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GB 50018—2002

# Technical Code of Cold-formed Thin-wall Steel Structures

# 冷弯薄壁型钢结构技术规范

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**GB 50018—2002**

Chief Development Department: Development planning committee of Hubei province

Approval Department: Ministry of Construction of the People's Republic of China

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# Announcement of Ministry of Construction of the People's Republic of China

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## Announcement of Publishing for the National Standard of "Technical Code of Cold-formed Thin-walled Steel Structures"

"Technical Code of Cold-formed Thin-walled Steel Structures" has been approved as a national standard with a serial number of GB 50018-2002, and it shall be come into force on January 1, 2003. Herein, Clauses 3.0.6、4.1.3、4.1.7、4.2.1、4.2.3、4.2.4、4.2.5、4.2.7、9.2.2、10.2.3 are mandatory clauses, which must be enforced strictly. The original "Technical Code of Cold-formed Thin-walled Steel Structures" GBJ 18—87 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

September 27, 2002

# Foreword

According to the requirement of the document Jian Biao[1998] No. 94 of the Ministry of Construction, the chief editorial department-Development Planning Committee of Hubei Province, the chief editorial unit-Central South Architectural Design Institute, together with relevant units, proceeded a comprehensive revision of "Technical Code of Cold-formed Thin-wall Steel Structures" GBJ 18—87 which was approved to execute by the former National Planning Committee in 1987.

There are 11 chapters and 5 appendixes in this code. The major amendments of this Revision are as follows:

1. According to the new edition of National Standard "Unified Standard for Reliability Design of Building Structures", the following content has been supplemented: When determining the safety grades, the designed working life should be combined to consider.

2. The following design rule has been supplemented: When designing the single storey buildings, the stressed skin action may be considered.

3. The formulae for checking the stability of beam-columns having singly symmetric open section bending in the non-symmetric plane have been replenished.

4. The calculation of effective width-to-thickness ratio about three different sorts of compressed elements has been amended to be as follows: the unified calculation formula is used while taking the plates assembly as a calculation unit, considering the restraint effect of the adjacent elements.

5. The new types of connection such as self-tapping(self-drill) screw, blind rivet, cartridge fired pins and flare groove weld have been supplemented.

6. The calculation method of the profiled steel sheeting which had been widely used as non-combined slab with the combined action of moment and shear force have been supplemented.

7. The design rules and detailing requirements of thin-walled steel side rails which has been applied very widely have been supplemented.

8. The calculation formula of effective length of the columns in multi-span gabled frame system and the regulation about the deflection of the beams and lateral drift on the top of the columns in gabled frame have been supplemented.

The clauses in the code may be partially revised, the information about the revision and the revised clauses will publish in the magazine "Standardization of Engineering Construction".

The 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 code, while Central South Architectural Design Institute will be responsible for explanation of concrete contents.

In order to improve the quality of the code, users are solicited to sum up experience in the course of code implementation. Comments and suggestions on this code are requested to send to the Administrative Group of the National Standard "Technical Code of Cold-formed Thin-walled Steel Structures", Central South Architectural Design Institute(Address: 10 Zhongnan Er Road, Wuchang, Wuhan, Hubei Province, Post code: 430071, E-mail: lwssc@public. wh. hb. cn).

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# Contents


**1 General Principles** ····· ( 1 )
**2 Terms and Symbols** ····· ( 2 )
2. 1 Terms ····· ( 2 )
2. 2 Symbols ····· ( 3 )
**3 Materials** ····· ( 6 )
**4 Basic Design Stipulations** ····· ( 8 )
4. 1 Design Principles ····· ( 8 )
4. 2 Design Indices ····· ( 9 )
4. 3 General Stipulations of Detailing requirements ····· (11)
**5 Calculation of Members** ····· (13)
5. 1 Axial Tension Members ····· (13)
5. 2 Axial Compression Members ····· (13)
5. 3 Flexural Members ····· (16)
5. 4 Members Subjected to Combined Axial Tension and Bending ····· (19)
5. 5 Members Subjected to Combined Axial Compression and Bending ····· (19)
5. 6 Compressive Elements in Members ····· (23)
**6 Calculation and Detailing Requirements of Connections** ····· (28)
6. 1 Calculation of Connections ····· (28)
6. 2 Detailing Requirements of Connections ····· (35)
**7 Profiled Steel Sheeting** ····· (38)
7. 1 Calculation of Profiled Steel Sheeting ····· (38)
7. 2 Detailing requirements of Profiled Steel Sheeting ····· