximum leg size at the edge of plate (thickness is t)shall also comply with the following requirements:

1) When $t \leqslant 6$mm, $h_f \leqslant t$;

2) When $t > 6$mm, $h_f \leqslant t-(1\sim 2)$mm.

The leg size of fillet welds in circular holes or slot holes should not be larger than 1/3 of the hole diameter or 1/3 of the smaller size of the slot.

3 Generally, the sizes of two legs of a fillet weld are equal. When the difference in the thickness of welded parts is large and equal leg size may not satisfy the requirements in Item 1 and 2 of this Clause , unequal leg size may be used . The leg welded to the thinner part shall comply with the requirements in Item 2, and the leg welded to the thicker part shall comply with the requirements in Item 1.

4 The effective length of a side or end fillet weld shall not be less than $8h_f$ and 40mm.

5 The effective length of a side fillet weld should not exceed $60h_f$. The excessive part will not be considered in calculation. The limitation is not applicable if the internal force is uniformly distributed along the full length of the side fillet weld.

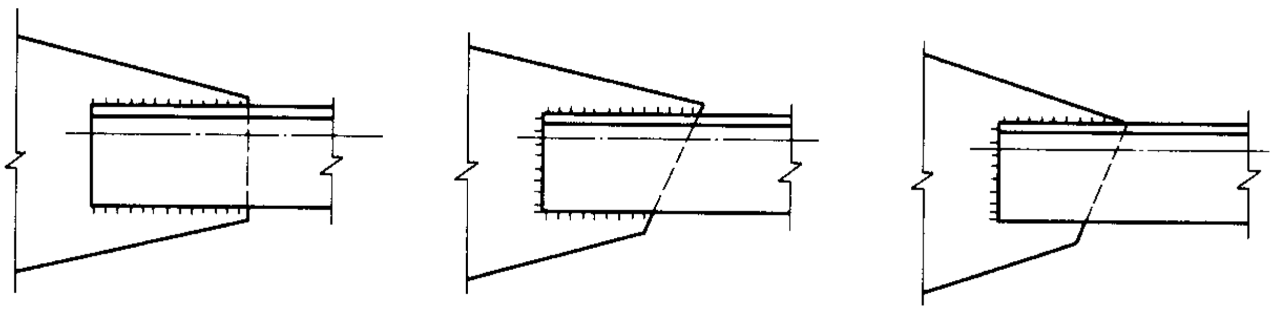
8.2.8 In structures directly subjected to dynamic loading, the surface of fillet welds shall be prepared flat or concave. The ratio of leg size may be 1:1.5 for end fillet weld(the longer leg parallel to the internal force)and 1:1 for side fillet weld.

8.2.9 In secondary members or secondary welded connections, intermittent fillet welds may be used. The length of an intermittent weld segment shall not be less than $10h_f$ or 50mm. The clear distance between the ends of welds shall not be larger than $15t$ (for member in compression) or $30t$ (for member in tension), where t is the thickness of the thinner part welded.

8.2.10 When the end of a plate is connected by two-side fillet welds only, the length of either side fillet weld should not be less than the spacing between the two side fillets welds. Meanwhile, the spacing between the two side fillet welds should not be larger than $16t$(for $t >$ 12mm) or 190mm(for $t \leqslant 12$mm), where t is the thickness of the thinner part welded.

8.2.11 The welds connecting a member to a gusset plate should be two-side fillet welds or three-side around welds(Fig. 8.2.11). L shape around fillet welds can be used for angle members. All the around fillet welds must be carried out continuously round the corners.

8.2.12 Whenever fillet welds at corners of a member end is returned around the corner for a length $2h_f$, it must be carried out continuously round the corner.



(a) (b) (c)

Fig. 8.2.11 Welded connections between member and gusset plate

(a)Two side welds; (b)Three side welds; (c)L shape welds

8.2.13 In lap joints, the lap length shall be neither less than 5 times the thickness of the thinner part welded nor less than 25mm.

8.3 Bolted and riveted connections

8.3.1 The number of permanent bolts(or rivets) used at the end of each member to be connected in a joint or a splice should not be less than two. For connecting the laces on built-up members, single bolt(or rivet)can be used at the end of a lace.

8.3.2 The hole for high strength bolts shall be drilled. The diameter of a bolt hole for high strength bolt in friction type joint shall be 1.5~2.0mm larger than the nominal bolt diameter d. The diameter of a bolt hole for high strength bolt in bearing type joint shall be 1.0~1.5mm larger than the nominal bolt diameter d.

8.3.3 The method of treatment for faying surface of the member in a high strength bolted connection shall be specified in the structural drawings.

8.3.4 The spacing of bolts or rivets shall comply with the requirements specified in Table 8.3.4.

Table 8.3.4 Maximum and minimum allowable distance of bolts or rivets

Case	Location and direction			Maximum allowable distance(taking the lesser)	Minimum allowable distance
Spacing between centers	Outer row(in the direction perpendicular or parallel to loading)			$8d_0$ or $12t$	$3d_0$
	Inner row	In the direction perpendicular to loading		$16d_0$ or $24t$	
		In the direction parallel to loading	Member in compression	$12d_0$ or $18t$	
			Member in tension	$16d_0$ or $24t$	
	In the diagonal direction			—	
Distance from center to member edge	In the direction parallel to loading			$4d_0$ or $8t$	$2d_0$
	In the direction perpendicular to loading	Sheared or manual flame cut edge			$1.5d_0$
		Rolled, automatic flame cut or sawn edge	High strength bolt		
			Other bolts or rivets		$1.2d_0$

Note: 1 d_0 is the hole diameter of bolts or rivets, and t is the thickness of the thinner outside part connected.

2 The maximum spacing of bolt or rivet rows on the edge of a plate connecing to the rigid member(e. g. angle or channel etc.)may be taken as that required for inner rows.

8.3.5 Grade C bolts should be used in joints subject to tensile force along the bolt shank, or may be used in shear connection for the following cases:

1 Secondary connections in structures subject to static loading or indirect dynamic loading;

2 Connections in detachable structures subject to static loading;

3 Erection connections for temporary fixing of members.

8.3.6 For ordinary bolted tension connections subject to direct dynamic loading, double nuts or other effective measures shall be taken to prevent nut loosening.

8.3.7 When rolled sections are spliced with high strength bolts, steel plates should be used as splicing materials.

8.3.8 Rivets with countersunk head or semi-countersunk head shall not be used in connections subject to tension along the rivet shank.

8.3.9 The end-plate (or flange) connected with bolts (or rivets) subject to tension along the bolt shank shall be adequately strengthened in stiffness (such as providing stiffeners) to reduce the negative effects caused by prying force on the tension capacity of the bolts (or rivets).

8.4 Structural members

(Ⅰ) Column

8.4.1 Lacing bars should be used for latticed columns with wider section or subject to comparatively large shear force in the lacing planes.

In battened column, the sum of the linear stiffness of batten plates (or transverse laced bars of rolled steel shape) at a cross section shall not be less than 6 times the linear stiffness of the larger component.

8.4.2 If the ratio of the effective web depth h_0 to the web thickness t_w of a solid-web column, $h_0/t_w > 80\sqrt{235/f_y}$, transverse web stiffeners shall be provided to strengthen the web, and the stiffener spacing shall not be larger than $3h_0$.

The size and details of transverse stiffeners shall comply with the stipulations in Clause 4.3.6.

8.4.3 For latticed columns or larger solid-web columns, transverse diaphragms shall be provided at the section subject to larger horizontal force, and at the end of each shipping unit. The spacing of transverse diaphragms shall be neither more than 9 times the larger width of column section nor 8m.

(Ⅱ) Truss

8.4.4 In welded trusses, the centroidal line of member shall be taken as the axis. In the bolted (or riveted) trusses, the gauge line of bolts or rivets close to the centroidal line of member shall be taken as the axis. At a joint, the axes shall intersect at a common point (except tubular structures).

In case of a truss chord with variable cross section and the eccentricity of axis due to such change not exceeding 5% of the depth of the larger chord section, the effect of axis variation may not be considered.

8.4.5 The joints in a truss can be assumed as hinges for structural analysis. For truss joints connected with gusset plates and members made from H-section or box-sections etc. with larger stiffness, the secondary moment caused by the joint stiffness shall be considered, if the ratio of section depth to the member length (distance between centers of the joints) is larger than 1/10 (for chord members) or 1/15 (for web members).

8.4.6 When the members in a welded truss are connected with gusset plates, the gap between chord and web members, or between web members themselves shall not be less than 20mm. The clear gap between the adjacent toes of filled welds shall not be less than 5mm.

When the members in a truss are connected without gusset plates, the clear gap between the toes of fillet welds connecting adjacent web members shall not be less than 5mm (except tubular structures).

8.4.7 The thickness of gusset plates is generally determined according to the internal forces of the members connected, but shall not be less than 6mm. Tolerances in fabrication and assembly shall be considered in determining the dimensions of a gusset plate.

8.4.8 For a truss with span larger than 36m, hinge-supported at both ends and subject to vertical loading, the horizontal thrust on the supporting members caused by the elastic elongation of bottom chord shall be considered.

(Ⅲ) Beams

8.4.9 The flange of a welded beam is generally formed with one layer of steel plate. When two layers of plates are used, the thickness ratio between the outer plate and inner plate may be taken as 0.5~1.0. In case of the outer plate not covering the full length of the beam, the extended length l_1 of the outer plate beyond the theoretical cutoff point shall comply with the following requirements:

Fillet welds are provided at the end of outer plate:

when $h_f \geq 0.75t, l_1 \geq b$;

when $h_f < 0.75t, l_1 \geq 1.5b$;

Fillet weld are not provided at the end of outer plate:

$$l_1 \geq 2b$$

b and t are the width and thickness of the outer flange plate respectively, and h_f is the leg size of side fillet weld and transverse fillet weld.

8.4.10 The flange of a riveted (or bolted with high strength bolt in friction type joint) beam should not be composed of more than three layers of plates. The area of flange angles should not be less than 30% of the overall flange area. If this requirement could not be satisfied even using the largest angles, side plates may be added (Fig. 8.4.10). The total area of side plates and angles shall not be less than 30% of the overall flange area.

When the outer plates are not covering the full length of the beam, the number of rivets (or high strength bolts in friction type joint) on the extended length beyond the theoretical cutoff point shall be determined according to 1/2 of the tension, or compression capacity of the outer plate net sectional area.

8.4.11 For a welded beam, the corner of transverse stiffeners intersecting with the flange shall be cut off . An oblique cutting may be prepared with $b_s/3$ (but not larger than 40mm) in width and $b_s/2$ (but not larger than 60mm) in depth, see Fig. 8. 4. 11, where b_s is the width of the stiffener.

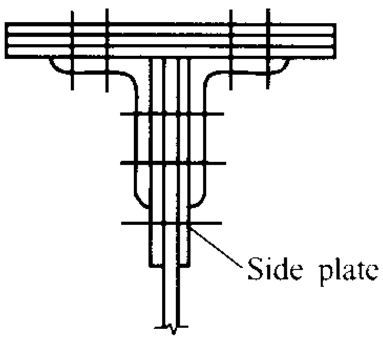


Fig. 8. 4. 10 Flange section of riveted (or high strength bolted in friction-type joint) girder

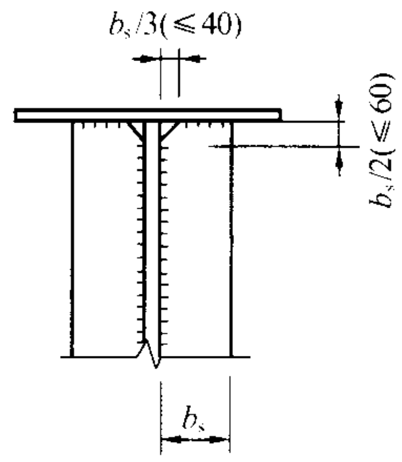


Fig. 8. 4. 11 Corner-cut of stiffeners

8.4.12 When the bottom end of the bearing stiffeners at beam end is designed according to the design value of bearing strength, the bottom end shall be planed and tightly fitted. The extended segment of the stiffener (Fig. 8. 4. 12b) shall not be longer than twice the thickness of the stiffener.

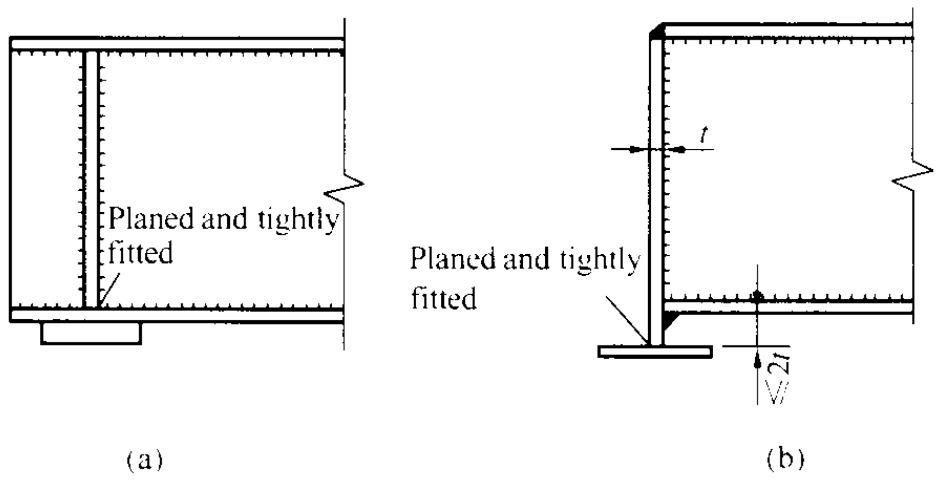


Fig. 8. 4. 12 End support of beam

(a) Flat plate support; (b) Extended support

(Ⅳ) Column base

8.4.13 The anchor bolt of a column base should not be used to resist the horizontal reaction at the bottom of the column. This reaction can be resisted by the friction force between the base plate and the concrete foundation (friction coefficient can be taken as 0.4) or by shear resistant keys provided.

8.4.14 The anchor bolts of a column base shall be embedded in the foundation with a depth sufficient to transmit the tensile force in the anchor bolt by bonding between bolt and concrete. If the embedment depth is limited, the anchor bolt shall be securely fixed to an anchor plate or anchor beam to transmit all the tensile force of the anchor bolt. In this case, the bonding between the anchor bolt and concrete may not be considered.

8.4.15 For the inserted column base, the minimum length d_{in} of the column inserted into the foundation cup may be determined according to Table 8.4.15, but should be neither less than 500mm nor 1/20 of the being erected member length.

Table 8.4.15 Minimum length of steel column inserted into foundation cup

Column section	Solid web section	Latticed column with two components(single cup or two cups)
Minimum inserted length d_{in}	$1.5h_c$ or $1.5d_c$	Larger of $0.5h_c$ and $1.5b_c$(or d_c)

Note: 1 h_c is the depth of the column section(longer side), b_c is the width of the column section, d_c is the outer diameter of the circular tubular column.

2 The distance of the column bottom end to the bottom of foundation cup is generally taken as 50mm. If a column base plate is provided, this distance could be taken as 200mm.

8.4.16 When the column base is encased in concrete structures, the thickness of concrete cover as well as encasing concrete shall not be less than 180mm.

Shear studs should be provided on the flanges of steel column segment encased or covered by concrete. The stud diameter shall not be less than 16mm and horizontal and vertical spacings from center to center shall not be larger than 200mm.

Horizontal stiffeners or diaphragms shall be provided at the top of the encased segment of the encased column base.

8.5 Requirements for crane girders and crane trusses(or similar structures)

8.5.1 The flange of a welded crane girder should be formed with single layer of steel plate. When two layers of plates are used, the outer plate should cover full length of the girder and special measures shall be taken in design and construction to ensure the two layers of top flange plates in close contact.

8.5.2 The crane truss and surge truss should not be used in supporting soaking pit crane, stripper crane or claw crane as well as similar cranes.

8.5.3 The welded crane trusses shall comply with the following requirements:

1 At the truss joint, the gap between web and chord members, a, should not be less than 50mm and the two edges of the gusset plate should be rounded with radius r not less than 60mm; The angle between the edge of the gusset plate and the axis of the web member, θ, shall not be less than 30° (Fig. 8.5.3-1). The start or stop of the welds connecting the chord member of angle sections to the gusset plate shall be set back at least 5mm from the plate edges(Fig. 8.5.3-1a). In T-joint, the combined butt and fillet weld connecting the gusset plate and the H-section chord member shall be furnished with complete penetration. The rounded part of the gusset plate shall be free from defects caused by start or stop of welds. Furthermore, in crane truss for heavy duty cranes, the rounded part shall be ground and provide a smooth transition from gusset to the chord member (Fig. 8.5.3-1b).

2 When welding the filler plate onto a member, the start or stop of the weld shall be set back at least 5mm (Fig. 8.5.3-1c). The filler plate in members of a crane truss for heavy-duty cranes shall be connected with high strength bolts.

3 For trusses using H-section members, the joint details may be formed as shown in Fig. 8.5.3-2.

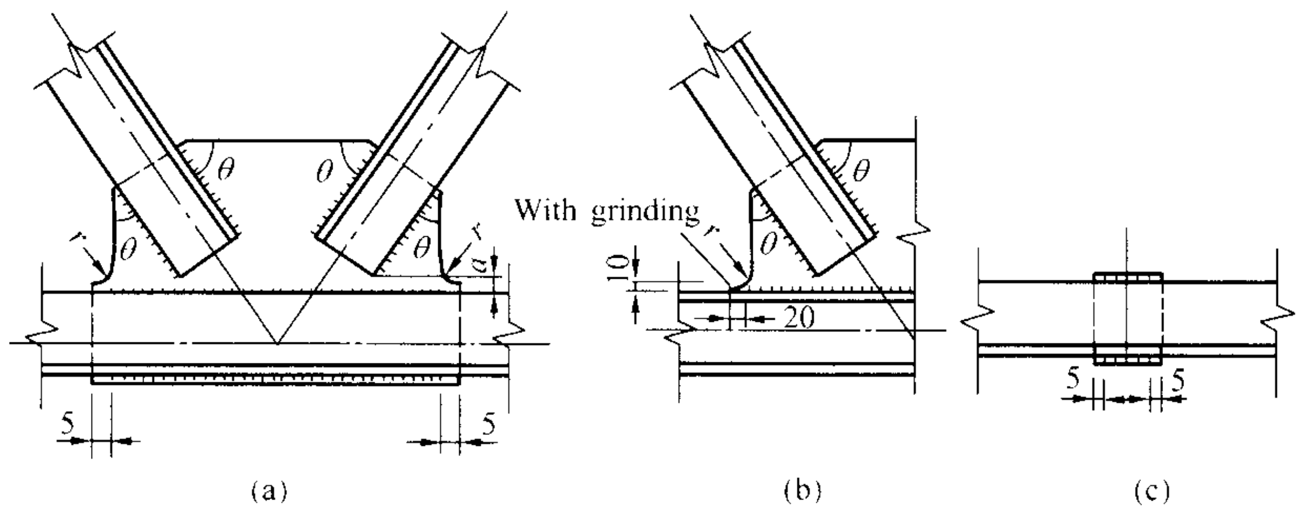


Fig. 8.5.3-1 Joints of crane truss (1)

8.5.4 The complete penetration butt weld with run on/off tabs shall be used for splicing the flange or the web of a crane girder. After removing the run on/off tabs, the locations shall be ground flush. Welding or high strength bolt in friction type joint shall be used for the in-situ splice of fabricated segments of a crane girder or crane truss.

8.5.5 In welded crane girder or crane truss, combined butt and fillet weld in T joint fully penetrated as required in Clause 7.1.1, should be formed as shown in Fig. 8.5.5.

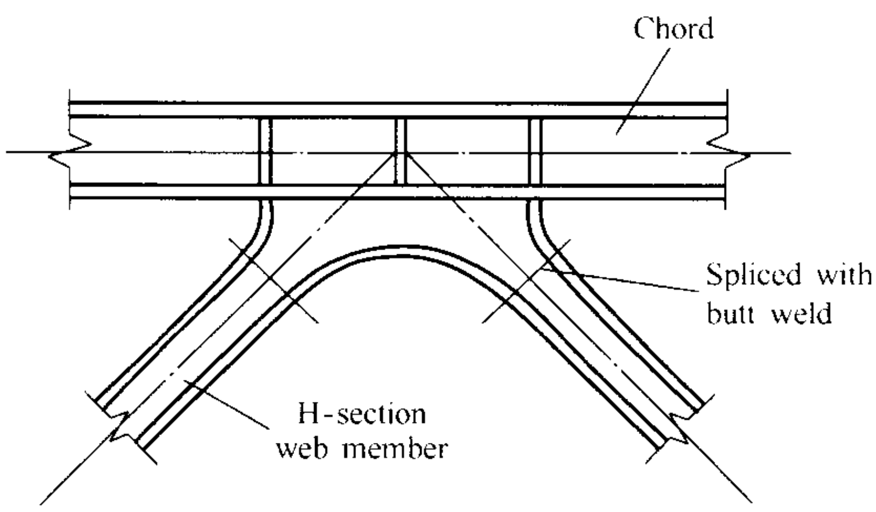


Fig. 8.5.3-2 Joints of crane truss (2)

8.5.6 The width of transverse stiffeners in crane girder should not be less than 90mm. Double and symmetric web stiffeners shall be provided at the end supports of the girder and their ends shall be planed and closely fitted against the top and bottom flange. Intermediate transverse stiffeners shall be planed and closely fitted against the top flange of the girder. The intermediate transverse stiffeners in a crane girder for heavy-duty crane shall also be symmetrically arranged on both sides of the girder web, but for medium or light duty cranes, transverse stiffeners may be provided on one side of the girder web or staggered on both sides of the girder web.

In welded crane girder, the transverse stiffeners (including short stiffeners) shall not be welded to the tension flange of girder, but may be welded to the compression flange of the girder. The end stiffeners may be welded to the top and bottom flanges of the girder. The bottom end of intermediate transverse stiffeners should be 50 ~ 100mm away from the tension flange. The welds connecting this stiffener to the web should not start or stop at the bottom end of the stiffener.

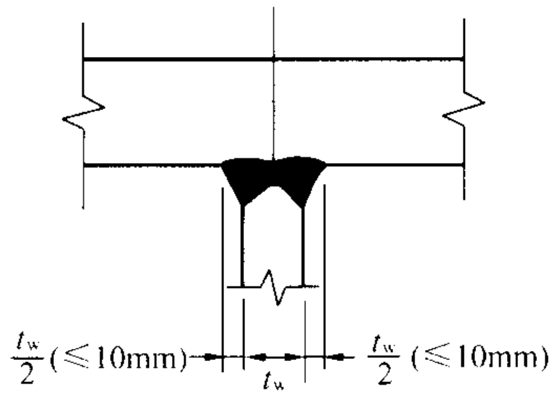


Fig. 8.5.5 Complete penetration of combined butt and fillet weld in T-joint

Welding should not be used for the connection between the bracing and the tension flange of a crane girder (or the bottom chord of a crane truss).

8.5.7 The requirements for detailing of the top chord of a crane truss where the crane rails are fixed directly shall be the same as those for a continuous crane girder.

8.5.8 For a girder supporting heavy-duty crane, high strength bolts in friction type joint should be used for connecting the top flange to the column as well as to the surge truss to transmit the horizontal force. High strength bolts in friction-type joint or welding may be used for connecting the top flange to the surge girder. The details for connecting the end of crane girder to the column shall be so designed as to minimize the additional stress at the connection caused by the flexural deformation of the crane girder.

8.5.9 When the span of a crane truss or a crane girder for heavy duty cranes is equal to or larger than 12m, or the span of a crane girder for light and medium duty cranes is equal to or larger than 18m, an auxiliary truss and horizontal bracing system on bottom flange (or bottom chord) should be provided. The vertical bracing, when provided, should not be located at the location where the maximum vertical deflection of crane girder or crane truss occurs.

Measures in detailing for crane truss shall be taken to prevent the top chord member from torsion caused by the eccentricity of rails.

8.5.10 The edges of the tension flange of a crane girder for heavy-duty crane (or the tension chord of a crane truss) should be rolled or prepared by automatic flame cutting, and the full length of the edges shall be planed if prepared by manual flame cutting or machine shearing.

8.5.11 The attachments for hanging appliance shall not be welded to the tension flange of a crane girder (or tension chord of a crane truss). Striking arc or welding clamp device should also not be allowed on the tension flange.

8.5.12 The joint details of crane rail shall be designed to ensure smooth passage of crane wheels. When welded long rails are used and fastened to the crane girder with clamping plates, a clearance (about 1mm) shall be preserved between the rail and clamping plates to allow the longitudinal expansion and contraction of the rail caused by temperature variations.

8.6 Large span roof structures

8.6.1 Large span roof structure designates roof structures with span equal to or larger than 60m. Plane structures such as truss, rigid frame, arch etc., or spatial structures such as layer grid, shell grid, suspended structures, cable-membrane structures etc. may be used.

8.6.2 In designing large span roof structure, the effects on structural internal forces generated by member deformation, supporting member displacement, boundary restraint conditions and temperature variations etc. shall be considered. Proper supporting system should be selected according to the structural conditions to accommodate the deformation so that the additional internal force can be relieved.

8.6.3 For roof trusses with hanging cranes, the allowable deflection may be taken as 1/500 of the span for deflection calculated according to the characteristic values of permanent plus variable load, and 1/600 of the span for deflection calculated according to the characteristic values of variable load. For the roof truss without hanging cranes, the allowable deflection may be taken as 1/250 of the span for deflection calculated using the characteristic values of permanent plus variable load; and if ceiling is furnished, the allowable deflection may be taken as 1/500 of the span for deflection calculated using the characteristic values of variable load.

8.6.4 For members in the large span roof structure subject to larger internal force or larger dynamic loading, the connections shall be designed with high strength bolts in friction-type joint (except tubular structures).

8.6.5 The large span roof structure shall be checked for erection stage. The erection method and hanging location should be determined by calculations to ensure the strength and stability of the roof structure at each stage of erection.

8.7 Requirements for preventing brittle fracture under low temperature

8.7.1 Proper structural types and fabrication process shall be selected to reduce the stress concentration on the structure as much as possible. In regions where the working temperature is equal to or below −30℃, thinner plates should be used to fabricate the welded members.

8.7.2 In regions where the working temperature is equal to or below −20℃, the details of welded structures should satisfy the following requirements:

1 On the gusset plate of a truss, the clear gap between weld toe of a web member and an adjacent weld toe of a chord member should not be less than $2.5t$ (t is the gusset plate thickness).

2 For the butted or T-butted gusset plate, the gusset plate should be rounded and ground with radius not less than 60mm at the location of butt weld to provide a smooth transition (see Fig. 8.5.3-1b).

3 In a member splice joint, the length of free segment of the splicing plate shall not be less than $5t$, where t is the thickness of splicing plate (Fig. 8.7.2).

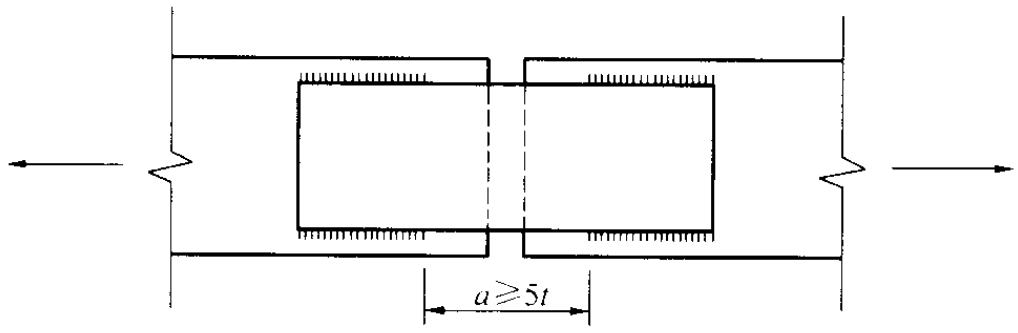


Fig. 8.7.2 Details of splice joint with covering plates

8.7.3 In regions where the working temperature is equal to or below −20℃, the structural fabrication and erection should comply with the following requirements:

1 Bolting should be used for erection connections;

2 The plate edge of a tension member should be rolled or automatically flame cut. If a plate thicker than 10mm is manually flame cut or machine sheared, its edge shall be planed along its full length;

3 The bolt holes shall be drilled, or sub-punched and reamed;

4 The quality level of butt weld shall not be lower than class 2.

8.8 Fabrication, transportation and erection

8.8.1 In dividing a structure into units for transportation, attentions shall be given to economical rationality, convenience of transportation and storage, and ease of assembly, in addition to the structural loading conditions.

8.8.2 The connections used for structural erection shall be reliable in transmitting load, easy to fabricate, simple in details and convenient to adjust.

8.8.3 When welded connections are used for erection, measures shall be taken for member positioning and temporary fixings.

8.9 Protection and heat insulation

8.9.1 In steel structures, in addition to protection against corrosion (painting and galvanizing after cleaning rust), structural details shall be designed as much as possible to avoid places difficult for inspection, cleaning and painting, as well as hidden areas where moisture or lot of dust may accumulate. The closed section shall be sealed with welding along its full length and at the ends.

The paintings and quality level of rust-cleaning required for rust and corrosion protection, as well as the detailing requirements for corrosion protection of steel structures, shall comply with the current national standard of "Code for anticorrosion design of industrial constructions" GB 50046 and "Rust grades and preparation grades of steel surface before application of paints" GB 8923. The quality level of rust-cleaning, as well as the painting (or galvanizing)materials and the thickness of painting(galvanizing)shall be specified in the design documents.

Unless specially required, the material thickness of steel section generally shall not be increased to compensate corrosion.

8.9.2 For buildings designed with working life equal to or longer than 25 years, special measures shall be taken for protection against corrosion for the locations where repainting is inaccessible during the service period.

8.9.3 The segment of column base below the ground level shall be encased with lower strength concrete(thickness of concrete cover shall not be less than 50mm), and the encasing concrete shall be at least 150mm above the ground level. When the bottom of column base is above the ground level, the bottom of column base shall be at least 100mm above the ground level.

8.9.4 The fire protection of steel structures shall comply with the current national standard of "Code for fire protection design of buildings" GBJ 16 and "Code for fire protection design of tall buildings" GB 50045. The fire protection coating on structural members shall be designed according to the fire protection level required for buildings as well as the fire ratings required for different structural members. The properties, thickness and quality requirement of fire protection coating shall comply with the current national standard of "Fire resistive coating for steel structures" GB 14907 and "Code for application technology of fire resistive coating for steel structures" CECS 24.

8.9.5 For structures subject to high temperatures, the following protective measures shall be taken according to different conditions:

1 When a structure may be exposed to hot and molten metal, bricks or other heat insulation materials shall be applied to cover and protect;

2 When the surface of a structure is exposed to radiant heat above 150℃ for long term, or exposed to flame for short term, effective protections shall be provided(e.g. heat insulation layer or water jacket etc.).

9 Plastic design

9.1 General stipulations

9.1.1 The provisions in this chapter are applicable to the fixed end beam, continuous beam as well as single or tow-story rigid frame with solid web members when they are not directly subjected to dynamic loading.

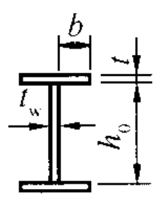
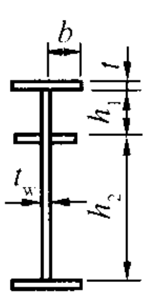
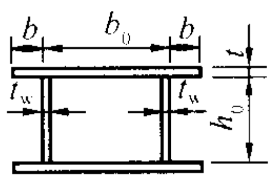
9.1.2 For plastic design of structures or structural members according to the ultimate limit state, the design value of load shall be used. The simplified plastic theory is used to analyze the plastic development and internal forces redistribution on the member sections.

The characteristic value of load and elastic theory of analysis can be used for serviceability limit state.

9.1.3 **For plastic design, the mechanical properties of steel material shall satisfy the tensile to yielding strength ratio $f_u/f_y \geq 1.2$, elongation $\delta_5 \geq 15\%$, the strain ε_u corresponding to tensile strength f_u not less than 20 times yielding strain ε_y.**

9.1.4 The width-to-thickness ratio of plate elements for plastic design shall comply with the stipulations in Table 9.1.4.

Table 9.1.4 Width-to-thickness ratio of plate element

Type of section	Flange	Web
 	$b/t \leq 9\sqrt{\frac{235}{f_y}}$	When $\frac{N}{Af} < 0.37$, $\frac{h_0}{t_w}\left(\frac{h_1}{t_w}, \frac{h_2}{t_w}\right) \leq \left(72 - 100\frac{N}{Af}\right)\sqrt{\frac{235}{f_y}}$ When $\frac{N}{Af} \geq 0.37$, $\frac{h_0}{t_w}\left(\frac{h_1}{t_w}, \frac{h_2}{t_w}\right) \leq 35\sqrt{\frac{235}{f_y}}$
	$b_0/t \leq 30\sqrt{\frac{235}{f_y}}$	Same as the web of I-section

9.2 Calculation of members

9.2.1 The bending strength of a flexural member subject to moment M_x (for H-and I-section, x axis is the strong axis) about its principal axis shall satisfy the following requirement:

$$M_x \leqslant W_{pnx} f \tag{9.2.1}$$

where W_{pnx}——Plastic modulus of net section about x axis.

9.2.2 The shear force V of a flexural member is assumed to be resisted by the section web and the shear strength shall satisty the following requirement:

$$V \leqslant h_w t_w f_v \tag{9.2.2}$$

where h_w, t_w——depth and thickness of section web respectively;

f_v——design value of shear strength of steel.

9.2.3 The strength of a beam-column subject to moment about its principal axis x shall satisfy the following requirements:

When $\frac{N}{A_n f} \leqslant 0.13$

$$M_x \leqslant W_{pnx} f \tag{9.2.3-1}$$

When $\frac{N}{A_n f} > 0.13$

$$M_x \leqslant 1.15\left(1 - \frac{N}{A_n f}\right) W_{pnx} f \tag{9.2.3-2}$$

where A_n——area of net cross section.

The compressive force N in a beam-column shall not exceed $0.6A_n f$. The shear strength of a beam-column shall comply with Eq. (9.2.2).

9.2.4 The stability of a beam-column subject to moment about its principal axis x shall satisfy the following requirement:

1 For in-plane stability:

$$\frac{N}{\varphi_x A f} + \frac{\beta_{mx} M_x}{W_{px} f\left(1 - 0.8\frac{N}{N'_{Ex}}\right)} \leqslant 1 \tag{9.2.4-1}$$

where W_{px}——plastic modulus of gross section about x axis. φ_x, N'_{Ex} and β_{mx} shall be determined in accordance with Clause 5.2.2 as defined for the calculation of in-plane stability.

2 For out-of-plane stability:

$$\frac{N}{\varphi_y A f} + \eta \frac{\beta_{tx} M_x}{\varphi_b W_{px} f} \leqslant 1 \tag{9.2.4-2}$$

φ_y, φ_b and β_{tx} shall be determined in accordance with Clause 5.2.2 as defined for the calculation of out-of-plane stability.

9.3 Allowable slenderness ratio and detailing requirements

9.3.1 The slenderness ratio of compression members should not be larger than $130\sqrt{235/f_y}$.

9.3.2 The lateral bracings must be provided at the member section where plastic hinge occurs. The slenderness ratio λ_y of the member segment between such braced location and its adjacent braced points shall satisfy the following requirements:

When $-1 \leqslant \frac{M_1}{W_{px}f} \leqslant 0.5$

$$\lambda_y \leqslant \left(60 - 40\frac{M_1}{W_{px}f}\right)\sqrt{\frac{235}{f_y}} \tag{9.3.2-1}$$

When $0.5 < \frac{M_1}{W_{px}f} \leqslant 1.0$

$$\lambda_y \leqslant \left(45 - 10\frac{M_1}{W_{px}f}\right)\sqrt{\frac{235}{f_y}} \tag{9.3.2-2}$$

where λ_y——out-of-plane slenderness ratio, $\lambda_y = l_1/i_y$; l_1 is the distance between the laterally braced points; i_y is radius of gyration of member section;

M_1——bending moment at the laterally braced points with distance l_1 from the plastic hinge section; $M_1/(W_{px}f)$ is taken as positive when the segment l_1 is bent in single curvature, or negative when bent in reverse curvature.

On the member segment where no plastic hinges occur, the distance between two laterally braced points shall satisfy the requirements for out-of-plane stability as defined in Chapter 4 and Chapter 5.

9.3.3 The axial force on the lateral bracings provided for reducing the out-of-plane effective length of the member shall be determined in accordance with Clause 4.2.6 or 5.2.8.

9.3.4 All joints and their connections shall have adequate stiffness to ensure the angles between members joined to be unchanged before plastic hinges occur.

The splices and connections of members shall be capable to transmit 1.1 times the maximum design moment at these locations. The moment capacity shall not be less than 0.25 $W_{px}f$.

9.3.5 If the plate elements are cut with manual flame-cutting or machine shearing, the plate edge at the plastic hinge location shall be planed.

The bolt holes on the tensile plate element at the plastic hinge location of a member shall be drilled or punched and reamed.

10 Steel tubular structures

10.1 General stipulations

10.1.1 This chapter deals with the design rules applicable to steel truss structures consisting of circular, square or rectangular hollow sections, with bracing members (abbreviated to bracings) directly welded to chord members (abbreviated to chords), not subject to direct dynamic loading.

10.1.2 The ratio of external diameter to wall thickness of circular tubes shall not exceed $100(235/f_y)$, and the ratio of maximum external dimension to wall thickness of rectangular tubes shall not exceed $40\sqrt{235/f_y}$.

10.1.3 The yield strength and the ratio of the yield strength to ultimate strength f_y/f_u of the material for both hot finished and cold formed tubes shall not exceed 345N/mm^2 and 0.8 respectively. The wall thickness of tubes should not be greater than 25mm.

10.1.4 The axial force distribution in a truss may be determined on the assumption that the members are connected by pinned joints, provided that:

1 The joint geometry is within the range of validity specified in the provisions of this chapter as appropriate.

2 The ratio of the panel length to the member external depth (or diameter) in the plane of the truss is not less than: 12 for chords, and 24 for bracings.

10.1.5 Moments resulting from joint eccentricities may be neglected in calculating the capacity of the joint and that of the chord in tension, provided that the eccentricities are within the following limits:

$$-0.55 \leqslant e/h \text{ (or } e/d) \leqslant 0.25 \qquad (10.1.5)$$

where e——eccentricity, as defined in Fig. 10.1.5;

d——external diameter of chords;

h——external depth of rectangular chords, in the plane of truss.

But the moment $M = \Delta N \times e$ (ΔN is the difference between the axial forces of the chords on the two sides of the joint) need be taken into account in the compression chord design.

10.2 Detailing requirements

10.2.1 The detailing of steel tube joints shall comply with the following requirements:

1 The external dimension of the chords shall not be less than that of the bracings, and the wall thickness of the chords shall not be thinner than that of the bracings. The bracings shall not penetrate into the chords at their connections.

2 The angles between the chord and bracing axes and between adjacent bracing axes

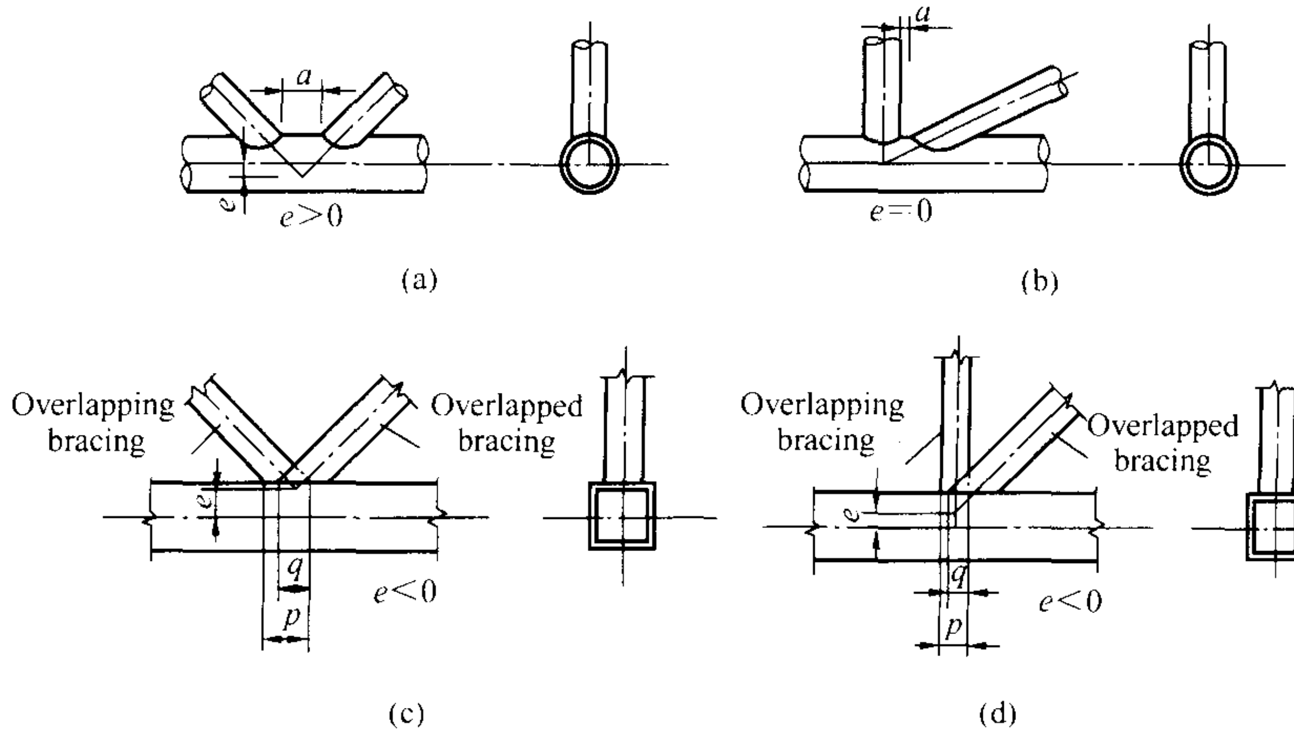


Fig. 10.1.5 Eccentricity, gap and overlap of K and N joints
(a)Gap K-joint; (b)Gap N-joint; (c)Overlap K-joint; (d)Overlap N-joint

should not be less than 30°.

3 Eccentricity should be avoided as far as possible at the joints connecting the bracing to the chord, except at the overlap joints.

4 The weld connecting the bracing to chord shall be continuous around the perimeter of the bracing and made with a smooth transition to the base metal.

5 The end of bracings should be prepared by the automatic tube cutting machine, and the end should not be grooved if the wall thickness of the bracing is less than 6mm.

10.2.2 In gap K and N-joints(Fig. 10.1.5a, b), the gap between the bracings shall not be less than the sum of the wall thickness of the two adjacent bracings.

10.2.3 In overlap K and N-joints(Fig. 10.1.5c, d), the overlap ratio, expressed as a percentage $O_v = q/p \times 100\%$, shall satisfy: $25\% \leqslant O_v \leqslant 100\%$, and the overlap shall be sufficient to ensure that the interconnection weld of the bracings is adequate for satisfactory force transfer from one bracing to the other.

10.2.4 Where overlapping bracings have different thicknesses, the thinner member shall overlap the thicker one. Where overlapping bracings are of different strength grades, the member with the lower(yield) strength shall overlap the member with the higher(yield) strength.

10.2.5 In welded joints between chord and bracing, the connection should be established around the entire perimeter of the hollow section by means of a fillet weld, or part butt weld and part fillet weld. In the region where the angle between the walls of bracing and chord exceeds or equals 120°, butt weld or grooved fillet weld may be used. The leg size of fillet weld should not be greater than twice the wall thickness of the bracing to be connected.

10.2.6 Steel tube members shall be properly reinforced at the locations of relatively large transverse loads to prevent excessive local deformations. Holes shall be avoided at major stressed locations of the steel tube member. If holes are necessary thereat, appropriate measures of strengthening shall be taken.

10.3 Capacity of members and joints

10.3.1 The design values of the internal axial forces both in the bracings and the chords for the directly welded steel tubular structures shall not exceed the design values of the capacity of relevant members determined according to Chapter 5 of this code, and the design value of the internal axial force in the bracings shall also not exceed the design value of the capacity of the joint.

10.3.2 The capacity of weld around the perimeter of the bracing at joint shall not less than the capacity of the joint.

The weld connecting a bracing to a chord may be assumed as a fillet weld around the entire perimeter of the bracing, calculated according to Formula(7.1.3-1), but taking $\beta_f = 1$. The effective thickness of the fillet weld is variable around the perimeter of bracing, and $0.7h_f$ may be taken as the average effective thickness when the bracing is axially loaded, The effective length of the weld may be calculated according to the following provisions:

1 In the circular tube structures, the length of intersecting curve between bracing and chord may be taken as follows:

When $d_i/d \leqslant 0.65$

$$l_w = (3.25d_i - 0.025d)\left(\frac{0.534}{\sin\theta_i} + 0.466\right) \tag{10.3.2-1}$$

When $d_i/d > 0.65$

$$l_w = (3.81d_i - 0.389d)\left(\frac{0.534}{\sin\theta_i} + 0.466\right) \tag{10.3.2-2}$$

where d, d_i——external diameter of the chord and bracing respectively;

θ_i——included angle between the chord and bracing.

2 In the rectangular tube structures, the effective length of intersecting curve between bracing and chord should be calculated as follows:

For gap K and N-joints:

When $\theta_i \geqslant 60°$

$$l_w = \frac{2h_i}{\sin\theta_i} + b_i \tag{10.3.2-3}$$

When $\theta_i \leqslant 50°$

$$l_w = \frac{2h_i}{\sin\theta_i} + 2b_i \tag{10.3.2-4}$$

When $50° < \theta_i < 60°$, a linear interpolation shall be taken.

For T, Y and X-joints(Fig.10.3.4):

$$l_w = \frac{2h_i}{\sin\theta_i} \tag{10.3.2-5}$$

where h_i, b_i——external depth and width of the bracing respectively.

For the welded joints between the circular bracing and rectangular chord, the effective length of weld should be taken as the difference between the length of intersecting curve and the external diameter d_i of the bracing.

10.3.3 The capacities of directly welded joints between circular tube chords and bracings shall be determined from the following provisions. The range of validity is: $0.2 \leqslant \beta \leqslant 1.0$; $d_i/t_i \leqslant 60$; $d/t \leqslant 100$; $\theta \geqslant 30°$; $60° \leqslant \phi \leqslant 120°$ (where β——bracing to chord external diameter ratio; d_i, t_i——external diameter and wall thickness of bracing; d, t——external diameter and wall thickness of chord; θ——included angle between the chord and bracing; ϕ——angle between bracing planes in multiplanar joints).

To ensure the chord strength at the joint, the axial forces in the bracings shall not exceed the design values of capacity determined as follows:

1 X-joints (Fig. 10.3.3a):

1) The design value of capacity, N_{cX}^{pj}, of the bracing in compression at joints shall be calculated by the following formula:

$$N_{cX}^{pj} = \frac{5.45}{(1 - 0.81\beta)\sin\theta}\psi_n t^2 f \qquad (10.3.3\text{-}1)$$

where ψ_n——parameter: $\psi_n = 1 - 0.3\frac{\sigma}{f_y} - 0.3\left(\frac{\sigma}{f_y}\right)^2$; for chord in tension at one or both sides of the joint, $\psi_n = 1.0$;

f——design value of steel strength in tension, compression and bending;

f_y——yield strength of the chord steel;

σ——lesser absolute value of the compressive axial stress in the chords at both sides of the joint.

2) The design value of capacity, N_{tX}^{pj}, of the bracing in tension at joints shall be calculated by the following formula:

$$N_{tX}^{pj} = 0.78\left(\frac{d}{t}\right)^{0.2} N_{cX}^{pj} \qquad (10.3.3\text{-}2)$$

2 T or Y-joints (Fig. 10.3.3b, c):

1) The design value of capacity, N_{cT}^{pj}, of the bracing in compression at joints shall be calculated by the following formula:

$$N_{cT}^{pj} = \frac{11.51}{\sin\theta}\left(\frac{d}{t}\right)^{0.2}\psi_n\psi_d t^2 f \qquad (10.3.3\text{-}3)$$

where ψ_d——parameter, taken as:

$$\psi_d = 0.069 + 0.93\beta \qquad \text{When } \beta \leqslant 0.7;$$

$$\psi_d = 2\beta - 0.68 \qquad \text{When } \beta > 0.7.$$

2) The design value of capacity, N_{tT}^{pj}, of the bracing in tension at joints shall be calculated by the following formulae:

When $\beta \leqslant 0.6$

$$N_{\mathrm{tT}}^{\mathrm{pj}} = 1.4N_{\mathrm{cT}}^{\mathrm{pj}} \tag{10.3.3-4}$$

When $\beta > 0.6$

$$N_{\mathrm{tT}}^{\mathrm{pj}} = (2 - \beta)N_{\mathrm{cT}}^{\mathrm{pj}} \tag{10.3.3-5}$$

3 K-joints (Fig.10.3.3d):

1) The design value of capacity, $N_{\mathrm{cK}}^{\mathrm{pj}}$, of the bracing in compression at joints shall be calculated by the following formula:

$$N_{\mathrm{cK}}^{\mathrm{pj}} = \frac{11.51}{\sin\theta_{\mathrm{c}}}\left(\frac{d}{t}\right)^{0.2}\psi_{\mathrm{n}}\psi_{\mathrm{d}}\psi_{\mathrm{a}}t^{2}f \tag{10.3.3-6}$$

where θ_{c}——angle between the axes of the compressive bracing and the chord;

ψ_{a}——parameter, calculated as follows:

$$\psi_{\mathrm{a}} = 1 + \frac{2.19}{1 + \frac{7.5a}{d}}\left(1 - \frac{20.1}{6.6 + \frac{d}{t}}\right)(1 - 0.77\beta) \tag{10.3.3-7}$$

a——gap between the two bracings; taking $a = 0$ when $a < 0$.

2) The design value of capacity, $N_{\mathrm{tK}}^{\mathrm{pj}}$, of the bracing in tension at joints shall be calculated by following formula:

$$N_{\mathrm{tK}}^{\mathrm{pj}} = \frac{\sin\theta_{\mathrm{c}}}{\sin\theta_{\mathrm{t}}}N_{\mathrm{cK}}^{\mathrm{pj}} \tag{10.3.3-8}$$

where θ_{t}——angle between the axes of the tensile bracing and the chord.

4 TT-joints(Fig. 10.3.3e):

1) The design value of capacity, $N_{\mathrm{cTT}}^{\mathrm{pj}}$, of the bracing in compression at joints shall be calculated by the following formula:

$$N_{\mathrm{cTT}}^{\mathrm{pj}} = \psi_{\mathrm{g}}N_{\mathrm{cT}}^{\mathrm{pj}} \tag{10.3.3-9}$$

where $\psi_{\mathrm{g}} = 1.28 - 0.64\frac{g}{d} \leqslant 1.1$, g is the transverse gap between the two bracings.

2) The design value of capacity, $N_{\mathrm{tTT}}^{\mathrm{pj}}$, of the bracing in tension at joints shall be calculated by the following formula:

$$N_{\mathrm{tTT}}^{\mathrm{pj}} = N_{\mathrm{tT}}^{\mathrm{pj}} \tag{10.3.3-10}$$

5 KK-joints(Fig.10.3.3f):

The design values of capacity, $N_{\mathrm{cKK}}^{\mathrm{pj}}$ or $N_{\mathrm{tKK}}^{\mathrm{pj}}$, of the bracings in compression or in tension at KK-joints shall be equal to 0.9 times the relevant design values of capacity, $N_{\mathrm{cK}}^{\mathrm{pj}}$ or $N_{\mathrm{tK}}^{\mathrm{pj}}$ respectively, of the bracings at a K-joint.

10.3.4 The capacities of directly welded joints composed of rectangular tubes shall be determined from the following provisions(Fig.10.3.4). The range of validity is given in Table 10.3.4.

To ensure the rectangular chord strength at the joints, both the axial forces, N_i in brac-

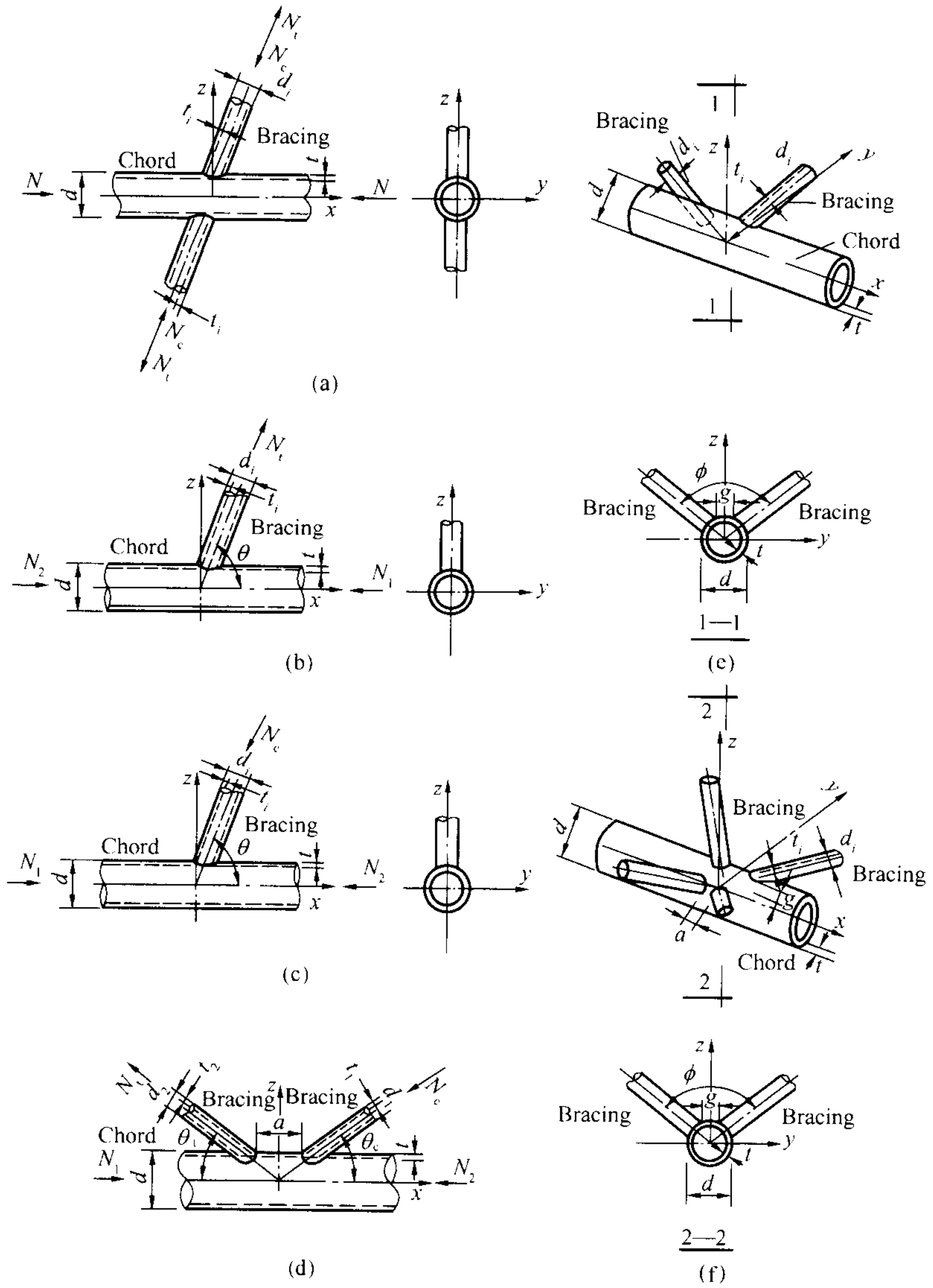


Fig. 10.3.3 Types of joint in circular tube structures

(a) X-joint; (b) T and Y-joint in tension; (c) T and Y-joint in compression; (d) K-joint; (e) TT-joint; (f)KK-joint

ing and N in chord, shall not exceed the design value of joint capacity determined as follows:

1 T, Y and X-joints with rectangular bracings(Fig. 10.3.4a, b):

1) When $\beta \leqslant 0.85$, the design value of capacity, N_i^{pj}, of the bracing at joints shall be calculated by following formula:

$$N_i^{pj} = 1.8\left(\frac{h_i}{bc\sin\theta_i} + 2\right)\frac{t^2 f}{c\sin\theta_i}\psi_n \tag{10.3.4-1}$$

$$c = (1 - \beta)^{0.5}$$

where ψ_n——parameter,

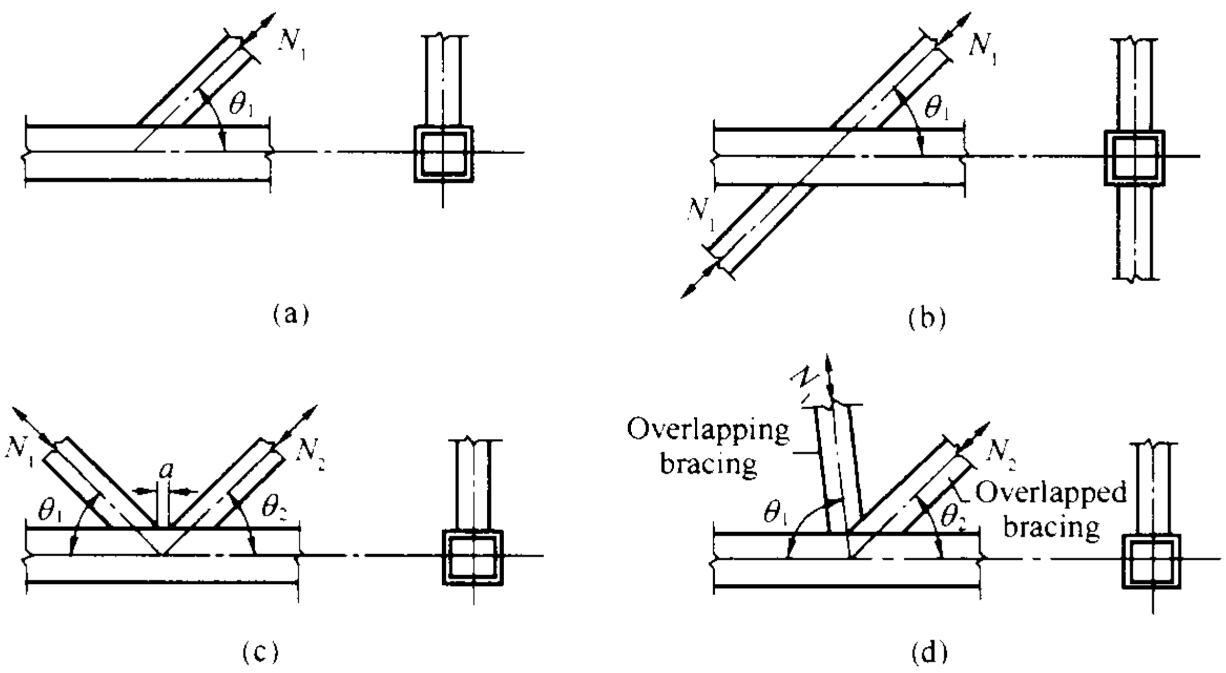


Fig. 10.3.4 Directly welded uniplanar joints composed of rectangular tubes

(a)T、Y-joint; (b)X-joint; (c)Gap K,N-joint; (d)Overlap K,N-joint

$$\psi_n = 1.0 - \frac{0.25}{\beta} \cdot \frac{\sigma}{f};\text{for chord in compression;}$$

$$\psi_n = 1.0;\text{for chord in tension;}$$

σ——the larger absolute value of the compressive axial stress in the chord at both sides of the joint.

2) When $\beta = 1.0$, the design value of capacity, N_i^{pj}, of the bracing at joints shall be calculated by following formula:

$$N_i^{pj} = 2.0\left(\frac{h_i}{\sin\theta_i} + 5t\right)\frac{tf_k}{\sin\theta_i}\psi_n \tag{10.3.4-2}$$

For X-joints, when $\theta_i < 90°$ and $h \geqslant h_i/\cos\theta_i$, it shall furthermore be checked by the following formula:

$$N_i^{pj} = \frac{2htf_v}{\sin\theta_i} \tag{10.3.4-3}$$

where f_k——design value of chord strength : for bracing in tension , $f_k = f$; for bracing in compression, $f_k = 0.8\varphi f$, for T, Y joints and $f_k = (0.65 \sin\theta_i)\varphi f$, for X joint; φ is the stability factor of axially loaded compression members determined according to the slenderness ratio $\lambda = 1.73(\frac{h}{t} - 2)(\frac{1}{\sin\theta_i})^{0.5}$;

f_v——design value of shear strength of chord steel.

3) When $0.85 < \beta < 1.0$, the design value of capacity of bracing at joints shall be determined by linear interpolation of Formulae (10.3.4-1)and (10.3.4-2)or (10.3.4-3) according to β. In addition, it shall not exceed the values calculated by the following two formulae:

$$N_i^{pj} = 2.0(h_i - 2t_i + b_e)t_i f_i \tag{10.3.4-4}$$

$$b_e = \frac{10}{b/t} \cdot \frac{f_y t}{f_{yi} t_i} \cdot b_i \leqslant b_i$$

Table 10.3.4 Geometric parameter range of validity for the joints between rectangular tubes

Type of member section		Type of joints	Joint parameters, $i=1$ or $i=2$ for bracings; j—for overlapped bracing					
			$\frac{b_i}{b},\frac{h_i}{b}$ $\left(\text{or }\frac{d_i}{b}\right)$	$\frac{b_i}{t_i},\frac{h_i}{t_i}\left(\text{or }\frac{d_i}{t_i}\right)$ compression	$\frac{b_i}{t_i},\frac{h_i}{t_i}\left(\text{or }\frac{d_i}{t_i}\right)$ tension	$\frac{h_i}{b_i}$	$\frac{b}{t}$, $\frac{h}{t}$	a or O_v $\frac{b_i}{b_j},\frac{t_i}{t_j}$
Rectangular chords	Rectangular bracings	T, Y, X	$\geqslant 0.25$	$\leqslant 37\sqrt{\frac{235}{f_{yi}}}$ $\leqslant 35$	$\leqslant 35$	$0.5 \leqslant \frac{h_i}{b_i} \leqslant 0.2$	$\leqslant 35$	
		Gap K, N	$\geqslant 0.1+\frac{0.01b}{t}$ $\beta \geqslant 0.35$					$0.5(1-\beta) \leqslant \frac{a}{b} \leqslant 1.5(1-\beta)^*$ $a \geqslant t_1+t_2$
		Overlap K, N	$\geqslant 0.25$	$\leqslant 33\sqrt{\frac{235}{f_{yi}}}$			$\leqslant 40$	$25\% \leqslant O_v \leqslant 100\%$ $\frac{t_i}{t_j} \leqslant 1.0$, $1.0 \geqslant \frac{b_i}{b_j} \geqslant 0.75$
	Circular bracings		$0.4 \leqslant \frac{d_i}{b} \leqslant 0.8$	$\leqslant 44\sqrt{\frac{235}{f_{yi}}}$	$\leqslant 50$	Limitations as above but with d_i replacing b_i		

Note: 1 At the symbol*, if $a/b > 1.5(1-\beta)$, treat as a T or Y-joint;

2 b_i, h_i, t_i——external width, depth and wall thickness respectively of the i-th rectangular bracing;

d_i, t_i——external diameter and wall thickness respectively of the i-th circular bracing;

b, h, t——external width, depth and wall thickness respectively of rectangular chord;

a——gap between the bracings, see Fig. 10.3.4;

O_v——overlap ratio, see the Clause 10.2.3;

β——parameter; $\beta = \frac{b_i}{b}$ or $\frac{d_i}{b}$, for T, Y, X joints; $\beta = \frac{b_1+b_2+h_1+h_2}{4b}$ or $\beta = \frac{d_1+d_2}{2b}$, for K, N joints;

f_{yi}——steel yield strength of the i-th bracing.

When $0.85 \leqslant \beta \leqslant 1 - \frac{2t}{b}$

$$N_i^{pj} = 2.0\left(\frac{h_i}{\sin\theta_i} + b_{ep}\right)\frac{tf_v}{\sin\theta_i} \tag{10.3.4-5}$$

$$b_{ep} = \frac{10}{b/t} \cdot b_i \leqslant b_i$$

where h_i, t_i, f_i——external depth, wall thickness, design value of tensile (compressive and bending) strength of bracing respectively.

2 Gap K and N-joints with rectangular bracings (Fig. 10.3.4c):

1) The design value of capacity for any bracing at the joints shall not exceed the lesser value determined from the following formulae:

$$N_i^{pj} = 1.42\frac{b_1 + b_2 + h_1 + h_2}{b\sin\theta_i}(\frac{b}{t})^{0.5}t^2 f\psi_n \qquad (10.3.4\text{-}6)$$

$$N_i^{pj} = \frac{A_v f_v}{\sin\theta_i} \qquad (10.3.4\text{-}7)$$

$$N_i^{pj} = 2.0(h_i - 2t_i + \frac{b_i + b_e}{2})t_i f_i \qquad (10.3.4\text{-}8)$$

When $\beta \leqslant 1 - \frac{2t}{b}$, it shall furthermore not exceed:

$$N_i^{pj} = 2.0(\frac{h_i}{\sin\theta_i} + \frac{b_i + b_{ep}}{2})\frac{tf_v}{\sin\theta_i} \qquad (10.3.4\text{-}9)$$

where A_v——shear area of the chord determined from the following:

$$A_v = (2h + \alpha b)t \qquad (10.3.4\text{-}10)$$

$$\alpha = \sqrt{\frac{3t^2}{3t^2 + 4a^2}} \qquad (10.3.4\text{-}11)$$

2) The design value of capacity in the axially loaded chord at the gap of the joint shall be calculated by following formula:

$$N^{pj} = (A - \alpha_v A_v)f \qquad (10.3.4\text{-}12)$$

where α_v——interaction factor used to consider the reduction of axial load capacity of the chord, due to shear, determined from the following:

$$\alpha_v = 1 - \sqrt{1 - (\frac{V}{V_p})^2} \qquad (10.3.4\text{-}13)$$

$$V_p = A_v f_v$$

V——shear force in the chord at the gap, taking the vertical component of the force in any one bracing.

3 Overlap K and N-joints with rectangular bracings (Fig. 10.3.4d):

The design values of capacity of overlapping bracing shall be calculated by Formulae (10.3.4-14) through (10.3.4-16), according to different overlap ratios (subscript j to denote the overlapped bracing):

1) When $25\% \leqslant O_v < 50\%$

$$N_i^{pj} = 2.0\left[(h_i - 2t_i)\frac{O_v}{0.5} + \frac{b_e + b_{ej}}{2}\right]t_i f_i \qquad (10.3.4\text{-}14)$$

$$b_{ej} = \frac{10}{b_j/t_j} \cdot \frac{t_j f_{yj}}{t_i f_{yj}} \cdot b_i \leqslant b_i$$

2) When $50\% \leqslant O_v < 80\%$

$$N_i^{pj} = 2.0\left(h_i - 2t_i + \frac{b_e + b_{ej}}{2}\right)t_i f_i \qquad (10.3.4\text{-}15)$$

3) When $80\% \leqslant O_v \leqslant 100\%$

$$N_i^{pj} = 2.0\left(h_i - 2t_i + \frac{b_i + b_{ej}}{2}\right)t_i f_i \qquad (10.3.4\text{-}16)$$

The capacity of overlapped bracing shall satisfy the following requirement:

$$\frac{N_j^{pj}}{A_j f_{yj}} \leqslant \frac{N_i^{pj}}{A_i f_{yi}} \qquad (10.3.4\text{-}17)$$

4 Various joints with circular bracings:

For the joints with circular bracings, all the above capacity formulae are still applicable, but with d_i replacing b_i and h_i, multiplying the right side of the above formulae by $\pi/4$, and taking the α equal to zero in the Formula (10.3.4-10).

$$\frac{N_j^{pj}}{A_j f_{yj}} \leqslant \frac{N_i^{pj}}{A_i f_{yi}} \tag{10.3.4-17}$$

4 Various joints with circular bracings:

For the joints with circular bracings, all the above capacity formulae are still applicable, but with d_i replacing b_i and h_i, multiplying the right side of the above formulae by $\pi/4$, and taking the α equal to zero in the Formula (10.3.4-10).

11 Composite steel and concrete beams

11.1 General stipulations

11.1.1 The provisions in this chapter generally apply to the composite beam composed of ncrete flange and steel beam interconnected by shear connectors, not subject to dynamic ading directly.

The flange of composite beam may be in-situ concrete slab, concrete laminated slab or ofiled composite slab with profiled steel sheeting. The concrete slab shall be designed in ac- rdance with the current national standard "Code for design of concrete structures" GB 50010.

11.1.2 The effective width of concrete flange b_e (Fig. 11.1.2) shall be determined as fol- ws:

$$b_e = b_0 + b_1 + b_2 \tag{11.1.2}$$

where b_0——width of the top of concrete haunch, calculated with $\alpha = 45°$, when $\alpha < 45°$, where α is the inclination of haunch, and taken as the width of top flange of steel beam when there is no concrete haunch;

b_1, b_2——effective widths of concrete flanges on the outer side and inner side of the steel beam respectively, each taken as the lesser of 1/6 of the beam span l and 6 times the thickness of concrete flange h_{c1}. Besides, b_1 shall not exceed the actual overhanging width of the concrete flange s_1; b_2 shall not exceed 1/2 of the clear distance s_0 between two adjacent top flanges of steel beams or concrete haunches. b_1 is equal to b_2 in Eq(11.1.2) when the composite beam is an intermediate beam.

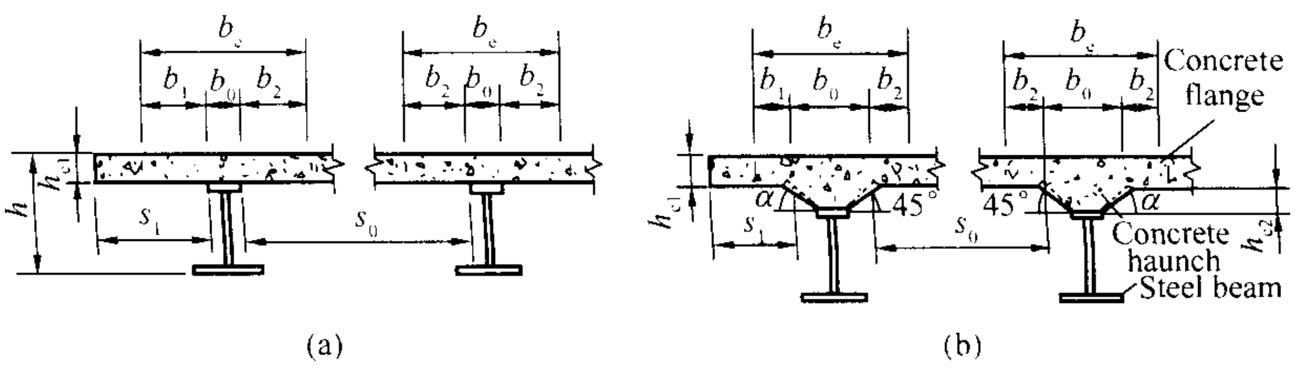


Fig. 11.1.2 Effective width of concrete flange

In Fig. 11.1.2, h_{c1} is the thickness of concrete flange. When using the composite slab with profiled steel sheeting, h_{c1} is equal to the overall thickness of composite slab minus the pth of ribs of profiled steel sheeting. However, when determining the effective width of mposite slab with profiled steel sheeting, h_{c1} may be taken as the overall thickness of con- rete slab at ribs. h_{c2} is the depth of concrete haunch; $h_{c2} = 0$ when there is no haunch.

11.1.3 The deflection of composite beams (including composite beams with partial shear connection and composite beams composed of steel beams and composite slabs) shall be determined using the elastic analysis method, and the bending stiffness of composite beams shall be reduced in accordance with Clause 11.4.2 to consider the effect of slip between the concrete slab and the steel beam.

For continuous composite beams, within the region of $0.15l$ (l is the span of beam) each side from the internal support, the effect of concrete in tension to the stiffness of the beam is neglected, but the effect of longitudinal reinforcement within the effective width of concrete slab b_e shall be considered. In other regions, the reduced stiffness shall be used. Besides checking the deflection by the method mentioned above, the maximum crack width w_{max} in negative moment region shall be checked according to the current national standard "Code for design of concrete structures" GB 50010.

The cross-section of concrete haunch may be neglected in the calculation of strength, deflection and crack of composite beams.

The longitudinal shear of concrete slab shall be checked according to related specifications.

11.1.4 When the steel beam is not propped during construction of the composite beam, the weight of materials and the construction loads applied prior the hardening of concrete shall be carried by the steel beam. The strength, stability and deflection of the steel beam shall be calculated according to the provisions in Chapter 3 and Chapter 4. In service stage after construction, the deflection of composite beam by additional loads shall be superimposed upon the deflection of steel beam during construction.

11.1.5 While satisfying the strength and deflection requirements, if the shear connectors at the interface of composite beams could not provide adequate longitudinal horizontal shearing resistance to guarantee the moment resistance capacity at the cross-section of maximum positive moment to develop sufficiently, the composite beam may be designed as that with partial shear connection. The composite beam using profiled steel sheeting as formwork may also be designed as composite beams with partial shear connection. The partial shear connection may only be used in composite beams with uniform cross-section, whose span should not be greater than 20m.

11.1.6 When the strength of composite beams is calculated according to this chapter, considering development of plasticity over the entire composite cross-section, the design value of strength f of the steel beam shall be taken as that specified in Clauses 3.4.1 and 3.4.2. If the thicknesses of the composing plate elements are not equal, the design value of steel strength of the thickest plate may be taken as the unified one. The design value of strength of reinforcement bars in negative moment region shall be taken in accordance with the provisions of the current national standard "Code for design of concrete structures" GB 50010. If elastic analysis method is used in calculating continuous composite beams, the moment redistribution coefficient considering the plastic development should not be greater than 15%.

In the compression region of composite beams, the width-to-thickness ratio of steel plates shall satisfy the requirements of Clause 9.1.4 in Chapter 9.

11.2 Design of composite beams

11.2.1 The bending strength of composite beams with full shear connection shall be calculated according to the following provisions:

1 The positive moment region:

1) When the plastic neutral axis is within the concrete flange (Fig. 11.2.1-1), that is, when $Af \leqslant b_e h_{c1} f_c$:

$$M \leqslant b_e x f_c y \tag{11.2.1-1}$$

$$x = Af/(b_e f_c) \tag{11.2.1-2}$$

where M——design value of positive moment;

A——cross-sectional area of steel beam;

x——depth of the compression zone of concrete flange;

y——distance between the stress resultant of steel beam section and that of the compression zone of concrete section;

f_c——design value of the compressive strength of concrete.

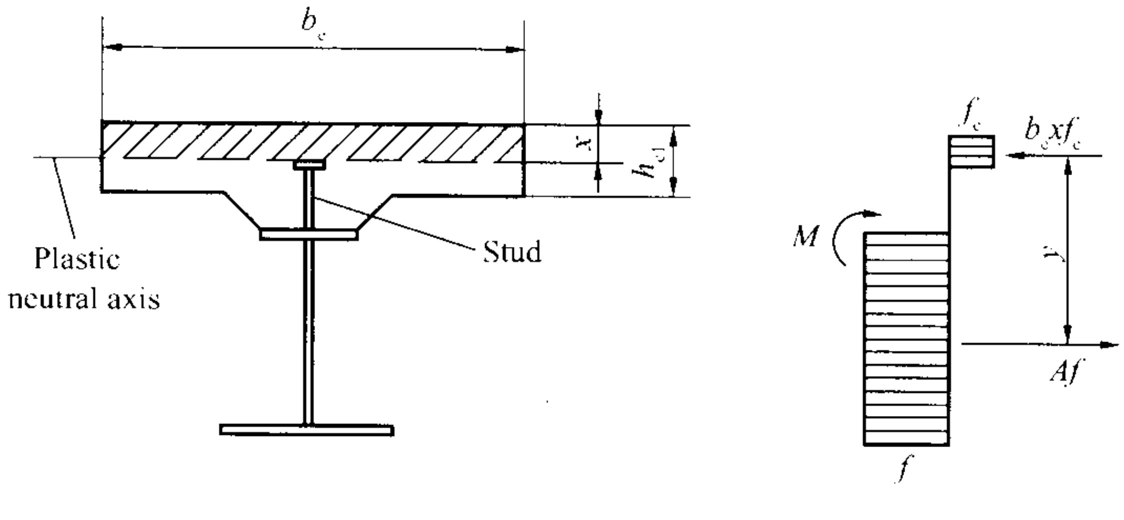


Fig. 11.2.1-1 Cross-section and stress diagram of composite beams with plastic neutral axis in concrete flange

2) When the plastic neutral axis is within the steel beam cross-section (Fig. 11.2.1-2), that is, when $Af > b_e h_{c1} f_c$:

$$M \leqslant b_e h_{c1} f_c y_1 + A_c f y_2 \tag{11.2.1-3}$$

$$A_c = 0.5(A - b_e h_{c1} f_c / f) \tag{11.2.1-4}$$

where A_c——area of the compression zone of steel beam section;

y_1——distance between the centroid of the tension zone of steel beam section and that of the compression zone of concrete flange section;

y_2——distance between the centroid of the tension zone of steel beam section and that of the compression zone of steel beam section.

2 The negative moment region (Fig. 11.2.1-3):

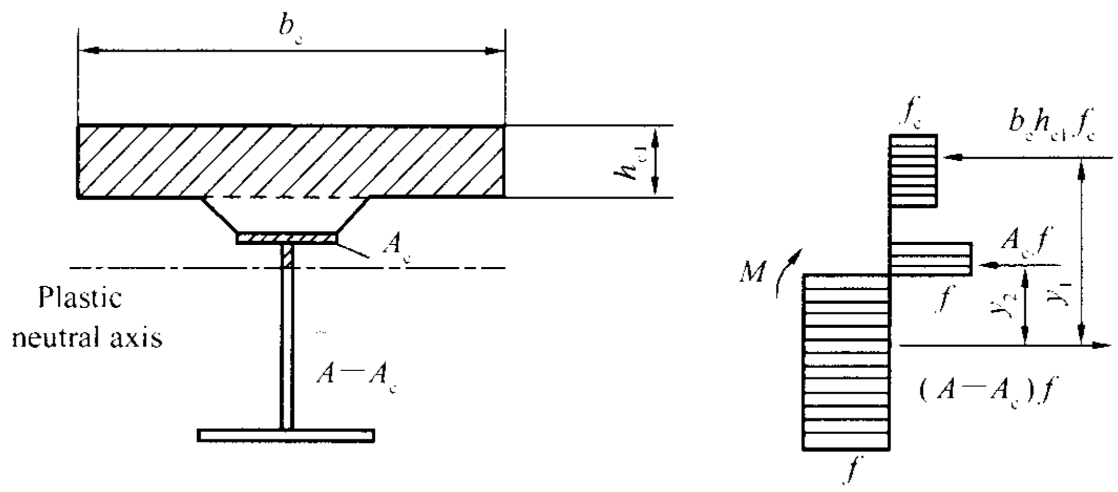


Fig. 11.2.1-2 Cross-section and stress diagram of composite beam with plastic neutral axis in steel beam

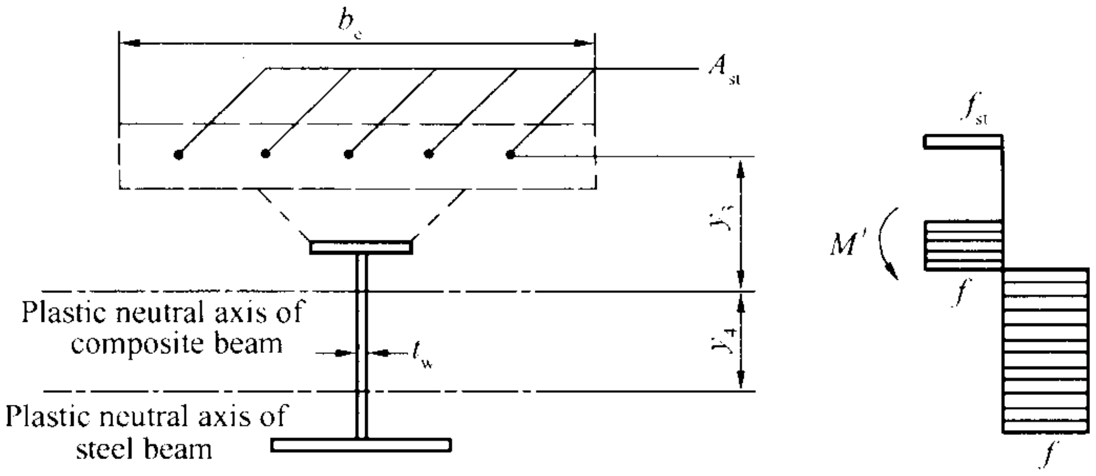


Fig. 11.2.1-3 Cross-section and stress diagram of composite beams under negative moment

$$M' \leqslant M_s + A_{st} f_{st}(y_3 + y_4/2) \tag{11.2.1-5}$$

$$M_s = (S_1 + S_2)f \tag{11.2.1-6}$$

where M'——design value of negative moment;

S_1, S_2——static moment about the plastic neutral axis of steel beam (the axis dividing the aera of steel beam section equally)of the section above and below the axis respectively;

A_{st}——sectional area of longitudinal reinforcement within the effective width of concrete flange in negative moment region;

f_{st}——design value of tensile strength of reinforcement;

y_3——distance between the centroid of the longitudinal reinforcment and the plastic neutral axis of composite beam;

y_4——distance between the plastic neutral axis of composite beam and that of steel beam. It shall be taken as $y_4 = A_{st} f_{st}/(2t_w f)$ when the plastic neutral axis of composite beam is within the web of steel beam; and y_4 may be taken as the distance between the plastic neutral axis of steel beam and the top edge of the web of steel beam when the plastic neutral axis is in the flange of steel beam.

11.2.2 The bending strength of composite beams with partial shear connection in positive moment region shall be calculated in accordance with the following formulae (Fig. 11.2.2):

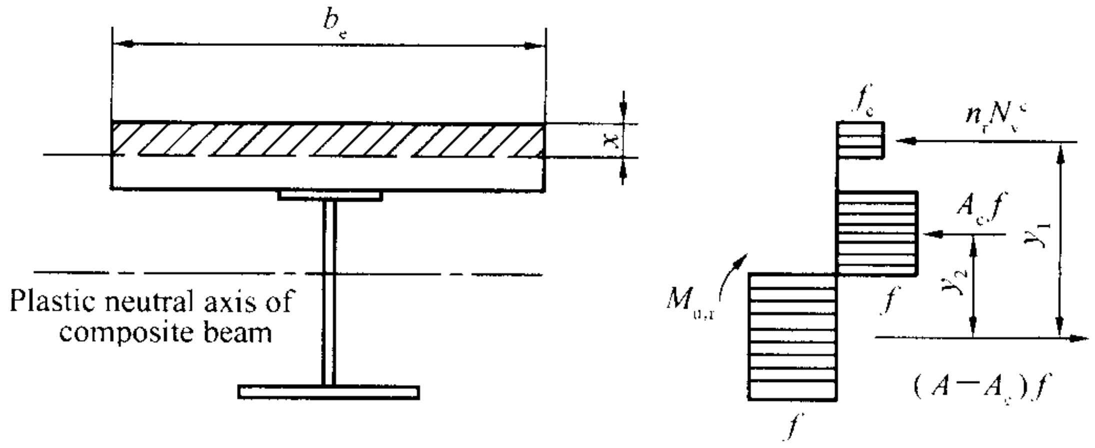


Fig. 11.2.2 Calculation diagram of composite beams with partial shear connection

$$x = n_r N_v^c / (b_e f_c) \tag{11.2.2-1}$$

$$A_c = (Af - n_r N_v^c)/(2f) \tag{11.2.2-2}$$

$$M_{u,r} = n_r N_v^c y_1 + 0.5(Af - n_r N_v^c) y_2 \tag{11.2.2-3}$$

where $M_{u,r}$——bending resistance of composite beams with partial shear connection;

n_r——number of shear connectors in one shear span of composite beams with partial shear connection;

N_v^c——longitudinal shear capacity of a shear connector, calculated in accordance with the relevant formulae in Section 11.3.

The bending strength of composite beams with partial shear connection in negative moment region shall be determined with the lesser of $n_r N_v^c$ and $A_{st} f_{st}$

11.2.3 The total shear force of the composite beam section is assumed to be resisted by the web plate of steel beam alone and shall be calculated in accordance with Formula (9.2.2).

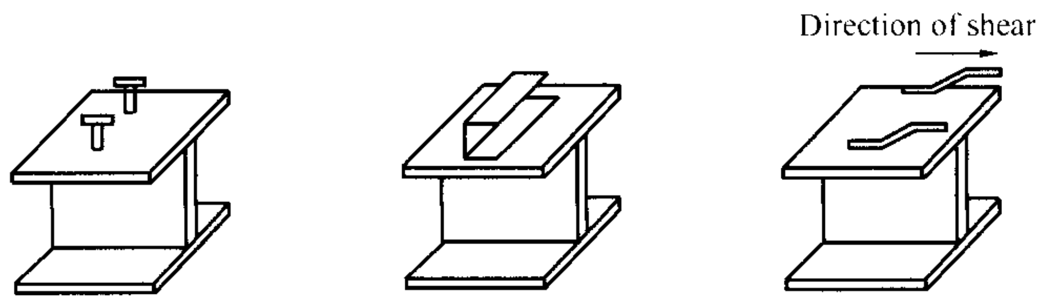
11.2.4 When the plastic design method is used to calculate the strength of composite beams, the interaction effect of moment and shear may be neglected in the following locations:

1 Cross-section of composite beams subject to positive moment;

2 Cross-section of composite beams subject to negative moment when $A_{st} f_{st} \geqslant 0.15Af$.

11.3 Calculation of shear connectors

11.3.1 It is suitable to adopt studs as shear connectors of composite beams; channels, bent-up bars and other types of reliable shear connectors may also be used. The arrangements of stud, channel and bent-up bar connectors are shown in Fig. 11.3.1; the design value of shear capacity of a shear connector may be determined by the following formulae:



(a) (b) (c)

Fig. 11.3.1 Appearances and arrangements of shear connectors

(a) Stud shear connector; (b) Channel connector; (c) Bent-up bar connector

1 Headed welded stud connector:

$$N_v^c = 0.43A_s\sqrt{E_c f_c} \leqslant 0.7A_s\gamma f \qquad (11.3.1\text{-}1)$$

where E_c——elastic modulus of concrete;

A_s——cross-sectional area of the shank of headed welded stud;

f——design value of tensile strength of headed welded stud;

γ——ratio of the minimum tensile strength and the yield strength of the material of stud. $f = 215\text{N/mm}^2$ and $\gamma = 1.67$ when the material property grade of stud is 4.6.

2 Channel connector:

$$N_v^c = 0.26(t + 0.5t_w)l_c\sqrt{E_c f_c} \qquad (11.3.1\text{-}2)$$

where t——mean thickness of the flange of channel;

t_w——thickness of the web of channel;

l_c——length of channel.

Channel connectors are connected to steel beam by two full-length fillet welds at the tip and the back of the flange of channel. The fillet welds shall be calculated to resist the design value of shear capacity of the connector N_v^c.

3 Bent-up bar connector:

$$N_v^c = A_{st}f_{st} \qquad (11.3.1\text{-}3)$$

where A_{st}——cross-sectional area of the bar;

f_{st}——design value of tensile strength of the bar.

11.3.2 For the composite beams with composite slab using profiled steel sheeting as flange (Fig. 11.3.2), the design value of shear capacity of stud connectors shall be reduced according to the following two situations:

1 When the ribs of profiled steel sheeting run parallel to the steel beam (Fig. 11.3.2a), and $b_w/h_e < 1.5$, N_v^c calculated in accordance with Formula (11.3.1-1)

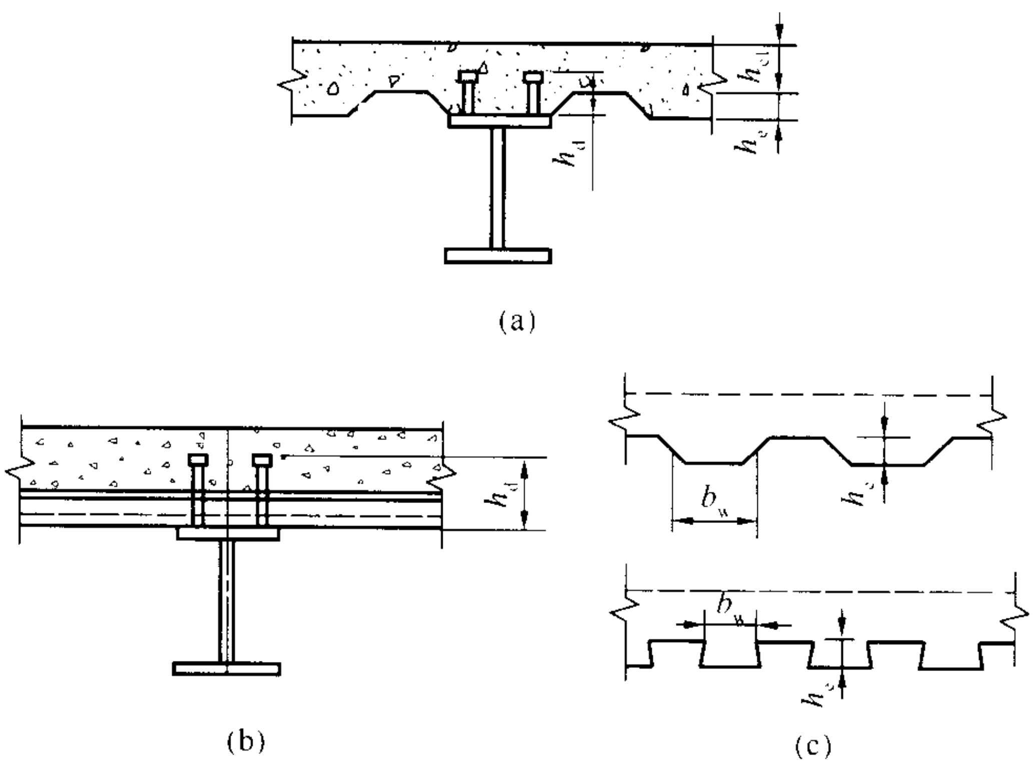


Fig. 11.3.2 Composite beams with composite slab using profiled steel sheeting as flange

(a)Cross-section of composite beams with composite slab using profiled steel sheeting(ribs parallel to steel beam); (b)Cross-section of composite beams with composite slab using profiled steel sheeting(ribs perpendicular to steel beam); (c)Cross-section of composite slab with profiled steel sheeting

shall be reduced by multiplying a reduction factor β_v, determined by the following formula:

$$\beta_v = 0.6 \frac{b_w}{h_e}\left(\frac{h_d - h_e}{h_e}\right) \leqslant 1 \qquad (11.3.2\text{-}1)$$

where b_w——mean width of concrete ribs, taken as the upper width of concrete ribs if the latter is less than the lower width (Fig. 11.3.2c).

h_e——depth of concrete rib;

h_d——height of stud.

2 When the ribs of profiled steel sheeting run perpendicular to steel beam (Fig. 11.3.2b), the reduction factor for design shear capacity of stud connector shall be determined by the following formula:

$$\beta_v = \frac{0.85}{\sqrt{n_0}} \frac{b_w}{h_e}\left(\frac{h_d - h_e}{h_e}\right) \leqslant 1 \qquad (11.3.2\text{-}2)$$

where n_0——number of studs in one rib of a certain cross-section of beam, taken as 3 when it is greater than 3.

11.3.3 For the shear connectors in negative moment region, the design value of shear capacity N_v^c shall be reduced by multiplying reduction factor 0.9(beside the mid support) and 0.8 (in the overhanging region).

11.3.4 The calculation of shear connectors shall be carried out successively for each shear span

(Fig. 11.3.4) which is a beam segment between the section of maximum bending moment, positive or negative, and the adjacent section of zero moment. The longitudinal shear V_s at the interface of steel beam and concrete flange in each shear span shall be determined as follows:

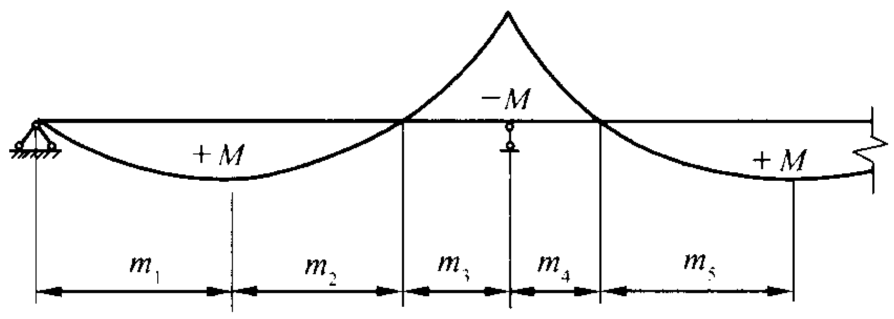


Fig. 11.3.4 Division diagram of shear spans of continuous beam

1 V_s shall be taken as the lesser of Af and $b_e h_{c1} f_c$ for the shear span in the positive moment region.

2 For the shear span in the negative moment region:

$$V_s = A_{st} f_{st} \tag{11.3.4-1}$$

If the composite beams are designed as beams with full shear connection, the total number of shear connectors required in each shear span, n_f, shall be determined by the following formula:

$$n_f = V_s / N_v^c \tag{11.3.4-2}$$

For composite beams with partial shear connection, the actual number of shear connectors used shall not be less than 50% of n_f.

The number of shear connectors determined by Formula (11.3.4-2) may be arranged uniformly in the corresponding shear span. If there is any comparatively large concentrated load in the shear span, the number of shear connectors n_f shall be divided in parts according to the ratio of areas of shear diagram and then arranged uniformly in each part.

Note: If the stud and channel shear connectors are used, the shear spans m_2 and m_3, m_4 and m_5 in Fig. 11.3.4 may be combined respectively for the arrangement of shear connectors; after the combination, the longitudinal shear in combined region shall be taken as $V_s = b_e h_{c1} f_c + A_{st} f_{st}$. It is proposed to use full shear connection in the combined regions.

11.4 Calculation of deflection

11.4.1 The deflection of composite beams shall be calculated for the characteristic combination of load effects and the quasi-permanent combination of load effects respectively, and take the greater one as the basis. The deflection may be determined from the relevant formulae in Structural Mechanics. The bending stiffness shall be taken as the reduced stiffness considering the effect of slip for the composite beams subject to positive moment only, and shall be treated as variable cross-section stiffness (see Clause 11.1.3) for continuous composite beams. For the two combinations of load effects mentioned above, the stiffness of composite beams shall be taken as the reduced stiffness corresponding to each combination.

11.4.2 The reduced stiffness of composite beams considering the effect of slip, B, may be determined by the following formula:

$$B = \frac{EI_{eq}}{1 + \zeta} \tag{11.4.2}$$

where E——elastic modulus of steel beam;

I_{eq}——moment of inertia of transformed cross-section of composite beam. The moment of inertia of the whole transformed section may be calculated after transforming the concrete area to equivalent steel one by dividing the effective width of concrete flange by the elastic modular ratio of steel to concrete, α_E, for the characteristic combination of load effects, and dividing it by $2\alpha_E$ for the quasi-permanent combination of load effects. For the composite beams composed by steel beam and composite slab with profiled steel sheeting, the calculation shall be carried out at the weaker cross-section of the beam, and the effect of profiled steel sheeting should be neglected.

ζ——reduction coefficient of stiffness, calculated in accordance with Clause 11.4.3.

11.4.3 The reduction coefficient of stiffness ζ is calculated as follows (when $\zeta \leqslant 0$, take $\zeta = 0$):

$$\zeta = \eta\left[0.4 - \frac{3}{(jl)^2}\right] \tag{11.4.3-1}$$

$$\eta = \frac{36Ed_c pA_0}{n_s khl^2} \tag{11.4.3-2}$$

$$j = 0.81\sqrt{\frac{n_s kA_1}{EI_0 p}} \quad (\mathrm{mm}^{-1}) \tag{11.4.3-3}$$

$$A_0 = \frac{A_{cf}A}{\alpha_E A + A_{cf}} \tag{11.4.3-4}$$

$$A_1 = \frac{I_0 + A_0 d_c^2}{A_0} \tag{11.4.3-5}$$

$$I_0 = I + \frac{I_{cf}}{\alpha_E} \tag{11.4.3-6}$$

where A_{cf}——area of concrete flange section; taken as the area of the weaker cross-section neglecting the profiled steel sheeting for the flanges of composite slab with profiled steel sheeting;

A——area of steel beam section;

I——moment of inertia of steel beam cross-section;

I_{cf}——moment of inertia of concrete flange cross-section; taken as the moment of inertia of the weaker cross-section neglecting the profiled steel sheeting for the flanges of composite slab with profiled steel sheeting;

d_c——distance between the centroid of the section of steel beam and that of concrete flange (taken as the weaker cross-section for composite slab with profiled steel sheeting);

h——depth of cross-section of composite beam;

l——span of composite beam(mm);

k——stiffness factor of shear connector, $k = N_v^c$ (N/mm);

p——average longitudinal spacing between shear connectors(mm);

n_s——number of rows of shear connectors on one beam;

α_E——elastic modular ratio of steel to concrete.

Note: When calculating for the quasi-permanent combination of load effects, α_E, in Formulae(11.4.3-4) and (11.4.3-6) shall be multiplied by 2.

11.5 Detailing requirements

11.5.1 The depth of cross-section of composite beams should not be greater than 2.5 times the depth of cross-section of steel beams; the depth of concrete haunch h_{c2} should not be greater than 1.5 times the thickness of concrete flange h_{c1}; the width of the top of concrete haunch should not be less than the sum of the width of top flange of steel beam and $1.5h_{c2}$.

11.5.2 The detailing of concrete flange of boundary composite beams shall satisfy the requirements of Fig.11.5.2. For beams with concrete haunch, the overhanging length should not be less than h_{c2}; for beams without concrete haunch, the overhanging length from the center line of steel beam shall not be less than 150mm, and that from the flange edge of steel beam shall not be less than 50mm.

11.5.3 For the continuous composite beams, the longitudinal bars and distribution bars in the negative moment region at the internal support shall be placed in accordance with the current national standards " Code for design of concrete structures"GB 50010.

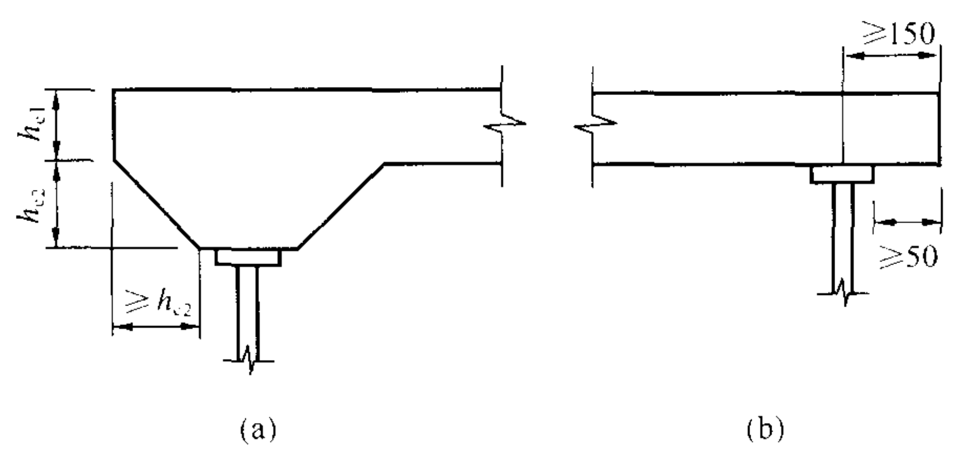


Fig. 11.5.2 Detailing of boundary beams

11.5.4 The arrangement of shear connectors shall be in accordance with following provisions:

1 The distance from the bottom of the head of stud connectors or the bottom of the top flange of channel connectors to the top of bottom steel reinforcing bars in concrete flange should not be less than 30mm;

2 The maximum spacing between the connectors in the direction of beam span shall not be greater than 4 times the thickness of concrete flange (including concrete haunch), and shall not be greater than 40mm;

3 The distance between the exterior margin of connector and the edge of flange of steel beam shall not be less than 20mm;

4 The distance between the exterior margin of connector and the edge of concrete flange shall not be less than 100mm;

5 The thickness of concrete cover above the top of the connectors shall not be less than 15mm.

11.5.5 Besides the requirements in Clause 11.5.4, the stud connectors shall also conform to the following provisions;

1 When the stud is not aligned with the web plate of steel beam, if the top flange of steel beam is in tension, the diameter of the stud shall not be greater than 1.5 times the thickness of the top flange of steel beam; and if the top flange of steel beam is not in tension, the diameter of the stud shall not be greater than 2.5 times the thickness of the top flange of steel beam;

2 The length of the stud shall not be less than 4 times the diameter of the stud;

3 The spacing of studs along the direction of beam span shall not be less than 6 times the diameter of the stud; and the spacing of studs perpendicular to the direction of beam span shall not be less than 4 times the diameter of the stud;

4 For the composite beams using profiled steel sheeting as formwork, the diameter of stud should not be greater than 19mm, the width of concrete protuberant ribs shall not be less than 2.5 times the diameter of stud; the length of stud shall satisfy $(h_c + 30) \leqslant h_d \leqslant (h_c + 75)$ (Fig. 11.3.2).

11.5.6 Besides the requirements in Clause 11.5.4, the bent-up bar connectors shall yet satisfy the following provisions: the bar connectors should be made from reinforcing bars of diameter not less than 12mm, and be arranged in pairs; the connectors should be welded onto the flange of steel beam by two side welds, the length of which should not be less than 4 times (grade Ⅰ reinforcing steel bar) or 5 times (grade Ⅱ reinforcing steel bar) the diameter of steel bar; the angle of bending is 45° in general, and the direction of bending shall be consistent with that of the horizontal shear between the concrete flange and the steel beam. In the mid-span where the direction of longitudinal horizontal shear changes, bars must be arranged in both directions. The length of the bar from the point of bending to its end should not be less than 25 times its diameter (a hook shall be added at the end for grade Ⅰ reinforcing steel bar), in which the length of horizontal segment should not be less than 10 times its diameter. The spacing of bar connectors in the direction of beam span should not be less than 0.7 times the thickness of concrete flange (including the thickness of concrete haunch).

11.5.7 Generally, Q235 steel is used for channel connectors, and its cross-section should not be larger than [12.6.

11.5.8 The top surface of the steel beam shall not be painted. Before casting (or erecting) the concrete flange, rust, welding slag, ice piece, snow, earth and other miscellaneous things shall be cleaned out.

Appendix A Allowable deflection of structures or structural members

A.1 Allowable deflection of flexural members

A.1.1 The deflection of crane girders, floor beams, roof girders, working platform beams and members of wall framing should not exceed the relevant allowable value listed in Table A.1.1.

Table A.1.1 Allowable deflection of flexural members

Item No.	Type of member	Allowable deflection $[v_T]$	Allowable deflection $[v_Q]$
1	Crane girders and crane trusses (calculated deflection under self-weight and one of the cranes of largest capacity) (1) For hand operated cranes and monorails(including underslung cranes) (2) For bridge cranes of light duty (3) For bridge cranes of medium duty (4) For bridge cranes of heavy duty	 $l/500$ $l/800$ $l/1000$ $l/1200$	—
2	Beam-rails for hand operated or electric hoists	$l/400$	—
3	Beams of working platform under track with heavy rail (weighing 38kg/m or more) Beams of working platform under track with light rail(weighing 24kg/m or less)	$l/600$ $l/400$	—
4	Floor(roof) beams or trusses, platform beams(except Item No.3) and platform slabs (1) Main girders or trusses (including those with underslung hoisting equipment) (2) Beams with plastered ceiling (3) Beams other than Item No.1 and 2(including stair beams) (4)Purlins under roofing of corrugated iron and asbestos sheet with no dust accumulation profiled metal sheet, corrugated iron and asbestos sheet etc. with dust accumulation other material (5) Platform slabs	 $l/400$ $l/250$ $l/250$ $l/150$ $l/200$ $l/200$ $l/150$	 $l/500$ $l/350$ $l/300$ — — — —
5	Members of wall framing (taking no account of gust coefficient for wind load) (1) Stud (2) Wind truss (acting as support to continuous stud) (3) Girt (horizontal) in masonry wall (4)Girt(horizontal) for cladding of profiled metal sheet, corrugated iron and asbestos sheet (5) Girt (vertical and horizontal) for glass window	 — — — — $l/200$	 $l/400$ $l/1000$ $l/300$ $l/200$ $l/200$

Note: 1 l denotes the span length of a flexural member (for cantilever beam and overhanging beam, is twofold the overhang).

2 $[v_T]$ is the allowable deflection under the characteristic value of permanent and variable load (the camber shall be deducted if there exists any);

$[v_Q]$ is the allowable deflection under the characteristic value of variable load.

A.1.2 In metallurgical works and similar workshop equipped with cranes of category A7, A8, the deflection of the surge girder on each side of the bay, caused by transverse horizontal load (conforming to the Load Code) from one of the cranes of largest capacity, should not exceed 1/2200 of its span length.

A.2 Allowable horizontal displacement of framed structures

A.2.1 The horizontal displacement of the column top and the story drift in a frame should not exceed the following values under the action of unfactored wind load.

1 Column top displacement of single story frame without bridge crane $H/150$

2 Column top displacement of single story frame equipped with bridge crane $H/400$

3 Column top displacement of multistory frame $H/500$

4 Story drift of multistory frame $h/400$

H is the total height from top of foundation to column top; h is the story height.

Note: 1 For multistory framed structure of a civil building requiring refined indoor decoration, the story drift should be reduced appropriately. For multistory frames without wall, the story drift may be appropriately enlarged.

2 For light frame structures, both the column top displacement and the story drift may be enlarged appropriately.

A.2.2 In metallurgical works and similar workshop equipped with cranes of category A7, A8 and for open-air gantry columns equipped with medium and heavy duty cranes, the calculated deformation of the column, caused by the horizontal load from one of the cranes of largest capacity, at the top elevation of the crane girder or crane truss, should not exceed the relevant allowable value given in Table A.2.2 (the magnitude of load conforming with the Load Code).

Table A.2.2 Allowable value of column horizontal displacement (calculated value)

Item No.	Type of displacement	Calculation based on	
		plane structure scheme	space structure scheme
1	Transverse displacement of mill building columns	$H_c/1250$	$H_c/2000$
2	Transverse displacement of open-air gantry columns	$H_c/2500$	—
3	Longitudinal displacement of columns of mill building and open-air gantry	$H_c/4000$	—

Note: 1 H_c denotes the height from the foundation top to the top of the crane girder or truss.

2 In the calculation of longitudinal displacement of mill building or open-air gantry column, the longitudinal load of the crane may be distributed to all the inter-column bracings or longitudinal frame, in a section between expansion points.

3 For mill buildings equipped with cranes of category A8, the allowable calculated displacement of the columns should be reduced by 10%.

4 For mill buildings equipped with cranes of category A6, the longitudinal displacement of the columns should comply with the requirement of this table.

Appendix B Overall stability factor of beams

B.1 Simply supported beam of uniform welded I- and rolled H-section

The overall stability factor, φ_b, of simply supported beams of uniform welded I-and rolled H-section (Fig.B.1) shall be determined by the following formula:

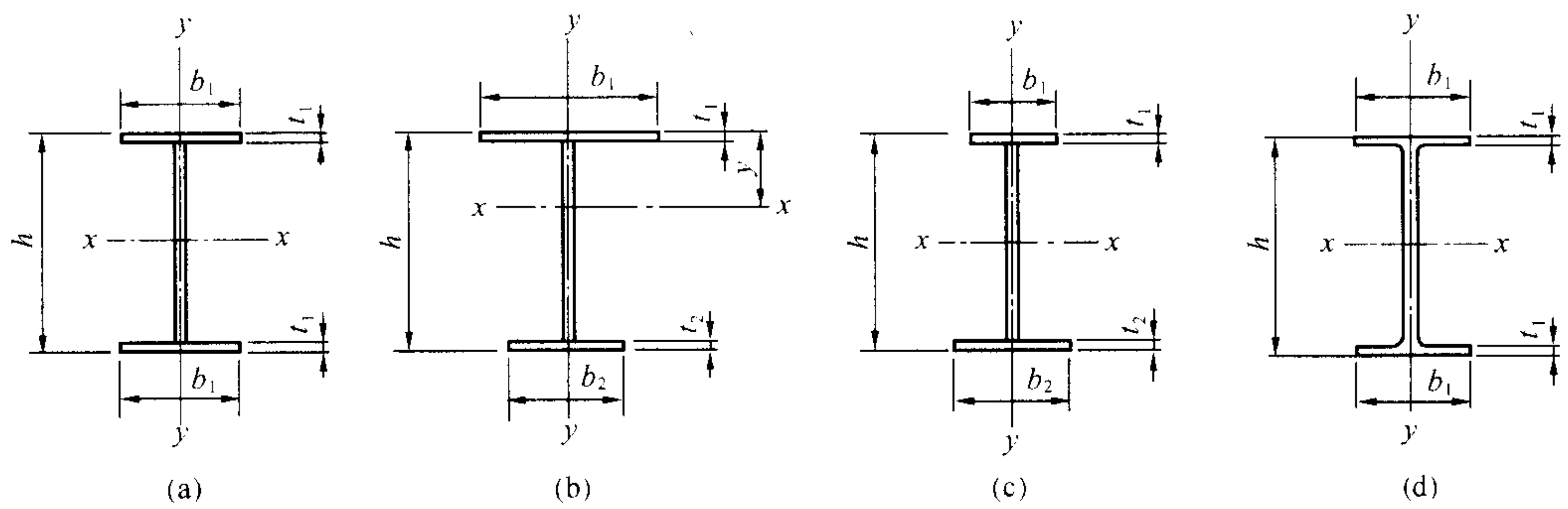


Fig.B.1 Welded I- and rolled H-section
(a)Doubly symmetric welded I-section; (b)Monosymmetric welded I-section with enlarged compression flange; (c)Monosymmetric welded I-section with enlarged tension flange; (d)Rolled H-section

$$\varphi_b = \beta_b \frac{4320}{\lambda_y^2} \cdot \frac{Ah}{W_x} \left[\sqrt{1 + \left(\frac{\lambda_y t_1}{4.4h} \right)^2} + \eta_b \right] \frac{235}{f_y} \tag{B.1-1}$$

where β_b——factor of equivalent critical moment for overall stability of beams, given in Table B.1;

λ_y——slenderness ratio about the weak axis y-y of the portion of the beam between laterally supported points, $\lambda_y = l_1/i_y$, l_1 taken as in Clause 4.2.1, i_y being the radius of gyration of the gross section about the y-axis;

A——gross sectional area of the beam;

h, t_1——beam depth and thickness of the compression flange, respectively;

η_b——factor of unsymmetry of a beam section:

For doubly symmetric section (see Fig.B.1a,d) $\eta_b = 0$

For monosymmetric I-section (see Fig.B.1b,c)

With enlarged compression flange $\eta_b = 0.8(2\alpha_b - 1)$

With enlarged tension flange $\eta_b = 2\alpha_b - 1$

$\alpha_b = \frac{I_1}{I_1 + I_2}$——$I_1$ and I_2 being the moment of inertia about y-axis of the compression and tension flanges respectively.

When the value of φ_b calculated according to Formula (B.1-1) is larger than 0.60, they shall be replaced by the corresponding values of φ'_b given by the following formula:

$$\varphi'_b = 1.07 - 0.282/\varphi_b \leqslant 1.0 \tag{B.1-2}$$

Note: The Formula (B.1-1) also applies to simply supported riveted (or high-strength-bolted) beams of uniform section. The thickness of compression flange t_1 of these beams includes that of the flange angle.

Table B.1 Factor β_b for simply supported beams of uniform I- and H-section

Item No.	Lateral support	Loading		$\xi \leqslant 2.0$	$\xi > 2.0$	Applicability
1	No intermediate lateral support	Uniform load acting at	upper flange	$0.69+0.13\xi$	0.95	Only for sections a, b and d in Fig.B.1
2			lower flange	$1.73-0.20\xi$	1.33	
3		Concentrated load acting at	upper flange	$0.73+0.18\xi$	1.09	
4			lower flange	$2.23-0.28\xi$	1.67	
5	One lateral support at mid-span	Uniform load acting at	upper flange	1.15		For all sections of Fig. B. 1
6			lower flange	1.40		
7		Concentrated load acting at any level		1.75		
8	Not less than two intermediate equally spaced lateral supports	Any kind of load acting at	upper flange	1.20		
9			lower flange	1.40		
10	Subjected to end moments, without any transverse load within the span			$1.75-1.05\left(\frac{M_2}{M_1}\right)+0.3\left(\frac{M_2}{M_1}\right)^2$, but not larger than 2.3		

Note: 1 $\xi = l_1 t_1 / b_1 h$—parameter, see Clause 4.2.1 for the values of b_1 and l_1.

2 M_1, M_2 are end moments of the beam, having same sign when causing curvatures of same sign and vice versa, $|M_1| \geqslant |M_2|$.

3 Concentrated loading in Item Nos.3,4 and 7 denotes one single load or a few concentrated loads near the mid-span. For other cases of concentrated loading β_b shall be taken from Item Nos.1,2,5 and 6.

4 In Item Nos.8 and 9, β_b shall be taken equal to 1.20 when concentrated loads act on the laterally supported points.

5 Loading acting at the upper flange means that acting on the flange surface and pointing toward the centroid of cross section in direction; loading acting at the lower flange means that acting on the flange surface but pointing away from the centroid in direction.

6 For I-section with enlarged compression flange so as $\alpha_b > 0.8$, the value of β_b shall be multiplied by a coefficient as follows:

0.95 for Item No.1, when $\xi \leqslant 1.0$;

0.90 for Item No.3, when $\xi \leqslant 0.5$;

0.95 for Item No.3, when $0.5 < \xi \leqslant 1.0$.

B.2 Simply supported beam of regular rolled I-section

The overall stability factor, φ_b, of simply supported beams of regular rolled I-section shall be taken from Table B.2. Values of φ_b larger than 0.6 shall be replaced by the corresponding values of φ'_b computed by Formula (B.1-2).

Table B.2 Factor φ_b of simply supported beams of regular rolled I-section

Item No.	Loading pattern			I-section designation	Unsupported length l_1(m)								
					2	3	4	5	6	7	8	9	10
1	No intermediate lateral support	Concentrated load acting at	Upper flange	10~20	2.00	1.30	0.99	0.80	0.68	0.58	0.53	0.48	0.43
				22~32	2.40	1.48	1.09	0.86	0.72	0.62	0.54	0.49	0.45
				36~63	2.80	1.60	1.07	0.83	0.68	0.56	0.50	0.45	0.40
2			Lower flange	10~20	3.10	1.95	1.34	1.01	0.82	0.69	0.63	0.57	0.52
				22~40	5.50	2.80	1.84	1.37	1.07	0.86	0.73	0.64	0.56
				45~63	7.30	3.60	2.30	1.62	1.20	0.96	0.80	0.69	0.60
3		Uniform load acting at	Upper flange	10~20	1.70	1.12	0.84	0.68	0.57	0.50	0.45	0.41	0.37
				22~40	2.10	1.30	0.93	0.73	0.60	0.51	0.45	0.40	0.36
				45~63	2.60	1.45	0.97	0.73	0.59	0.50	0.44	0.38	0.35
4			Lower flange	10~20	2.50	1.55	1.08	0.83	0.68	0.56	0.52	0.47	0.42
				22~40	4.00	2.20	1.45	1.10	0.85	0.70	0.60	0.52	0.46
				45~63	5.60	2.80	1.80	1.25	0.95	0.78	0.65	0.55	0.49
5	With intermediate lateral support (without regard to the position of load application point along the beam depth)			10~20	2.20	1.39	1.01	0.79	0.66	0.57	0.52	0.47	0.42
				22~40	3.00	1.80	1.24	0.96	0.76	0.65	0.56	0.49	0.43
				45~63	4.00	2.20	1.38	1.01	0.80	0.66	0.56	0.49	0.43

Note: 1 Same as Notes 3 and 5 of Table B.1.

2 Values of φ_b in this table applies to Q235 steel and shall be multiplied by $235/f_y$ for other steel grades.

B.3 Simply supported beams of rolled channel-section

The overall stability factor φ_b of simply supported beams of rolled channel-section shall be calculated by the following expression without regard to the pattern of loading and the position of load application point along the beam depth:

$$\varphi_b = \frac{570bt}{l_1 h} \cdot \frac{235}{f_y} \tag{B.3}$$

where h, b, t——depth, flange width and average flange thickness of the channel section respectively.

Values of φ_b larger than 0.6 obtained from Formula (B.3) shall be replaced by the corresponding values of φ'_b computed by Formula (B.1-2).

B.4 Cantilever beams of doubly symmetric uniform I-section (including rolled H shape)

The overall stability factor, φ_b, for cantilever beams of doubly symmetric I-section (including rolled H shape) may be calculated by Formula (B.1-1) but the factor β_b shall be taken from Table B.4 and l_1 of $\lambda_y = l_1/i_y$ shall be taken as the cantilever length. The calculated values of φ_b larger than 0.6 shall be replaced by the corresponding values of φ'_b computed by Formula (B.1-2).

Table B.4 Factor β_b for cantilever beams of doubly symmetric I-section (including rolled H shape)

Item No.	Loading pattern		$0.60 \leqslant \xi \leqslant 1.24$	$1.24 < \xi \leqslant 1.96$	$1.96 < \xi \leqslant 3.10$
1	One concentrated load at the free end acting on	Upper flange	$0.21+0.67\xi$	$0.72+0.26\xi$	$1.17+0.03\xi$
2		Lower flange	$2.94-0.65\xi$	$2.64-0.40\xi$	$2.15-0.15\xi$
3	Uniform load acting on the upper flange		$0.62+0.82\xi$	$1.25+0.31\xi$	$1.66+0.10\xi$

Note: 1 This table is based on the condition that the supported end is rigidly restrained. Measures shall be taken to increase the torsional capacity at the support when using values of this table for an overhanging beam extending from adjacent span.

2 For values of ξ, see Note 1 of Table B.1.

B.5 Approximate calculation of overall stability factors

The overall stability factor φ_b of uniformly bent beams, when $\lambda_y \leqslant 120\sqrt{235/f_y}$, may be calculated by the following approximate formulae:

1 I-section (including rolled H shape)

Doubly symmetric:

$$\varphi_b = 1.07 - \frac{\lambda_y^2}{44000} \cdot \frac{f_y}{235} \tag{B.5-1}$$

Monosymmetric:

$$\varphi_b = 1.07 - \frac{W_x}{(2\alpha_b + 0.1)Ah} \cdot \frac{\lambda_y^2}{14000} \cdot \frac{f_y}{235} \tag{B.5-2}$$

2 T-section (moment acting in the plane of symmetric axis, about x-axis):

1) When the flange is in compression under the moment:

T-section composed of twin angles:

$$\varphi_b = 1 - 0.0017\lambda_y\sqrt{f_y/235} \tag{B.5-3}$$

Rolled cut T-section and T-section built-up of two plates:

$$\varphi_b = 1 - 0.0022\lambda_y\sqrt{f_y/235} \tag{B.5-4}$$

2) When the flange is in tension under the moment and web width-thickness ratio not larger than $18\sqrt{235/f_y}$:

$$\varphi_b = 1.0 - 0.0005\lambda_y\sqrt{f_y/235} \tag{B.5-5}$$

Values of φ_b given by Formulae (B.5-1) through (B.5-5) need not be replaced by φ'_b of Formula (B.1-2) when larger than 0.60. Values of φ_b given by Formulae (B.5-1) and (B.5-2) shall be taken as 1.0 when larger than 1.0.

Appendix C Stability factor of axial compression members

Table C-1 Stability factor φ, of axial compression members, for Class a sections

$\lambda\sqrt{\frac{f_y}{235}}$	0	1	2	3	4	5	6	7	8	9
0	1.000	1.000	1.000	1.000	0.999	0.999	0.998	0.998	0.997	0.996
10	0.995	0.994	0.993	0.992	0.991	0.989	0.988	0.986	0.985	0.983
20	0.981	0.979	0.977	0.976	0.974	0.972	0.970	0.968	0.966	0.964
30	0.963	0.961	0.959	0.957	0.955	0.952	0.950	0.948	0.946	0.944
40	0.941	0.939	0.937	0.934	0.932	0.929	0.927	0.924	0.921	0.919
50	0.916	0.913	0.910	0.907	0.904	0.900	0.897	0.894	0.890	0.886
60	0.883	0.879	0.875	0.871	0.867	0.863	0.858	0.854	0.849	0.844
70	0.839	0.834	0.829	0.824	0.818	0.813	0.807	0.801	0.795	0.789
80	0.783	0.776	0.770	0.763	0.757	0.750	0.743	0.736	0.728	0.721
90	0.714	0.706	0.699	0.691	0.684	0.676	0.668	0.661	0.653	0.645
100	0.638	0.630	0.622	0.615	0.607	0.600	0.592	0.585	0.577	0.570
110	0.563	0.555	0.548	0.541	0.534	0.527	0.520	0.514	0.507	0.500
120	0.494	0.488	0.481	0.475	0.469	0.463	0.457	0.451	0.445	0.440
130	0.434	0.429	0.423	0.418	0.412	0.407	0.402	0.397	0.392	0.387
140	0.383	0.378	0.373	0.369	0.364	0.360	0.356	0.351	0.347	0.343
150	0.339	0.335	0.331	0.327	0.323	0.320	0.316	0.312	0.309	0.305
160	0.302	0.298	0.295	0.292	0.289	0.285	0.282	0.279	0.276	0.273
170	0.270	0.267	0.264	0.262	0.259	0.256	0.253	0.251	0.248	0.246
180	0.243	0.241	0.238	0.236	0.233	0.231	0.229	0.226	0.224	0.222
190	0.220	0.218	0.215	0.213	0.211	0.209	0.207	0.205	0.203	0.201
200	0.199	0.198	0.196	0.194	0.192	0.190	0.189	0.187	0.185	0.183
210	0.182	0.180	0.179	0.177	0.175	0.174	0.172	0.171	0.169	0.168
220	0.166	0.165	0.164	0.162	0.161	0.159	0.158	0.157	0.155	0.154
230	0.153	0.152	0.150	0.149	0.148	0.147	0.146	0.144	0.143	0.142
240	0.141	0.140	0.139	0.138	0.136	0.135	0.134	0.133	0.132	0.131
250	0.130									

Note: See Note of Table C-4.

Table C-2 Stability factor φ, of axial compression members, for Class b sections

$\lambda\sqrt{\frac{f_y}{235}}$	0	1	2	3	4	5	6	7	8	9
0	1.000	1.000	1.000	0.999	0.999	0.998	0.997	0.996	0.995	0.994
10	0.992	0.991	0.989	0.987	0.985	0.983	0.981	0.978	0.976	0.973
20	0.970	0.967	0.963	0.960	0.957	0.953	0.950	0.946	0.943	0.939
30	0.936	0.932	0.929	0.925	0.922	0.918	0.914	0.910	0.906	0.903
40	0.899	0.895	0.891	0.887	0.882	0.878	0.874	0.870	0.865	0.861
50	0.856	0.852	0.847	0.842	0.838	0.833	0.828	0.823	0.818	0.813
60	0.807	0.802	0.797	0.791	0.786	0.780	0.774	0.769	0.763	0.757
70	0.751	0.745	0.739	0.732	0.726	0.720	0.714	0.707	0.701	0.694
80	0.688	0.681	0.675	0.668	0.661	0.655	0.648	0.641	0.635	0.628
90	0.621	0.614	0.608	0.601	0.594	0.588	0.581	0.575	0.568	0.561
100	0.555	0.549	0.542	0.536	0.529	0.523	0.517	0.511	0.605	0.499
110	0.493	0.487	0.481	0.475	0.470	0.464	0.458	0.453	0.447	0.442
120	0.437	0.432	0.426	0.421	0.416	0.411	0.406	0.402	0.397	0.392
130	0.387	0.383	0.378	0.374	0.370	0.365	0.361	0.357	0.353	0.349
140	0.345	0.341	0.337	0.333	0.329	0.326	0.322	0.318	0.315	0.311
150	0.308	0.304	0.301	0.298	0.295	0.291	0.288	0.285	0.282	0.279
160	0.276	0.273	0.270	0.267	0.265	0.262	0.259	0.256	0.254	0.251
170	0.249	0.246	0.244	0.241	0.239	0.236	0.234	0.232	0.229	0.227
180	0.225	0.223	0.220	0.218	0.216	0.214	0.212	0.210	0.208	0.206
190	0.204	0.202	0.200	0.198	0.197	0.195	0.193	0.191	0.190	0.188
200	0.186	0.184	0.183	0.181	0.180	0.178	0.176	0.175	0.173	0.172
210	0.170	0.169	0.167	0.166	0.165	0.163	0.162	0.160	0.159	0.158
220	0.156	0.155	0.154	0.153	0.151	0.150	0.149	0.148	0.146	0.145
230	0.144	0.143	0.142	0.141	0.140	0.138	0.137	0.136	0.135	0.134
240	0.133	0.132	0.131	0.130	0.129	0.128	0.127	0.126	0.125	0.124
250	0.123									

Note: See Note of Table C-4.

Table C-3　Stability factor φ, of axial compression members, for Class c sections

$\lambda\sqrt{\frac{f_y}{235}}$	0	1	2	3	4	5	6	7	8	9
0	1.000	1.000	1.000	0.999	0.999	0.998	0.997	0.996	0.995	0.993
10	0.992	0.990	0.988	0.986	0.983	0.981	0.978	0.976	0.973	0.970
20	0.966	0.959	0.953	0.947	0.940	0.934	0.928	0.921	0.915	0.909
30	0.902	0.896	0.890	0.884	0.877	0.871	0.865	0.858	0.852	0.846
40	0.839	0.833	0.826	0.820	0.814	0.807	0.801	0.794	0.788	0.781
50	0.775	0.768	0.762	0.755	0.748	0.742	0.735	0.729	0.722	0.715
60	0.709	0.702	0.695	0.689	0.682	0.676	0.669	0.662	0.656	0.649
70	0.643	0.636	0.629	0.623	0.616	0.610	0.604	0.597	0.591	0.584
80	0.578	0.572	0.566	0.559	0.553	0.547	0.541	0.535	0.529	0.523
90	0.517	0.511	0.505	0.500	0.494	0.488	0.483	0.477	0.472	0.467
100	0.463	0.458	0.454	0.449	0.445	0.441	0.436	0.432	0.428	0.423
110	0.419	0.415	0.411	0.407	0.403	0.399	0.395	0.391	0.387	0.383
120	0.379	0.375	0.371	0.367	0.364	0.360	0.356	0.353	0.349	0.346
130	0.342	0.339	0.335	0.332	0.328	0.325	0.322	0.319	0.315	0.312
140	0.309	0.306	0.303	0.300	0.297	0.294	0.291	0.288	0.285	0.282
150	0.280	0.277	0.274	0.271	0.269	0.266	0.264	0.261	0.258	0.256
160	0.254	0.251	0.249	0.246	0.244	0.242	0.239	0.237	0.235	0.233
170	0.230	0.228	0.226	0.224	0.222	0.220	0.218	0.216	0.214	0.212
180	0.210	0.208	0.206	0.205	0.203	0.201	0.199	0.197	0.196	0.194
190	0.192	0.190	0.189	0.187	0.186	0.184	0.182	0.181	0.179	0.178
200	0.176	0.175	0.173	0.172	0.170	0.169	0.168	0.166	0.165	0.163
210	0.162	0.161	0.159	0.158	0.157	0.156	0.154	0.153	0.152	0.151
220	0.150	0.148	0.147	0.146	0.145	0.144	0.143	0.142	0.140	0.139
230	0.138	0.137	0.136	0.135	0.134	0.133	0.132	0.131	0.130	0.129
240	0.128	0.127	0.126	0.125	0.124	0.124	0.123	0.122	0.121	0.120
250	0.119									

Note: See Note of Table C-4.

Table C-4 Stability factor φ, of axial compression members, for Class d sections

$\lambda\sqrt{\frac{f_y}{235}}$	0	1	2	3	4	5	6	7	8	9
0	1.000	1.000	0.999	0.999	0.998	0.996	0.994	0.992	0.990	0.987
10	0.984	0.981	0.978	0.974	0.969	0.965	0.960	0.955	0.949	0.944
20	0.937	0.927	0.918	0.909	0.900	0.891	0.883	0.874	0.865	0.857
30	0.878	0.840	0.831	0.823	0.815	0.807	0.799	0.790	0.782	0.774
40	0.766	0.759	0.751	0.743	0.735	0.728	0.720	0.712	0.705	0.697
50	0.690	0.683	0.675	0.668	0.661	0.654	0.646	0.639	0.632	0.625
60	0.618	0.612	0.605	0.598	0.591	0.585	0.578	0.572	0.565	0.559
70	0.552	0.546	0.540	0.534	0.528	0.522	0.516	0.510	0.504	0.498
80	0.493	0.487	0.481	0.476	0.470	0.465	0.460	0.454	0.449	0.444
90	0.439	0.434	0.429	0.424	0.419	0.414	0.410	0.405	0.401	0.397
100	0.394	0.390	0.387	0.383	0.380	0.376	0.373	0.370	0.366	0.363
110	0.359	0.356	0.353	0.350	0.346	0.343	0.340	0.337	0.334	0.331
120	0.328	0.325	0.322	0.319	0.316	0.313	0.310	0.307	0.304	0.301
130	0.299	0.296	0.293	0.290	0.288	0.285	0.282	0.280	0.277	0.275
140	0.272	0.270	0.267	0.265	0.262	0.260	0.258	0.255	0.253	0.251
150	0.248	0.246	0.244	0.242	0.240	0.237	0.235	0.233	0.231	0.229
160	0.227	0.225	0.223	0.221	0.219	0.217	0.215	0.213	0.212	0.210
170	0.208	0.206	0.204	0.203	0.201	0.199	0.197	0.196	0.194	0.192
180	0.191	0.189	0.188	0.186	0.184	0.183	0.181	0.180	0.178	0.177
190	0.176	0.174	0.173	0.171	0.170	0.168	0.167	0.166	0.164	0.163
200	0.162									

Note: 1 Values of φ in Tables C-1 through C-4 are calculated from the following formulae:

$$\varphi = 1 - \alpha_1\lambda_n^2 \quad \text{for } \lambda_n = \frac{\lambda}{\pi}\sqrt{\frac{f_y}{E}} \leqslant 0.215$$

$$\varphi = \frac{1}{2\lambda_n^2}\left[(\alpha_2 + \alpha_3\lambda_n + \lambda_n^2) - \sqrt{(\alpha_2 + \alpha_3\lambda_n + \lambda_n^2)^2 - 4\lambda_n^2}\right] \quad \text{for } \lambda_n > 0.215$$

where α_1, α_2, α_3 —— coefficients, taken from Table C-5 in accordance with the cross-section classification of Table 5.1.2 of this Code.

2 In case $\lambda\sqrt{f_y/235}$ of a member exceeds the entry of Tables C-1 through C-4, calculate φ value according to formulae given in Note 1.

Table C-5 Coefficients α_1, α_2, α_3

Class of sections		α_1	α_2	α_3
a		0.41	0.986	0.152
b		0.65	0.965	0.300
c	$\lambda_n \leqslant 1.05$	0.73	0.906	0.595
	$\lambda_n > 1.05$		1.216	0.302
d	$\lambda_n \leqslant 1.05$	1.35	0.868	0.915
	$\lambda_n > 1.05$		1.375	0.432

Appendix D Effective length factor for columns

Table D-1 Column effective length factor μ for frames without sidesway

K_2 \ K_1	0	0.05	0.1	0.2	0.3	0.4	0.5	1	2	3	4	5	≥10
0	1.000	0.990	0.981	0.964	0.949	0.935	0.922	0.875	0.820	0.791	0.773	0.760	0.732
0.05	0.990	0.981	0.971	0.955	0.940	0.926	0.914	0.867	0.814	0.784	0.766	0.754	0.726
0.1	0.981	0.971	0.962	0.946	0.931	0.918	0.906	0.860	0.807	0.778	0.760	0.748	0.721
0.2	0.964	0.955	0.946	0.930	0.916	0.903	0.891	0.846	0.795	0.767	0.749	0.737	0.711
0.3	0.949	0.940	0.931	0.916	0.902	0.889	0.878	0.834	0.784	0.756	0.739	0.728	0.701
0.4	0.935	0.926	0.918	0.903	0.889	0.877	0.866	0.823	0.774	0.747	0.730	0.719	0.693
0.5	0.922	0.914	0.906	0.891	0.878	0.866	0.855	0.813	0.765	0.738	0.721	0.710	0.685
1	0.875	0.867	0.860	0.846	0.834	0.823	0.813	0.774	0.729	0.704	0.688	0.677	0.654
2	0.820	0.814	0.807	0.795	0.784	0.774	0.765	0.729	0.686	0.663	0.648	0.638	0.615
3	0.791	0.784	0.778	0.767	0.756	0.747	0.738	0.704	0.663	0.640	0.625	0.616	0.593
4	0.773	0.766	0.760	0.749	0.739	0.730	0.721	0.688	0.648	0.625	0.611	0.601	0.580
5	0.760	0.754	0.748	0.737	0.728	0.719	0.710	0.677	0.638	0.616	0.601	0.592	0.570
≥10	0.732	0.726	0.721	0.711	0.701	0.693	0.685	0.654	0.615	0.593	0.580	0.570	0.549

Note: 1 Values of factor μ in this Table are calculated from the following equation

$$\left[\left(\frac{\pi}{\mu}\right)^2 + 2(K_1 + K_2) - 4K_1K_2\right]\frac{\pi}{\mu}\cdot\sin\frac{\pi}{\mu} - 2\left[(K_1 + K_2)\left(\frac{\pi}{\mu}\right)^2 + 4K_1K_2\right]\cos\frac{\pi}{\mu} + 8K_1K_2 = 0$$

where K_1, K_2——ratio of the sum of girder linear stiffness to that of column linear stiffness at the top and bottom of the calculated column respectively. In case the far end of a girder is hinged, its linear stiffness shall be multiplied by 1.5 while for a far end fixed girder, its linear stiffness shall be multiplied by 2.

2 The linear stiffness of a girder hinge-connected to the column is taken as zero.

3 For columns of the bottom story: take $K_2 = 0$ for columns hinged to the foundation ($K_2 = 0.1$ may be used for base plate support); take $K_2 = 10$ for columns rigidly fixed to the foundation.

4 In case a girder rigidly connected to the column is subject to large axial compression N_b, its linear stiffness shall be multiplied by a reduction factor α_N as follows:

For girders with far end rigidly connected to column, and also those with far end hinged

$$\alpha_N = 1 - N_b/N_{Eb}$$

For girders with far end fixed

$$\alpha_N = 1 - N_b/(2N_{Eb})$$

where $N_{Eb} = \pi^2 EI_b/l^2$, I_b is the moment of inertia of the girder section and l is girder length.

Table D-2 Column effective length factor μ for frames with sidesway

K_2 \ K_1	0	0.05	0.1	0.2	0.3	0.4	0.5	1	2	3	4	5	≥10
0	∞	6.02	4.46	3.42	3.01	2.78	2.64	2.33	2.17	2.11	2.08	2.07	2.03
0.05	6.02	4.16	3.47	2.86	2.58	2.42	2.31	2.07	1.94	1.90	1.87	1.86	1.83
0.1	4.46	3.47	3.01	2.56	2.33	2.20	2.11	1.90	1.79	1.75	1.73	1.72	1.70
0.2	3.42	2.86	2.56	2.23	2.05	1.94	1.87	1.70	1.60	1.57	1.55	1.54	1.52
0.3	3.01	2.58	2.33	2.05	1.90	1.80	1.74	1.58	1.49	1.46	1.45	1.44	1.42
0.4	2.78	2.42	2.20	1.94	1.80	1.71	1.65	1.50	1.42	1.39	1.37	1.37	1.35
0.5	2.64	2.31	2.11	1.87	1.74	1.65	1.59	1.45	1.37	1.34	1.32	1.32	1.30
1	2.33	2.07	1.90	1.70	1.58	1.50	1.45	1.32	1.24	1.21	1.20	1.19	1.17
2	2.17	1.94	1.79	1.60	1.49	1.42	1.37	1.24	1.16	1.14	1.12	1.12	1.10
3	2.11	1.90	1.75	1.57	1.46	1.39	1.34	1.21	1.14	1.11	1.10	1.09	1.07
4	2.08	1.87	1.73	1.55	1.45	1.37	1.32	1.20	1.12	1.10	1.08	1.08	1.06
5	2.07	1.86	1.72	1.54	1.44	1.37	1.32	1.19	1.12	1.09	1.08	1.07	1.05
≥10	2.03	1.83	1.70	1.52	1.42	1.35	1.30	1.17	1.10	1.07	1.06	1.05	1.03

Note: 1 Values of factor μ in this Table are calculated from the following equation

$$\left[36K_1K_2 - \left(\frac{\pi}{\mu}\right)^2\right]\sin\frac{\pi}{\mu} + 6(K_1 + K_2)\frac{\pi}{\mu}\cdot\cos\frac{\pi}{\mu} = 0$$

where K_1, K_2——ratio of the sum of girder linear stiffness to that of column linear stiffness at the top and bottom of the calculated column respectively. In case the far end of a girder is hinged, its linear stiffness shall be multiplied by 0.5 while for a far end fixed girder, its linear stiffness shall be multiplied by 2/3.

2 The linear stiffness of a girder hinge-connected to the column is taken as zero.

3 For columns of the bottom story: take $K_2 = 0$ for columns hinged to the foundation ($K_2 = 0.1$ may be used for base plate support); take $K_2 = 10$ for columns rigidly fixed to the foundation.

4 In case a girder rigidly connected to the column is subject to large axial compression N_b, its linear stiffness shall be multiplied by a reduction factor α_N as follows:

For girders with far end rigidly connected to column:

$$\alpha_N = 1 - N_b/(4N_{Eb})$$

For girders with far end hinged:

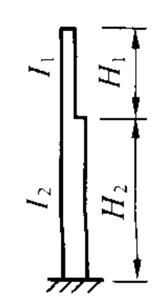
$$\alpha_N = 1 - N_b/N_{Eb}$$

For girders with far end fixed:

$$\alpha_N = 1 - N_b/(2N_{Eb})$$

See Note 4 of Table D-1 for calculation of N_{Eb}.

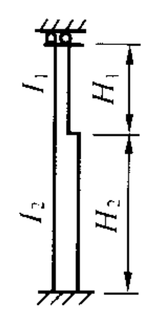
Table D-3 Effective length factor μ_2 for the lower portion of single-stepped columns with free upper end

Scheme	K_1 / η_1	0.06	0.08	0.10	0.12	0.14	0.16	0.18	0.20	0.22	0.24	0.26	0.28	0.3	0.4	0.5	0.6	0.7	0.8
	0.2	2.00	2.01	2.01	2.01	2.01	2.01	2.01	2.02	2.02	2.02	2.02	2.02	2.02	2.03	2.04	2.05	2.06	2.07
	0.3	2.01	2.02	2.02	2.02	2.03	2.03	2.03	2.04	2.04	2.05	2.05	2.05	2.06	2.08	2.10	2.12	2.13	2.15
	0.4	2.02	2.03	2.04	2.04	2.05	2.06	2.07	2.07	2.08	2.09	2.09	2.10	2.11	2.14	2.18	2.21	2.25	2.28
	0.5	2.04	2.05	2.06	2.07	2.09	2.10	2.11	2.12	2.13	2.15	2.16	2.17	2.18	2.24	2.29	2.35	2.40	2.45
	0.6	2.06	2.08	2.10	2.12	2.14	2.16	2.18	2.19	2.21	2.23	2.25	2.26	2.28	2.36	2.44	2.52	2.59	2.66
	0.7	2.10	2.13	2.16	2.18	2.21	2.24	2.26	2.29	2.31	2.34	2.36	2.38	2.41	2.52	2.62	2.72	2.81	2.90
	0.8	2.15	2.20	2.24	2.27	2.31	2.34	2.38	2.41	2.44	2.47	2.50	2.53	2.56	2.70	2.82	2.94	3.06	3.16
	0.9	2.24	2.29	2.35	2.39	2.44	2.48	2.52	2.56	2.60	2.63	2.67	2.71	2.74	2.90	3.05	3.19	3.32	3.44
	1.0	2.36	2.43	2.48	2.54	2.59	2.64	2.69	2.73	2.77	2.82	2.86	2.90	2.94	3.12	3.29	3.45	3.59	3.74
$K_1 = \frac{I_1}{I_2} \cdot \frac{H_2}{H_1}$	1.2	2.69	2.76	2.83	2.89	2.95	3.01	3.07	3.12	3.17	3.22	3.27	3.32	3.37	3.59	3.80	3.99	4.17	4.34
	1.4	3.07	3.14	3.22	3.29	3.36	3.42	3.48	3.55	3.61	3.66	3.72	3.78	3.83	4.09	4.33	4.56	4.77	4.97
$\eta_1 = \frac{H_1}{H_2}\sqrt{\frac{N_1}{N_2} \cdot \frac{I_2}{I_1}}$	1.6	3.47	3.55	3.63	3.71	3.78	3.85	3.92	3.99	4.07	4.12	4.18	4.25	4.31	4.61	4.88	5.14	5.38	5.62
	1.8	3.88	3.97	4.05	4.13	4.21	4.29	4.37	4.44	4.52	4.59	4.66	4.73	4.80	5.13	5.44	5.73	6.00	6.26
N_1, N_2: axial	2.0	4.29	4.39	4.48	4.57	4.65	4.74	4.82	4.90	4.99	5.07	5.14	5.22	5.30	5.66	6.00	6.32	6.63	6.92
forces of upper	2.2	4.71	4.81	4.91	5.00	5.10	5.19	5.28	5.37	5.46	5.54	5.63	5.71	5.80	6.19	6.57	6.92	7.26	7.58
and lower portions	2.4	5.13	5.24	5.34	5.44	5.54	5.64	5.74	5.84	5.93	6.03	6.12	6.21	6.30	6.73	7.14	7.52	7.89	8.24
respectively	2.6	5.55	5.66	5.77	5.88	5.99	6.10	6.20	6.31	6.41	6.51	6.61	6.71	6.80	7.27	7.71	8.13	8.52	8.90
	2.8	5.97	6.09	6.21	6.33	6.44	6.55	6.67	6.78	6.89	6.99	7.10	7.21	7.31	7.81	8.28	8.73	9.16	9.57
	3.0	6.39	6.52	6.64	6.77	6.89	7.01	7.13	7.25	7.37	7.48	7.59	7.71	7.82	8.35	8.86	9.34	9.80	10.24

Note: Values of factor μ_2 in this Table are calculated from the following equation:

$$\eta_1 K_1 \cdot \operatorname{tg}\frac{\pi}{\mu_2} \cdot \operatorname{tg}\frac{\pi \eta_1}{\mu_2} - 1 = 0$$

Table D-4 Effective length factor μ_2 for the lower portion of single-stepped columns with translation-free and rotation-fixed upper end

Scheme	K_1 / η_1	0.06	0.08	0.10	0.12	0.14	0.16	0.18	0.20	0.22	0.24	0.26	0.28	0.3	0.4	0.5	0.6	0.7	0.8
	0.2	1.96	1.94	1.93	1.91	1.90	1.89	1.88	1.86	1.85	1.84	1.83	1.82	1.81	1.76	1.72	1.68	1.65	1.62
	0.3	1.96	1.94	1.93	1.92	1.91	1.89	1.88	1.87	1.86	1.85	1.84	1.83	1.82	1.77	1.73	1.70	1.66	1.63
	0.4	1.96	1.95	1.94	1.92	1.91	1.90	1.89	1.88	1.87	1.86	1.85	1.84	1.83	1.79	1.75	1.72	1.68	1.66
	0.5	1.96	1.95	1.94	1.93	1.92	1.91	1.90	1.89	1.88	1.87	1.86	1.85	1.85	1.81	1.77	1.74	1.71	1.69
	0.6	1.97	1.96	1.95	1.94	1.93	1.92	1.91	1.90	1.90	1.89	1.88	1.87	1.87	1.83	1.80	1.78	1.75	1.73
	0.7	1.97	1.97	1.96	1.95	1.94	1.94	1.93	1.92	1.92	1.91	1.90	1.90	1.89	1.86	1.84	1.82	1.80	1.78
	0.8	1.98	1.98	1.97	1.96	1.96	1.95	1.95	1.94	1.94	1.93	1.93	1.93	1.92	1.90	1.88	1.87	1.86	1.84
	0.9	1.99	1.99	1.98	1.98	1.98	1.97	1.97	1.97	1.97	1.96	1.96	1.96	1.96	1.95	1.94	1.93	1.92	1.92
	1.0	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00
$K_1 = \frac{I_1}{I_2} \cdot \frac{H_2}{H_1}$	1.2	2.03	2.04	2.04	2.05	2.06	2.07	2.07	2.08	2.08	2.09	2.10	2.10	2.11	2.13	2.15	2.17	2.18	2.20
	1.4	2.07	2.09	2.11	2.12	2.14	2.16	2.17	2.18	2.20	2.21	2.22	2.23	2.24	2.29	2.33	2.37	2.40	2.42
$\eta_1 = \frac{H_1}{H_2}\sqrt{\frac{N_1}{N_2} \cdot \frac{I_2}{I_1}}$	1.6	2.13	2.16	2.19	2.22	2.25	2.27	2.30	2.32	2.34	2.36	2.37	2.39	2.41	2.48	2.54	2.59	2.63	2.67
	1.8	2.22	2.27	2.31	2.35	2.39	2.42	2.45	2.48	2.50	2.53	2.55	2.57	2.59	2.69	2.76	2.83	2.88	2.93
N_1, N_2: axial	2.0	2.35	2.41	2.46	2.50	2.55	2.59	2.62	2.66	2.69	2.72	2.75	2.77	2.80	2.91	3.00	3.08	3.14	3.20
forces of upper	2.2	2.51	2.57	2.63	2.68	2.73	2.77	2.81	2.85	2.89	2.92	2.95	2.98	3.01	3.14	3.25	3.33	3.41	3.47
and lower portions	2.4	2.68	2.75	2.81	2.87	2.92	2.97	3.01	3.05	3.09	3.13	3.17	3.20	3.24	3.38	3.50	3.59	3.68	3.75
respectively	2.6	2.87	2.94	3.00	3.06	3.12	3.17	3.22	3.27	3.31	3.35	3.39	3.43	3.46	3.62	3.75	3.86	3.95	4.03
	2.8	3.06	3.14	3.20	3.27	3.33	3.38	3.43	3.48	3.53	3.58	3.62	3.66	3.70	3.87	4.01	4.13	4.23	4.32
	3.0	3.26	3.34	3.41	3.47	3.54	3.60	3.65	3.70	3.75	3.80	3.85	3.89	3.93	4.12	4.27	4.40	4.51	4.61

Note: Values of factor μ_2 in this Table are calculated from the following equation:

$$\operatorname{tg}\frac{\pi \eta_1}{\mu_2} + \eta_1 K_1 \cdot \operatorname{tg}\frac{\pi}{\mu_2} = 0$$

Table D-5 Effective length factor μ_3 for the lower portion of double-stepped columns with free upper end

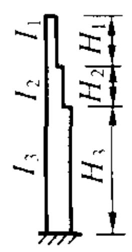
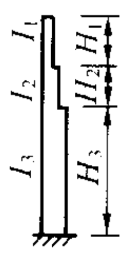
Scheme	η_1	η_2 \ K_2 (K_1 = 0.05)	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1	1.2
 $K_1 = \frac{I_1}{I_3} \cdot \frac{H_3}{H_1}$ $K_2 = \frac{I_2}{I_3} \cdot \frac{H_3}{H_2}$ $\eta_1 = \frac{H_1}{H_3}\sqrt{\frac{N_1}{N_3} \cdot \frac{I_3}{I_1}}$ $\eta_2 = \frac{H_2}{H_3}\sqrt{\frac{N_2}{N_3} \cdot \frac{I_3}{I_2}}$ N_1, N_2, N_3: axial forces of upper, middle and lower portions respectively	0.2	0.2	2.02	2.03	2.04	2.05	2.05	2.06	2.07	2.08	2.09	2.10	2.10
		0.4	2.08	2.11	2.15	2.19	2.22	2.25	2.29	2.32	2.35	2.39	2.42
		0.6	2.20	2.29	2.37	2.45	2.52	2.60	2.67	2.73	2.80	2.87	2.93
		0.8	2.42	2.57	2.71	2.83	2.95	3.06	3.17	3.27	3.37	3.47	3.56
		1.0	2.75	2.95	3.13	3.30	3.45	3.60	3.74	3.87	4.00	4.13	4.25
		1.2	3.13	3.38	3.60	3.80	4.00	4.18	4.35	4.51	4.67	4.82	4.97
	0.4	0.2	2.04	2.05	2.05	2.06	2.07	2.08	2.09	2.09	2.10	2.11	2.12
		0.4	2.10	2.14	2.17	2.20	2.24	2.27	2.31	2.34	2.37	2.40	2.43
		0.6	2.24	2.32	2.40	2.47	2.54	2.62	2.68	2.75	2.82	2.88	2.94
		0.8	2.47	2.60	2.73	2.85	2.97	3.08	3.19	3.29	3.38	3.48	3.57
		1.0	2.79	2.98	3.15	3.32	3.47	3.62	3.75	3.89	4.02	4.14	4.26
		1.2	3.18	3.41	3.62	3.82	4.01	4.19	4.36	4.52	4.68	4.83	4.98
	0.6	0.2	2.09	2.09	2.10	2.10	2.11	2.12	2.12	2.13	2.14	2.15	2.15
		0.4	2.17	2.19	2.22	2.25	2.28	2.31	2.34	2.38	2.41	2.44	2.47
		0.6	2.32	2.38	2.45	2.52	2.59	2.66	2.72	2.79	2.85	2.91	2.97
		0.8	2.56	2.67	2.79	2.90	3.01	3.11	3.22	3.32	3.41	3.50	3.60
		1.0	2.88	3.04	3.20	3.36	3.50	3.65	3.78	3.91	4.04	4.16	4.26
		1.2	3.26	3.46	3.66	3.86	4.04	4.22	4.38	4.55	4.70	4.85	5.00
	0.8	0.2	2.29	2.24	2.22	2.21	2.21	2.22	2.22	2.22	2.23	2.23	2.24
		0.4	2.37	2.34	2.34	2.36	2.38	2.40	2.43	2.45	2.48	2.51	2.54
		0.6	2.52	2.52	2.56	2.61	2.67	2.73	2.79	2.85	2.91	2.96	3.02
		0.8	2.74	2.79	2.88	2.98	3.08	3.17	3.27	3.36	3.46	3.55	3.63
		1.0	3.04	3.15	3.28	3.42	3.56	3.69	3.82	3.95	4.07	4.19	4.31
		1.2	3.39	3.55	3.73	3.91	4.08	4.25	4.42	4.58	4.73	4.88	5.02
	1.0	0.2	2.69	2.57	2.51	2.48	2.46	2.45	2.45	2.44	2.44	2.44	2.44
		0.4	2.75	2.64	2.60	2.59	2.59	2.59	2.60	2.62	2.63	2.65	2.67
		0.6	2.86	2.78	2.77	2.79	2.83	2.87	2.91	2.96	3.01	3.06	3.10
		0.8	3.04	3.01	3.05	3.11	3.19	3.27	3.35	3.44	3.52	3.61	3.69
		1.0	3.29	3.32	3.41	3.52	3.64	3.76	3.89	4.01	4.13	4.24	4.35
		1.2	3.60	3.69	3.83	3.99	4.15	4.31	4.47	4.62	4.77	4.92	5.06
	1.2	0.2	3.16	3.00	2.92	2.87	2.84	2.81	2.80	2.79	2.78	2.77	2.77
		0.4	3.21	3.05	2.98	2.94	2.92	2.90	2.90	2.90	2.90	2.91	2.92
		0.6	3.30	3.15	3.10	3.08	3.08	3.10	3.12	3.15	3.18	3.22	3.26
		0.8	3.43	3.32	3.30	3.33	3.37	3.43	3.49	3.56	3.63	3.71	3.78
		1.0	3.62	3.57	3.60	3.68	3.77	3.87	3.98	4.09	4.20	4.31	4.42
		1.2	3.88	3.88	3.98	4.11	4.25	4.39	4.54	4.68	4.83	4.97	5.10
	1.4	0.2	3.66	3.46	3.36	3.29	3.25	3.23	3.20	3.19	3.18	3.17	3.16
		0.4	3.70	3.50	3.40	3.35	3.31	3.29	3.27	3.26	3.26	3.26	3.26
		0.6	3.77	3.58	3.49	3.45	3.43	3.42	3.42	3.43	3.45	3.47	3.49
		0.8	3.87	3.70	3.64	3.63	3.64	3.67	3.70	3.75	3.81	3.86	3.92
		1.0	4.02	3.89	3.87	3.90	3.96	4.04	4.12	4.22	4.31	4.41	4.51
		1.2	4.23	4.15	4.19	4.27	4.39	4.51	4.64	4.77	4.91	5.04	5.17

Table D-5 (Continued)

Scheme



$$K_1 = \frac{I_1}{I_3} \cdot \frac{H_3}{H_1}$$

$$K_2 = \frac{I_2}{I_3} \cdot \frac{H_3}{H_2}$$

$$\eta_1 = \frac{H_1}{H_3}\sqrt{\frac{N_1}{N_3} \cdot \frac{I_3}{I_1}}$$

$$\eta_2 = \frac{H_2}{H_3}\sqrt{\frac{N_2}{N_3} \cdot \frac{I_3}{I_2}}$$

N_1, N_2, N_3: axial forces of upper, middle and lower portions respectively

K_1		0.10										
η_1	η_2 \ K_2	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1	1.2
0.2	0.2	2.03	2.03	2.04	2.05	2.06	2.07	2.08	2.08	2.09	2.10	2.11
	0.4	2.09	2.12	2.16	2.19	2.23	2.26	2.29	2.33	2.36	2.39	2.42
	0.6	2.21	2.30	2.38	2.46	2.53	2.60	2.67	2.74	2.81	2.87	2.93
	0.8	2.44	2.58	2.71	2.84	2.96	3.07	3.17	3.28	3.37	3.47	3.56
	1.0	2.76	2.96	3.14	3.30	3.46	3.60	3.74	3.88	4.01	4.13	4.25
	1.2	3.15	3.39	3.61	3.81	4.00	4.18	4.35	4.52	4.68	4.83	4.98
0.4	0.2	2.07	2.07	2.08	2.08	2.09	2.10	2.11	2.12	2.12	2.13	2.14
	0.4	2.14	2.17	2.20	2.23	2.26	2.30	2.33	2.36	2.39	4.42	2.46
	0.6	2.28	2.36	2.43	2.50	2.57	2.64	2.71	2.77	2.84	2.90	2.96
	0.8	2.53	2.65	2.77	2.88	3.00	3.10	3.21	3.31	3.40	3.50	3.59
	1.0	2.85	3.02	3.19	3.34	3.49	3.64	3.77	3.91	4.03	4.16	4.28
	1.2	3.24	3.45	3.65	3.85	4.03	4.21	4.38	4.54	4.70	4.85	4.99
0.6	0.2	2.22	2.19	2.18	2.17	2.18	2.18	2.19	2.19	2.20	2.20	2.21
	0.4	2.31	2.30	2.31	2.33	2.35	2.38	2.41	2.44	2.47	2.49	2.52
	0.6	2.48	2.49	2.54	2.60	2.66	2.72	2.78	2.84	2.90	2.96	3.02
	0.8	2.72	2.78	2.87	2.97	3.07	3.17	3.27	3.36	3.46	3.55	3.64
	1.0	3.04	3.15	3.28	3.42	3.56	3.70	3.83	3.95	4.08	4.20	4.31
	1.2	3.40	3.56	3.74	3.91	4.09	4.26	4.42	4.58	4.73	4.88	5.03
0.8	0.2	2.63	2.49	2.43	2.40	2.38	2.37	2.37	2.36	2.36	2.37	2.37
	0.4	2.71	2.59	2.53	2.55	2.54	2.55	2.57	2.59	2.61	2.63	2.65
	0.6	2.86	2.76	2.76	2.78	2.82	2.86	2.91	2.96	3.01	3.07	3.12
	0.8	3.06	3.02	3.06	3.13	3.20	3.29	3.37	3.46	3.54	3.63	3.71
	1.0	3.33	3.35	3.44	3.55	3.67	3.79	3.90	4.03	4.15	4.26	4.37
	1.2	3.65	3.73	3.86	4.02	4.18	4.34	4.49	4.64	4.79	4.94	5.08
1.0	0.2	3.18	2.95	2.84	2.77	2.73	2.70	2.68	2.67	2.66	2.65	2.65
	0.4	3.24	3.03	2.93	2.88	2.85	2.84	2.84	2.84	2.85	2.86	2.87
	0.6	3.36	3.16	3.09	3.07	3.08	3.09	3.12	3.15	3.19	3.23	3.27
	0.8	3.52	3.37	3.34	3.36	3.41	3.46	3.53	3.60	3.67	3.75	3.82
	1.0	3.74	3.64	3.67	3.74	3.83	3.93	4.03	4.14	4.25	4.35	4.46
	1.2	4.00	3.97	4.05	4.17	4.31	4.45	4.59	4.73	4.87	5.01	5.14
1.2	0.2	3.77	3.47	3.32	3.23	3.17	3.12	3.09	3.07	3.05	3.04	3.03
	0.4	3.82	3.53	3.39	3.31	3.26	3.22	3.20	3.19	3.19	3.19	3.19
	0.6	3.91	3.64	3.51	3.45	3.42	3.42	3.42	3.43	3.45	3.48	3.50
	0.8	4.04	3.80	3.71	3.68	3.69	3.72	3.76	3.81	3.86	3.92	3.98
	1.0	4.21	4.02	3.97	3.99	4.05	4.12	4.20	4.29	4.39	4.48	4.58
	1.2	4.43	4.30	4.31	4.38	4.48	4.60	4.72	4.85	4.98	5.11	5.24
1.4	0.2	4.37	4.01	3.82	3.71	3.63	3.58	3.54	3.51	3.49	3.47	3.45
	0.4	4.41	4.06	3.88	3.77	3.70	3.66	3.63	3.60	3.59	3.58	3.57
	0.6	4.48	4.15	3.98	3.89	3.83	3.80	3.79	3.78	3.79	3.80	3.81
	0.8	4.59	4.28	4.13	4.07	4.04	4.04	4.06	4.08	4.12	4.16	4.21
	1.0	4.74	4.45	4.35	4.32	4.34	4.38	4.43	4.50	4.58	4.66	4.74
	1.2	4.92	4.69	4.63	4.65	4.72	4.80	4.90	5.10	5.13	5.24	5.36

Table D-5 (Continued)

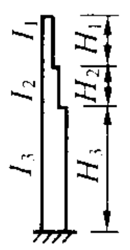
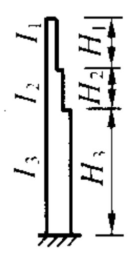
Scheme	η_1	K_1 / K_2 / η_2	0.20										
			0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1	1.2
 $K_1 = \frac{I_1}{I_3} \cdot \frac{H_3}{H_1}$ $K_2 = \frac{I_2}{I_3} \cdot \frac{H_3}{H_2}$ $\eta_1 = \frac{H_1}{H_3}\sqrt{\frac{N_1}{N_3} \cdot \frac{I_3}{I_1}}$ $\eta_2 = \frac{H_2}{H_3}\sqrt{\frac{N_2}{N_3} \cdot \frac{I_3}{I_2}}$ N_1, N_2, N_3: axial forces of upper, middle and lower portions respectively	0.2	0.2	2.04	2.04	2.05	2.06	2.07	2.08	2.08	2.09	2.10	2.11	2.12
		0.4	2.10	2.13	2.17	2.20	2.24	2.27	2.30	2.34	2.37	2.40	2.43
		0.6	2.23	2.31	2.39	2.47	2.54	2.61	2.68	2.75	2.82	2.88	2.94
		0.8	2.46	2.60	2.73	2.85	2.97	3.08	3.18	3.29	3.38	3.48	3.57
		1.0	2.79	2.98	3.15	3.32	3.47	3.61	3.75	3.89	4.02	4.14	4.26
		1.2	3.18	3.41	3.62	3.82	4.01	4.19	4.36	4.52	4.68	4.83	4.98
	0.4	0.2	2.15	2.13	2.13	2.14	2.14	2.15	2.15	2.16	2.17	2.17	2.18
		0.4	2.24	2.24	2.26	2.29	2.32	2.35	2.38	2.41	2.44	2.47	2.50
		0.6	2.40	2.44	2.60	2.56	2.63	2.69	2.76	2.82	2.88	2.94	3.00
		0.8	2.66	2.74	2.84	2.95	3.05	3.15	3.25	3.35	3.44	3.53	3.62
		1.0	2.98	3.12	3.25	3.40	3.54	3.68	3.81	3.94	4.07	4.19	4.30
		1.2	3.35	3.53	3.71	3.90	4.08	4.25	4.41	4.57	4.73	4.87	5.02
	0.6	0.2	2.57	2.42	2.37	2.34	2.33	2.32	2.32	2.32	2.32	2.32	2.33
		0.4	2.67	2.54	2.50	2.50	2.51	2.52	2.54	2.56	2.58	2.61	2.63
		0.6	2.83	2.74	2.73	2.76	2.80	2.85	2.90	2.96	3.01	3.06	3.12
		0.8	3.06	3.01	3.05	3.12	3.20	3.29	3.38	3.46	3.55	3.63	3.72
		1.0	3.34	3.35	3.44	3.56	3.68	3.80	3.92	4.04	4.15	4.27	4.38
		1.2	3.67	3.74	3.88	4.03	4.19	4.35	4.50	4.65	4.80	4.94	5.08
	0.8	0.2	3.25	2.96	2.82	2.74	2.69	2.66	2.64	2.62	2.61	2.61	2.60
		0.4	3.33	3.05	2.93	2.87	2.84	2.83	2.83	2.83	2.84	2.85	2.87
		0.6	3.45	3.21	3.12	3.10	3.10	3.12	3.14	3.18	3.22	3.26	3.30
		0.8	3.63	3.44	3.39	3.41	3.45	3.51	3.57	3.64	3.71	3.79	3.86
		1.0	3.86	3.73	3.73	3.80	3.88	3.98	4.08	4.18	4.29	4.39	4.50
		1.2	4.13	4.07	4.13	4.24	4.36	4.50	4.64	4.78	4.91	5.05	5.18
	1.0	0.2	4.00	3.60	3.39	3.26	3.18	3.13	3.08	3.05	3.03	3.01	3.00
		0.4	4.06	3.67	3.48	3.37	3.30	3.26	3.23	3.21	3.21	3.20	3.20
		0.6	4.15	3.79	3.63	3.54	3.50	3.48	3.49	3.50	3.51	3.54	3.57
		0.8	4.29	3.97	3.84	3.80	3.79	3.81	3.85	3.90	3.95	4.01	4.07
		1.0	4.48	4.21	4.13	4.13	4.17	4.23	4.31	4.39	4.48	4.57	4.66
		1.2	4.70	4.49	4.47	4.52	4.60	4.71	4.82	4.94	5.07	5.19	5.31
	1.2	0.2	4.76	4.26	4.00	3.83	3.72	3.65	3.59	3.54	3.51	3.48	3.46
		0.4	4.81	4.32	4.07	3.91	3.82	3.75	3.70	3.67	3.65	3.63	3.62
		0.6	4.89	4.43	4.19	4.05	3.98	3.93	3.91	3.89	3.89	3.90	3.91
		0.8	5.00	4.57	4.36	4.26	4.21	4.20	4.21	4.23	4.26	4.30	4.34
		1.0	5.15	4.76	4.59	4.53	4.53	4.55	4.60	4.66	4.73	4.80	4.88
		1.2	5.34	5.00	4.88	4.87	4.91	4.98	5.07	5.17	5.27	5.38	5.49
	1.4	0.2	5.53	4.94	4.62	4.42	4.29	4.19	4.12	4.06	4.02	3.98	3.95
		0.4	5.57	4.99	4.68	4.49	4.36	4.27	4.21	4.16	4.13	4.10	4.08
		0.6	5.64	5.07	4.78	4.60	4.49	4.42	4.38	4.35	4.33	4.32	4.32
		0.8	5.74	5.19	4.92	4.77	4.69	4.64	4.62	4.62	4.63	4.65	4.67
		1.0	5.86	5.35	5.12	5.00	4.95	4.94	4.96	4.99	5.03	5.09	5.15
		1.2	6.02	5.55	5.36	5.29	5.28	5.31	5.37	5.44	5.52	5.61	5.71

Table D-5 (Continued)

η_1	η_2 \ K_1 = 0.30, K_2 =	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1	1.2
0.2	0.2	2.05	2.05	2.06	2.07	2.08	2.09	2.09	2.10	2.11	2.12	2.13
	0.4	2.12	2.15	2.18	2.21	2.25	2.28	2.31	2.35	2.38	2.41	2.44
	0.6	2.25	2.33	2.41	2.48	2.56	2.63	2.69	2.76	2.83	2.89	2.95
	0.8	2.49	2.62	2.75	2.87	2.98	3.09	3.20	3.30	3.39	3.49	3.58
	1.0	3.82	3.00	3.17	3.33	3.48	3.63	3.76	3.90	4.02	4.15	4.27
	1.2	3.20	3.43	3.64	3.83	4.02	4.20	4.37	4.53	4.69	4.84	4.99
0.4	0.2	2.26	2.21	2.20	2.19	2.19	2.20	2.20	2.21	2.21	2.22	2.23
	0.4	2.36	2.33	2.33	2.35	2.38	2.40	2.43	2.46	2.49	2.51	2.54
	0.6	2.54	2.54	2.58	2.63	2.69	2.75	2.81	2.87	2.93	2.99	3.04
	0.8	2.79	2.83	2.91	3.01	3.10	3.20	3.30	3.39	3.48	3.57	3.66
	1.0	3.11	3.20	3.32	3.46	3.59	3.72	3.85	3.98	4.10	4.22	4.33
	1.2	3.47	3.60	3.77	3.95	4.12	4.28	4.45	4.60	4.75	4.90	5.04
0.6	0.2	2.93	2.68	2.57	2.52	2.49	2.47	2.46	2.45	2.45	2.45	2.45
	0.4	3.02	2.79	2.71	2.67	2.66	2.66	2.67	2.69	2.70	2.72	2.74
	0.6	3.17	2.98	2.93	2.93	2.95	2.98	3.02	3.07	3.11	3.16	3.21
	0.8	3.37	3.24	3.23	3.27	3.33	3.41	3.48	3.56	3.64	3.72	3.80
	1.0	3.63	3.56	3.60	3.69	3.79	3.90	4.01	4.12	4.23	4.34	4.45
	1.2	3.94	3.92	4.02	4.15	4.29	4.43	4.58	4.72	4.87	5.01	5.14
0.8	0.2	3.78	3.38	3.18	3.06	2.98	2.93	2.89	2.86	2.84	2.83	2.82
	0.4	3.85	3.47	3.28	3.18	3.12	3.09	3.07	3.06	3.06	3.06	3.06
	0.6	3.96	3.61	3.46	3.39	3.36	3.35	3.36	3.38	3.41	3.44	3.47
	0.8	4.12	3.82	3.70	3.67	3.68	3.72	3.76	3.82	3.88	3.94	4.01
	1.0	4.32	4.07	4.01	4.03	4.08	4.16	4.24	4.33	4.43	4.52	4.62
	1.2	4.57	4.38	4.38	4.44	4.54	4.66	4.78	4.90	5.03	5.16	5.29
1.0	0.2	4.68	4.15	3.86	3.69	3.57	3.49	3.43	3.38	3.35	3.32	3.30
	0.4	4.73	4.21	3.94	3.78	3.68	3.61	3.57	3.54	3.51	3.50	3.49
	0.6	4.82	4.33	4.08	3.95	3.87	3.83	3.80	3.80	3.80	3.81	3.83
	0.8	4.94	4.49	4.28	4.18	4.14	4.13	4.14	4.17	4.20	4.25	4.29
	1.0	5.10	4.70	4.53	4.48	4.48	4.51	4.56	4.62	4.70	4.77	4.85
	1.2	5.30	4.95	4.84	4.83	4.88	4.96	5.05	5.15	5.26	5.37	5.48
1.2	0.2	5.58	4.93	4.57	4.35	4.20	4.10	4.01	3.95	3.90	3.86	3.83
	0.4	5.62	4.98	4.64	4.43	4.29	4.19	4.12	4.07	4.03	4.01	3.98
	0.6	5.70	5.08	4.75	4.56	4.44	4.37	4.32	4.29	4.27	4.26	4.26
	0.8	5.80	5.21	4.91	4.75	4.66	4.61	4.59	4.59	4.60	4.62	4.65
	1.0	5.93	5.38	5.12	5.00	4.95	4.94	4.95	4.99	5.03	5.09	5.15
	1.2	6.10	5.59	5.38	5.31	5.30	5.33	5.39	5.46	5.54	5.63	5.73
1.4	0.2	6.49	5.72	5.30	5.03	4.85	4.72	4.62	4.54	4.48	4.43	4.38
	0.4	6.53	5.77	5.35	5.10	4.93	4.80	4.71	4.64	4.59	4.55	4.51
	0.6	6.59	5.85	5.45	5.21	5.05	4.95	4.87	4.82	4.78	4.76	4.74
	0.8	6.68	5.96	5.59	5.37	5.24	5.15	5.10	5.08	5.06	5.06	5.07
	1.0	6.79	6.10	5.76	5.58	5.48	5.43	5.41	5.41	5.44	5.47	5.51
	1.2	6.93	6.28	5.98	5.84	5.78	5.76	5.79	5.83	5.89	5.95	6.03

Scheme



$$K_1 = \frac{I_1}{I_3} \cdot \frac{H_3}{H_1}$$

$$K_2 = \frac{I_2}{I_3} \cdot \frac{H_3}{H_2}$$

$$\eta_1 = \frac{H_1}{H_3}\sqrt{\frac{N_1}{N_3} \cdot \frac{I_3}{I_1}}$$

$$\eta_2 = \frac{H_2}{H_3}\sqrt{\frac{N_2}{N_3} \cdot \frac{I_3}{I_2}}$$

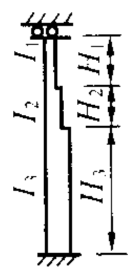
N_1, N_2, N_3: axial forces of upper, middle and lower portions respectively

Note: Values of factor μ_3 in this Table are calculated from the following equation:

$$\frac{\eta_1 K_1}{\eta_2 K_2} \cdot \mathrm{tg}\frac{\pi\eta_1}{\mu_3} \cdot \mathrm{tg}\frac{\pi\eta_2}{\mu_3} + \eta_1 K_1 \cdot \mathrm{tg}\frac{\pi\eta_1}{\mu_3} \cdot \mathrm{tg}\frac{\pi}{\mu_3} + \eta_2 K_2 \cdot \mathrm{tg}\frac{\pi\eta_2}{\mu_3} \cdot \mathrm{tg}\frac{\pi}{\mu_3} - 1 = 0$$

Table D-6 Effective length factor μ_3 for the lower portion of double-stepped columns with translation-free and rotation-fixed upper end

Scheme:



$$K_1 = \frac{I_1}{I_3} \cdot \frac{H_3}{H_1}$$

$$K_2 = \frac{I_2}{I_3} \cdot \frac{H_3}{H_2}$$

$$\eta_1 = \frac{H_1}{H_3}\sqrt{\frac{N_1}{N_3} \cdot \frac{I_3}{I_1}}$$

$$\eta_2 = \frac{H_2}{H_3}\sqrt{\frac{N_2}{N_3} \cdot \frac{I_3}{I_2}}$$

N_1, N_2, N_3: axial forces of upper, middle and lower portions respectively

η_1	η_2 \ K_2 (K_1 = 0.05)	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1	1.2
0.2	0.2	1.99	1.99	2.00	2.00	2.01	2.02	2.02	2.03	2.04	2.05	2.06
	0.4	2.03	2.06	2.09	2.12	2.16	2.19	2.22	2.25	2.29	2.32	2.35
	0.6	2.12	2.20	2.28	2.36	2.43	2.50	2.57	2.64	2.71	2.77	2.83
	0.8	2.28	2.43	2.57	2.70	2.82	2.94	3.04	3.15	3.25	3.34	3.43
	1.0	2.53	2.76	2.96	3.13	3.29	3.44	3.59	3.72	3.85	3.98	4.10
	1.2	2.86	3.15	3.39	3.61	3.80	3.99	4.16	4.33	4.49	4.64	4.79
0.4	0.2	1.99	1.99	2.00	2.01	2.01	2.02	2.03	2.04	2.04	2.05	2.06
	0.4	2.03	2.06	2.09	2.13	2.16	2.19	2.33	2.26	2.29	2.32	2.35
	0.6	2.12	2.20	2.28	2.36	2.44	2.51	2.58	2.64	2.71	2.77	2.84
	0.8	2.29	2.44	2.58	2.71	2.83	2.94	3.05	3.15	3.25	3.35	3.44
	1.0	2.54	2.77	2.96	3.14	3.30	3.45	3.59	3.73	3.85	3.98	4.10
	1.2	2.87	3.15	3.40	3.61	3.81	3.99	4.17	4.33	4.49	4.65	4.79
0.6	0.2	1.99	1.98	2.00	2.01	2.02	2.03	2.04	2.04	2.05	2.06	2.07
	0.4	2.04	2.07	2.10	2.14	2.17	2.20	2.23	2.27	2.30	2.33	2.36
	0.6	2.13	2.21	2.29	2.37	2.45	2.52	2.59	2.65	2.72	2.78	2.84
	0.8	2.30	2.45	2.59	2.72	2.84	2.95	3.06	3.16	3.26	3.35	3.44
	1.0	2.56	2.78	2.97	3.15	3.31	3.46	3.60	3.73	3.86	3.99	4.11
	1.2	2.89	3.17	3.41	3.62	3.82	4.00	4.17	4.34	4.50	4.65	4.80
0.8	0.2	2.00	2.01	2.02	2.02	2.03	2.04	2.05	2.05	2.06	2.07	2.08
	0.4	2.05	2.08	2.12	2.15	2.18	2.21	2.25	2.28	2.31	2.34	2.37
	0.6	2.15	2.23	2.31	2.39	2.46	2.53	2.60	2.67	2.73	2.79	2.85
	0.8	2.32	2.47	2.61	2.73	2.85	2.96	3.07	3.17	3.27	3.36	3.45
	1.0	2.59	2.80	2.99	3.16	3.32	3.47	3.61	3.74	3.87	3.99	4.11
	1.2	2.92	3.19	3.42	3.63	3.83	4.01	4.18	4.35	4.51	4.66	4.81
1.0	0.2	2.02	2.02	2.03	2.04	2.05	2.05	2.06	2.07	2.08	2.09	2.09
	0.4	2.07	2.10	2.14	2.17	2.20	2.23	2.26	2.30	2.33	2.36	2.39
	0.6	2.17	2.26	2.33	2.41	2.48	2.55	2.62	2.68	2.75	2.81	2.87
	0.8	2.36	2.50	2.63	2.76	2.87	2.98	3.08	3.19	3.28	3.38	3.47
	1.0	2.62	2.83	3.01	3.18	3.34	3.48	3.62	3.75	3.88	4.01	4.12
	1.2	2.95	3.21	3.44	3.65	3.82	4.02	4.20	4.36	4.52	4.67	4.81
1.2	0.2	2.04	2.05	2.06	2.06	2.07	2.08	2.09	2.09	2.10	2.11	2.12
	0.4	2.10	2.13	2.17	2.20	2.23	2.26	2.29	2.32	2.35	2.38	2.41
	0.6	2.22	2.29	2.37	2.44	2.51	2.58	2.64	2.71	2.77	2.83	2.89
	0.8	2.41	2.54	2.67	2.78	2.90	3.00	3.11	3.20	3.30	3.39	3.48
	1.0	2.68	2.87	3.04	3.21	3.36	3.50	3.64	3.77	3.90	4.02	4.14
	1.2	3.00	3.25	3.47	3.67	3.86	4.04	4.21	4.37	4.53	4.68	4.83
1.4	0.2	2.10	2.10	2.10	2.11	2.11	2.12	2.13	2.13	2.14	2.15	2.15
	0.4	2.17	2.19	2.21	2.24	2.27	2.30	2.33	2.36	2.39	2.41	2.44
	0.6	2.29	2.35	2.41	2.48	2.55	2.61	2.67	2.74	2.80	2.86	2.91
	0.8	2.48	2.60	2.71	2.82	2.93	3.03	3.13	3.23	3.32	3.41	3.50
	1.0	2.74	2.92	3.08	3.24	3.39	3.53	3.66	3.79	3.92	4.04	4.15
	1.2	3.06	3.29	3.50	3.70	3.89	4.06	4.23	4.39	4.55	4.70	4.84

Table D-6 (Continued)

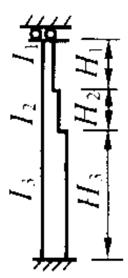
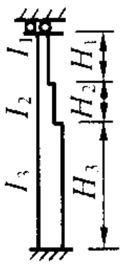
Scheme	η_1	K_1 / K_2 / η_2	0.10										
			0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1	1.2
 $K_1 = \frac{I_1}{I_3} \cdot \frac{H_3}{H_1}$ $K_2 = \frac{I_2}{I_3} \cdot \frac{H_3}{H_2}$ $\eta_1 = \frac{H_1}{H_3}\sqrt{\frac{N_1}{N_3} \cdot \frac{I_3}{I_1}}$ $\eta_2 = \frac{H_2}{H_3}\sqrt{\frac{N_2}{N_3} \cdot \frac{I_3}{I_2}}$ N_1, N_2, N_3: axial forces of upper, middle and lower portions respectively	0.2	0.2	1.96	1.96	1.97	1.97	1.98	1.98	1.99	2.00	2.00	2.01	2.02
		0.4	2.00	2.02	2.05	2.08	2.11	2.14	2.17	2.20	2.23	2.26	2.29
		0.6	2.07	2.14	2.22	2.29	2.36	2.43	2.50	2.56	2.63	2.69	2.75
		0.8	2.20	2.35	2.48	2.61	2.73	2.84	2.94	3.05	3.14	3.24	3.33
		1.0	2.41	2.64	2.83	3.01	3.17	3.32	3.46	3.59	3.72	3.85	3.97
		1.2	2.70	2.99	3.23	3.45	3.65	3.84	4.01	4.18	4.34	4.49	4.64
	0.4	0.2	1.96	1.97	1.97	1.98	1.98	1.99	2.00	2.00	2.01	2.02	2.03
		0.4	2.00	2.03	2.06	2.09	2.12	2.15	2.18	2.21	2.24	2.27	2.30
		0.6	2.08	2.15	2.23	2.30	2.37	2.44	2.51	2.57	2.64	2.70	2.76
		0.8	2.21	2.36	2.49	2.62	2.73	2.85	2.95	3.05	3.15	3.24	3.34
		1.0	2.43	2.65	2.84	3.02	3.18	3.33	3.47	3.60	3.73	3.85	3.97
		1.2	2.71	3.00	3.24	3.46	3.66	3.85	4.02	4.19	4.34	4.49	4.64
	0.6	0.2	1.97	1.98	1.98	1.99	2.00	2.00	2.01	2.02	2.02	2.03	2.04
		0.4	2.01	2.04	2.07	2.10	2.13	2.16	2.19	2.22	2.26	2.29	2.32
		0.6	2.09	2.17	2.24	2.32	2.39	2.46	2.52	2.59	2.65	2.71	2.77
		0.8	2.23	2.38	2.51	2.64	2.75	2.86	2.97	3.07	3.16	3.26	3.35
		1.0	2.45	2.68	2.86	3.03	3.19	3.34	3.48	3.61	3.74	3.86	3.98
		1.2	2.74	3.02	3.26	3.48	3.67	3.86	4.03	4.20	4.35	4.50	4.65
	0.8	0.2	1.99	1.99	2.00	2.01	2.01	2.02	2.03	2.04	2.04	2.05	2.06
		0.4	2.03	2.06	2.09	2.12	2.15	2.19	2.22	2.25	2.28	2.31	2.34
		0.6	2.12	2.19	2.27	2.34	2.41	2.48	2.55	2.61	2.67	2.73	2.79
		0.8	2.27	2.41	2.54	2.66	2.78	2.89	2.99	3.09	3.18	3.28	3.37
		1.0	2.49	2.70	2.89	3.06	3.21	3.36	3.50	3.63	3.76	3.88	4.00
		1.2	2.78	3.05	3.29	3.50	3.69	3.88	4.05	4.21	4.37	4.52	4.66
	1.0	0.2	2.01	2.02	2.03	2.04	2.04	2.05	2.06	2.07	2.07	2.08	2.09
		0.4	2.06	2.10	2.13	2.16	2.19	2.22	2.25	2.28	2.31	2.34	2.37
		0.6	2.16	2.24	2.31	2.38	2.45	2.51	2.58	2.64	2.70	2.76	2.82
		0.8	2.32	2.46	2.58	2.70	2.81	2.92	3.02	3.12	3.21	3.30	3.39
		1.0	2.55	2.75	2.93	3.09	3.25	3.39	3.53	3.66	3.78	3.90	4.02
		1.2	2.84	3.10	3.32	3.53	3.72	3.90	4.07	4.23	4.39	4.54	4.68
	1.2	0.2	2.07	2.08	2.08	2.09	2.09	2.10	2.11	2.11	2.12	2.13	2.13
		0.4	2.13	2.16	2.18	2.21	2.24	2.27	2.30	2.33	2.35	2.38	2.41
		0.6	2.24	2.30	2.37	2.43	2.50	2.56	2.63	2.68	2.74	2.80	2.86
		0.8	2.41	2.53	2.64	2.75	2.86	2.96	3.06	3.15	3.24	3.33	3.42
		1.0	2.64	2.82	2.98	3.14	3.29	3.43	3.56	3.69	3.81	3.93	4.04
		1.2	2.92	3.16	3.37	3.57	3.76	3.93	4.10	4.26	4.41	4.56	4.70
	1.4	0.2	2.20	2.18	2.17	2.17	2.17	2.18	2.18	2.19	2.19	2.20	2.20
		0.4	2.26	2.26	2.27	2.29	2.32	2.34	2.37	2.39	2.42	2.44	2.47
		0.6	2.37	2.41	2.46	2.51	2.57	2.63	2.68	2.74	2.80	2.85	2.91
		0.8	2.53	2.62	2.72	2.82	2.92	3.01	3.11	3.20	3.29	3.37	3.46
		1.0	2.75	2.90	3.05	3.20	3.34	3.47	3.60	3.72	3.84	3.96	4.07
		1.2	3.02	3.23	3.43	3.62	3.80	3.97	4.13	4.29	4.44	4.59	4.73

Table D-6 (Continued)

Scheme



$$K_1 = \frac{I_1}{I_3} \cdot \frac{H_3}{H_1}$$

$$K_2 = \frac{I_2}{I_3} \cdot \frac{H_3}{H_2}$$

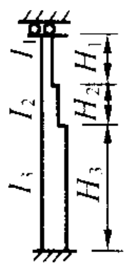
$$\eta_1 = \frac{H_1}{H_3}\sqrt{\frac{N_1}{N_3} \cdot \frac{I_3}{I_1}}$$

$$\eta_2 = \frac{H_2}{H_3}\sqrt{\frac{N_2}{N_3} \cdot \frac{I_3}{I_2}}$$

N_1, N_2, N_3: axial forces of upper, middle and lower portions respectively

η_1	η_2 \ K_2 (K_1 = 0.20)	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1	1.2
0.2	0.2	1.94	1.93	1.93	1.93	1.93	1.93	1.94	1.94	1.95	1.95	1.96
	0.4	1.96	1.98	1.99	2.02	2.04	2.07	2.09	2.12	2.15	2.17	2.20
	0.6	2.02	2.07	2.13	2.19	2.26	2.32	2.38	2.44	2.50	2.56	2.62
	0.8	2.12	2.23	2.35	2.47	2.58	2.68	2.78	2.88	2.98	3.07	3.15
	1.0	2.28	2.47	2.65	2.82	2.97	3.12	3.26	3.39	3.51	3.63	3.75
	1.2	2.50	2.77	3.01	3.22	3.42	3.60	3.77	3.93	4.09	4.23	4.38
0.4	0.2	1.93	1.93	1.93	1.93	1.94	1.94	1.95	1.95	1.96	1.96	1.97
	0.4	1.97	1.98	2.00	2.03	2.05	2.08	2.11	2.13	2.16	2.19	2.22
	0.6	2.03	2.08	2.14	2.21	2.27	2.33	2.40	2.46	2.52	2.58	2.63
	0.8	2.13	2.25	2.37	2.48	2.59	2.70	2.80	2.90	2.99	3.08	3.17
	1.0	2.29	2.49	2.67	2.83	2.99	3.13	3.27	3.40	3.53	3.64	3.76
	1.2	2.52	2.79	3.02	3.23	3.43	3.61	3.78	3.94	4.10	4.24	4.39
0.6	0.2	1.95	1.95	1.95	1.95	1.96	1.96	1.97	1.97	1.98	1.98	1.99
	0.4	1.98	2.00	2.02	2.05	2.08	2.10	2.13	2.16	2.19	2.21	2.24
	0.6	2.04	2.10	2.17	2.23	2.30	2.36	2.42	2.48	2.54	2.60	2.66
	0.8	2.15	2.27	2.39	2.51	2.62	2.72	2.82	2.92	3.01	3.10	3.19
	1.0	2.32	2.52	2.70	2.86	3.01	3.16	3.29	3.42	3.55	3.66	3.78
	1.2	2.55	2.82	3.05	3.26	3.45	3.63	3.80	3.96	4.11	4.26	4.40
0.8	0.2	1.97	1.97	1.98	1.98	1.99	1.99	2.00	2.01	2.01	2.02	2.03
	0.4	2.00	2.03	2.06	2.08	2.11	2.14	2.17	2.20	2.22	2.25	2.28
	0.6	2.08	2.14	2.21	2.27	2.34	2.40	2.46	2.52	2.58	2.64	2.69
	0.8	2.19	2.32	2.44	2.55	2.66	2.76	2.86	2.96	3.05	3.13	3.22
	1.0	2.37	2.57	2.74	2.90	3.05	3.19	3.33	3.45	3.58	3.69	3.81
	1.2	2.61	2.87	3.09	3.30	3.49	3.66	3.83	3.99	4.14	4.29	4.42
1.0	0.2	2.01	2.02	2.03	2.03	2.04	2.05	2.05	2.06	2.07	2.07	2.08
	0.4	2.06	2.09	2.11	2.14	2.17	2.20	2.23	2.25	2.28	2.31	2.33
	0.6	2.14	2.21	2.27	2.34	2.40	2.46	2.52	2.58	2.63	2.69	2.74
	0.8	2.27	2.39	2.51	2.62	2.72	2.82	2.91	3.00	3.09	3.18	3.26
	1.0	2.46	2.64	2.81	2.96	3.10	3.24	3.37	3.50	3.61	3.73	3.84
	1.2	2.69	2.94	3.15	3.35	3.53	3.71	3.87	4.02	4.17	4.32	4.46
1.2	0.2	2.13	2.12	2.12	2.13	2.13	2.14	2.14	2.15	2.15	2.16	2.16
	0.4	2.18	2.19	2.21	2.24	2.26	2.29	2.31	2.34	2.36	2.38	2.41
	0.6	2.27	2.32	2.37	2.43	2.49	2.54	2.60	2.65	2.70	2.76	2.81
	0.8	2.41	2.50	2.60	2.70	2.80	2.89	2.98	3.07	3.15	3.23	3.32
	1.0	2.59	2.74	2.89	3.04	3.17	3.30	3.43	3.55	3.66	3.78	3.89
	1.2	2.81	3.03	3.23	3.42	3.59	3.76	3.92	4.07	4.22	4.36	4.49
1.4	0.2	2.35	2.31	2.29	2.28	2.27	2.27	2.27	2.27	2.27	2.28	2.28
	0.4	2.40	2.37	2.37	2.38	2.39	2.41	2.43	2.45	2.47	2.49	2.51
	0.6	2.48	2.49	2.52	2.56	2.61	2.65	2.70	2.75	2.80	2.85	2.89
	0.8	2.60	2.66	2.73	2.82	2.90	2.98	3.07	3.15	3.23	3.31	3.38
	1.0	2.77	2.88	3.01	3.14	3.26	3.38	3.50	3.62	3.73	3.84	3.94
	1.2	2.97	3.15	3.33	3.50	3.67	3.83	3.98	4.13	4.27	4.41	4.54

Table D-6 (Continued)

Scheme	η_1	η_2 \ K_2 (K_1 = 0.30)	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1	1.2
 $K_1 = \frac{I_1}{I_3} \cdot \frac{H_3}{H_1}$ $K_2 = \frac{I_2}{I_3} \cdot \frac{H_3}{H_2}$ $\eta_1 = \frac{H_1}{H_3}\sqrt{\frac{N_1}{N_3} \cdot \frac{I_3}{I_1}}$ $\eta_2 = \frac{H_2}{H_3}\sqrt{\frac{N_2}{N_3} \cdot \frac{I_3}{I_2}}$ N_1, N_2, N_3: axial forces of upper, middle and lower portions respectively	0.2	0.2	1.92	1.91	1.90	1.89	1.89	1.89	1.90	1.90	1.90	1.90	1.91
		0.4	1.95	1.95	1.96	1.97	1.99	2.01	2.04	2.06	2.08	2.11	2.13
		0.6	1.99	2.03	2.08	2.13	2.18	2.24	2.29	2.35	2.41	2.46	2.52
		0.8	2.07	2.16	2.27	2.37	2.47	2.57	2.66	2.75	2.84	2.93	3.01
		1.0	2.20	2.37	2.53	2.69	2.83	2.97	3.10	3.23	3.35	3.46	3.57
		1.2	2.39	2.63	2.85	3.05	3.24	3.42	3.58	3.74	3.89	4.03	4.17
	0.4	0.2	1.92	1.91	1.91	1.90	1.90	1.91	1.91	1.91	1.92	1.92	1.92
		0.4	1.95	1.96	1.97	1.99	2.01	2.03	2.05	2.08	2.10	2.12	2.15
		0.6	2.00	2.04	2.09	2.14	2.20	2.26	2.31	2.37	2.42	2.48	2.53
		0.8	2.08	2.18	2.28	2.39	2.49	2.59	2.68	2.77	2.86	2.95	3.03
		1.0	2.22	2.39	2.55	2.71	2.85	2.99	3.12	3.24	3.36	3.48	3.59
		1.2	2.41	2.65	2.87	3.07	3.26	3.43	3.60	3.75	3.90	4.04	4.18
	0.6	0.2	1.93	1.93	1.92	1.92	1.93	1.93	1.93	1.94	1.94	1.95	1.95
		0.4	1.96	1.97	1.99	2.01	2.03	2.06	2.08	2.11	2.13	2.16	2.18
		0.6	2.02	2.06	2.12	2.17	2.23	2.29	2.36	2.40	2.46	2.51	2.57
		0.8	2.11	2.21	2.32	2.42	2.52	2.62	2.71	2.80	2.89	2.98	3.06
		1.0	2.25	2.42	2.59	2.74	2.88	3.02	3.15	3.27	3.39	3.50	3.61
		1.2	2.44	2.69	2.91	3.11	3.29	3.46	3.62	3.78	3.93	4.07	4.20
	0.8	0.2	1.96	1.95	1.96	1.96	1.97	1.97	1.98	1.98	1.99	1.99	2.00
		0.4	1.99	2.01	2.03	2.05	2.08	2.10	2.13	2.15	2.18	2.21	2.23
		0.6	2.05	2.10	2.16	2.22	2.28	2.34	2.40	2.45	2.51	2.56	2.81
		0.8	2.15	2.26	2.37	2.47	2.57	2.67	2.76	2.85	2.94	3.02	3.10
		1.0	2.30	2.48	2.64	2.79	2.93	3.07	3.19	3.31	3.43	3.54	3.65
		1.2	2.50	2.74	2.96	3.15	3.33	3.50	3.66	3.81	3.96	4.10	4.23
	1.0	0.2	2.01	2.02	2.02	2.03	2.04	2.04	2.05	2.06	2.06	2.07	2.07
		0.4	2.05	2.08	2.10	2.13	2.16	2.18	2.21	2.23	2.26	2.28	2.31
		0.6	2.13	2.19	2.25	2.30	2.36	2.42	2.47	2.53	2.58	2.63	2.68
		0.8	2.24	2.35	2.45	2.55	2.65	2.74	2.83	2.92	3.00	3.08	3.16
		1.0	2.40	2.57	2.72	2.86	3.00	3.13	3.25	3.37	3.48	3.59	3.70
		1.2	2.60	2.83	3.03	3.22	3.39	3.56	3.71	3.86	4.01	4.14	4.28
	1.2	0.2	2.17	2.16	2.16	2.16	2.16	2.16	2.17	2.17	2.18	2.18	2.19
		0.4	2.22	2.22	2.24	2.26	2.28	2.30	2.32	2.34	2.36	2.39	2.41
		0.6	2.29	2.33	2.38	2.43	2.48	2.53	2.58	2.62	2.67	2.72	2.77
		0.8	2.41	2.49	2.58	2.67	2.75	2.84	2.92	3.00	3.08	3.16	3.23
		1.0	2.56	2.69	2.83	2.96	3.09	3.21	3.33	3.44	3.55	3.66	3.76
		1.2	2.74	2.94	3.13	3.30	3.47	3.63	3.78	3.92	4.06	4.20	4.33
	1.4	0.2	2.45	2.40	2.37	2.35	2.35	2.34	2.34	2.34	2.34	2.34	2.34
		0.4	2.48	2.45	2.44	2.44	2.45	2.46	2.48	2.49	2.51	2.53	2.55
		0.6	2.55	2.54	2.56	2.60	2.63	2.67	2.71	2.75	2.80	2.84	2.88
		0.8	2.64	2.68	2.74	2.81	2.89	2.96	3.04	3.11	3.18	3.25	3.33
		1.0	2.77	2.87	2.98	3.09	3.20	3.32	3.43	3.53	3.64	3.74	3.84
		1.2	2.94	3.09	3.26	3.41	3.57	3.72	3.86	4.00	4.13	4.26	4.39

Note: Values of factor μ_3 in this Table are calculated from the following equation:

$$\frac{\eta_1 K_1}{\eta_2 K_2} \cdot \operatorname{ctg}\frac{\pi\eta_1}{\mu_3} \cdot \operatorname{ctg}\frac{\pi\eta_2}{\mu_3} + \frac{\eta_1 K_1}{(\eta_2 K_2)^2} \cdot \operatorname{ctg}\frac{\pi\eta_1}{\mu_3} \cdot \operatorname{ctg}\frac{\pi}{\mu_3} + \frac{1}{\eta_2 K_2} \cdot \operatorname{ctg}\frac{\pi\eta_2}{\mu_3} \cdot \operatorname{ctg}\frac{\pi}{\mu_3} - 1 = 0$$

Appendix E Classification of members and connections for fatigue calculation

Table E Classification of members and connections

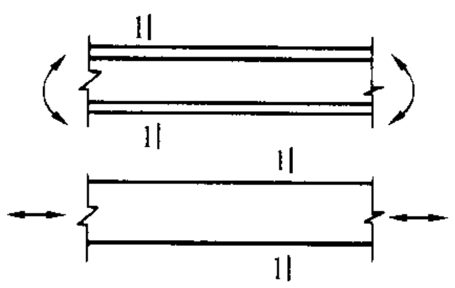
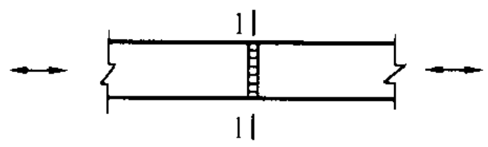
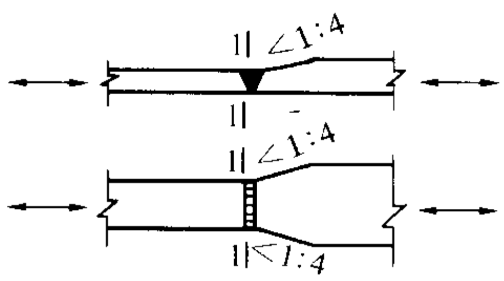
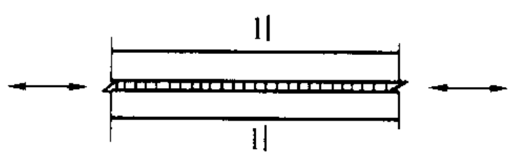
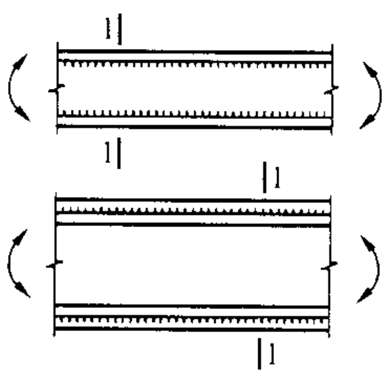
Item No.	Sketch	Description	Category
1		Base metal at locations without connection (1)Rolled shapes (2)Steel plates a. Both edges as rolled or planed b. Both edges auto- or semiauto-flame-cut (the cutting quality shall conform to the current national standard "Code for acceptance of construction quality of steel structures" GB 50205)	 1 1 2
2		Base metal adjacent to transverse butt weld: (1)Welds conforming to the first class standard given in "Code for acceptance of construction quality of steel structures" GB 50205 (2)Welds machined and ground flush and conforming to the first class standard	 3 2
3		Base metal adjacent to transverse butt weld connecting parts of different thicknesses (or widths), with weld machined and ground to a smooth transition and conforming to the first class standard	2
4		Base metal adjacent to longitudinal butt weld conforming to the second class standard	2
5		Base metal adjacent to flange connection weld (1) Flange-web connection weld a. Automatic second class combined butt T- and fillet weld b. Automatic fillet weld, appearance quality conforming to second class standard c. Manual fillet weld, appearance quality conforming to second class standard (2) Weld between twin flange plates a. Automatic fillet weld, appearance quality conforming to second class standard b. Manual fillet weld, appearance quality conforming to second class standard	 2 3 4 3 4

Table E (Continued)

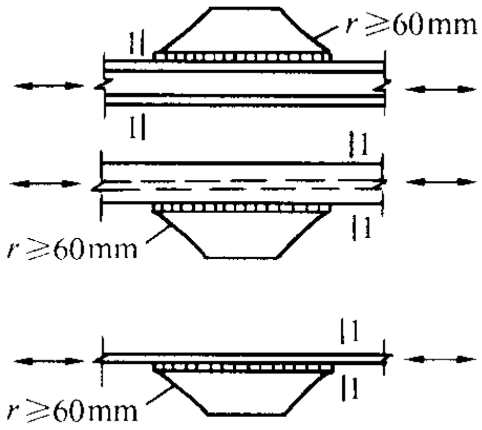
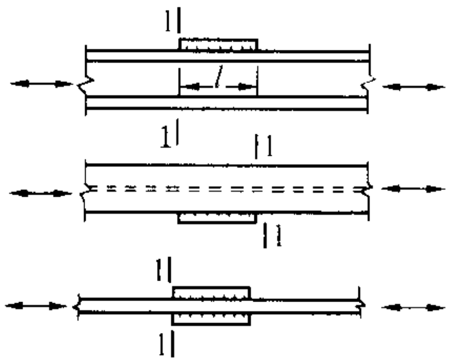
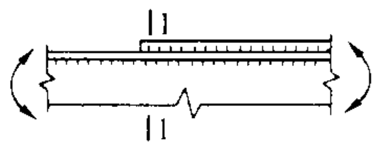
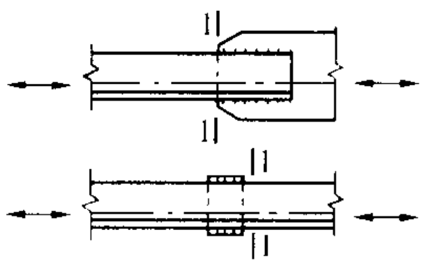
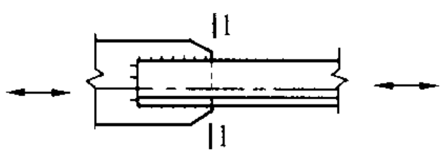
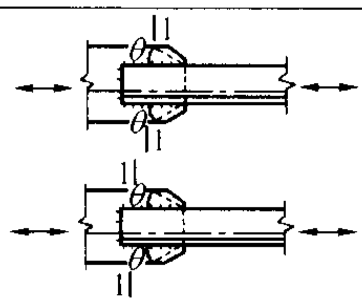
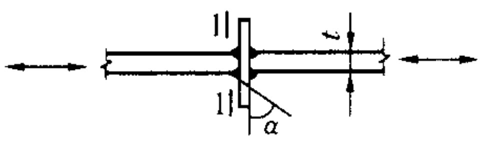
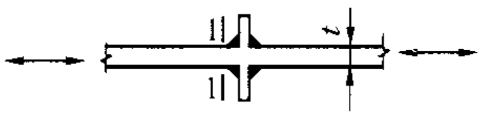
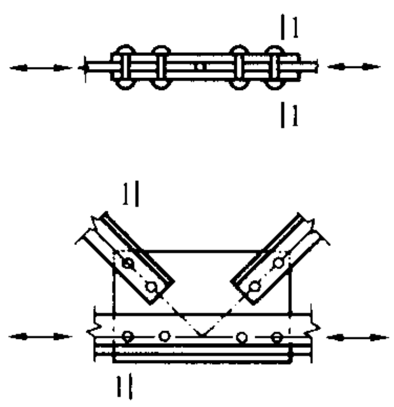
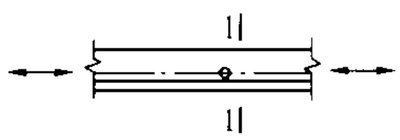
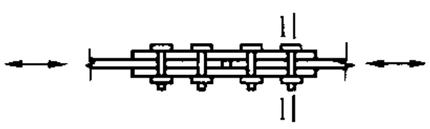
Item No.	Sketch	Description	Category
6		Base matal adjacent to the end of the transverse stiffeners (1)The weld-run does not stop at the end of the stiffener (return welding adopted) (2)The weld-run stops at the end of the stiffener	 4 5
7		Base metal at locations where a trapezoidal gusset plate is welded to beam flange, beam web and truss members with butt welds and, after welding, the transition is machined flush and ground to a smooth curve, where no defects due to start-stop of weld-run shall exist	5
8		Base metal at location where a rectangular gusset plate is welded to beam flange or web, $l>150$mm	7
9		Base metal of beam flange at the end of cover plate (with transverse fillet weld across the plate end)	7
10		Base metal at the transition region to the transverse fillet weld	6
11		Base metal at the end of welded connection with two side fillet welds	8
12		Base metal at the end of welded connection with 2 side and 1 transverse fillet welds	7
13		Base metal of gusset plate connected with 2 side fillet welds, or 2 side and 1 transverse fillet weld (the effective width of gusset plate is calculated according to the stress dispersal angle equal to 30°)	7

Table E (Continued)

Item No.	Sketch	Description	Category
14		Base metal at location of K-shape groove combined butt T-and fillet welds, misalignment of two plate axes being$<0.15t$, second class welds and weld toe angle $\alpha \leqslant 45°$	5
15		Base metal of cruciform joints at location of the fillet welds, misalignment of two plate axes being$<0.15t$	7
16	Fillet weld	Checked by the shear stress range determined with the effective area of fillet welds	8
17		Base metal at location of riveted connections	3
18		Base metal at locations of tie bolts and void holes	3
19		Base metal at location of high-strength bolted friction type connections	2

Note: 1 All butt welds as well as combined butt T-and fillet welds must be fully penetrated. The outside dimensions of all welds must conform to the requirements of the current standard "Weld outer dimensions of steel construction" JB 7949.

2 Fillet welds shall comply with the requirements in Clauses 8.2.7 and 8.2.8.

3 In Item No. 16, the shear stress range $\Delta\tau = \tau_{max} - \tau_{min}$, where τ_{min} is to be taken as positive when it is in the same direction as τ_{max}, and negative when it is in the opposite direction of τ_{max}.

4 Stresses in Items 17, 18 shall be computed on net cross-sectional area, while in Item 19 on gross sectional area.

Appendix F Stability calculation of truss gusset plate subject to compressive force of the diagonal web member

F.0.1 Basic assumptions

1 In Fig.F.0.1, the *B-A-C-D* is the buckling line when the gusset plate is instable, where $\overline{BA}$ is parallel to the chord member and $\overline{CD} \perp \overline{BA}$.

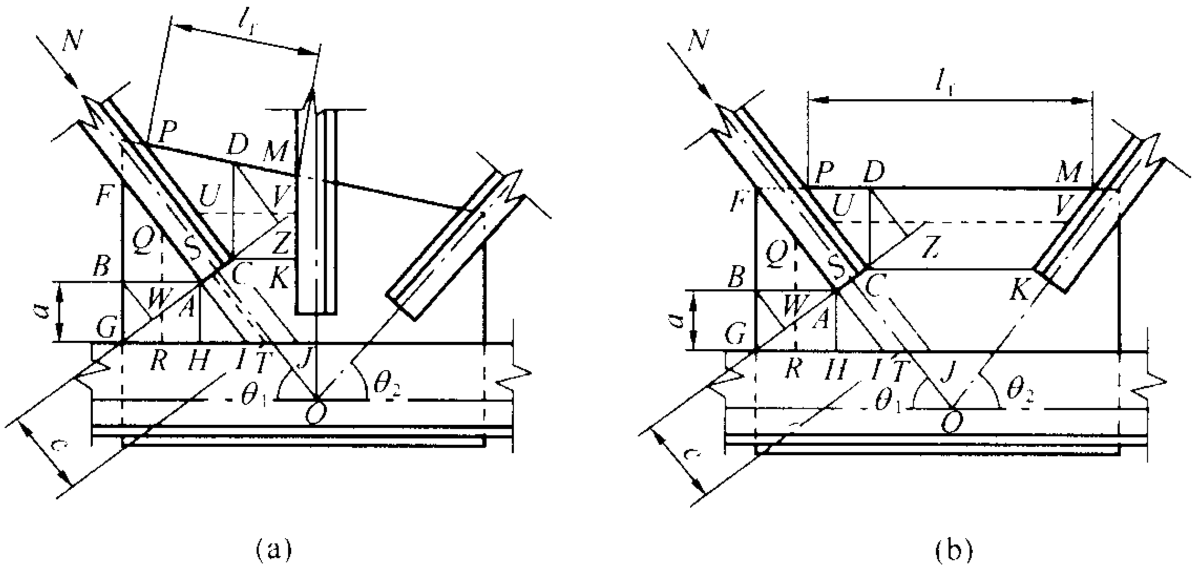


Fig. F.0.1 sketch for calculating stability of the gusset plate

(a) With vertical member; (b) Without vertical member

2 Under the action of compressive force N from diagonal web member, $\overline{BA}$ zone (plate *FBGHA*), $\overline{AC}$ zone (plate *AIJC*) and $\overline{CD}$ zone (plate *CKMP*) are in compression simultaneously. After one of the zones buckled, the other zones will be buckled subsequently, so that the stability of each zone shall be calculated individually.

F.0.2 Calculation method:

$\overline{BA}$ zone:

$$\frac{b_1}{(b_1 + b_2 + b_3)} N \sin\theta_1 \leqslant l_1 t \varphi_1 f \tag{F.0.2-1}$$

$\overline{AC}$ zone:

$$\frac{b_2}{(b_1 + b_2 + b_3)} N \leqslant l_2 t \varphi_2 f \tag{F.0.2-2}$$

$\overline{CD}$ zone:

$$\frac{b_3}{(b_1 + b_2 + b_3)} N \cos\theta_1 \leqslant l_3 t \varphi_3 f \tag{F.0.2-3}$$

where t——thickness of the gusset plate;

N——axial force of the diagonal web member in compression;

l_1, l_2, l_3——length of buckling lines $\overline{BA}$, $\overline{AC}$, $\overline{CD}$ respectively;

$\varphi_1, \varphi_2, \varphi_3$——stability factors of each plate zone in axial compression, can be taken as

Class b section; The corresponding slenderness ratio are: $\lambda_1 = 2.77\frac{\overline{QR}}{t}$, $\lambda_2 = 2.77\frac{\overline{ST}}{t}$, $\lambda_3 = 2.77\frac{\overline{UV}}{t}$, where $\overline{QR}$, $\overline{ST}$, $\overline{UV}$ are the length of mid lines of three plate zones $\overline{BA}$, $\overline{AC}$, $\overline{CD}$ in compression, and $\overline{ST} = c$; $b_1(\overline{WA})$, $b_2(\overline{AC})$, $b_3(\overline{CZ})$ are the length of each buckled lines projected on the lines of effective width. When a gusset plate with $l_f/t > 60\sqrt{235/f_y}$ (l_f is the free edge length of the gusset plate) has the free edge stiffened, and no vertical web member at the joint, the above calculation method can be applied, checking $\overline{BA}$ zone and $\overline{AC}$ zone only, but $\overline{CD}$ zone need not be checked.

Explanation of wording in this Code

1. In order to treat different situations according to their individual conditions during the implementation of this code, words denoting the different degrees of strictness of demands are explained as follows:

1) Words denoting a very strict or mandatory requirement:

"must" is used for affirmation, "must not" is used for negation.

2) Words denoting a strict requirement under normal conditions:

"shall" is used for affirmation, "shall not" is used for negation.

3) Words denoting a permission of slight choice or an indication of the most suitable choice when conditions allow:

"should" or "may" are used for affirmation, "should not" is used for negation.

2. "shall be in accordance with" or "shall comply with" is used to indicate that it is necessary to implement items in this code according to designated standards, codes or other relative regulations.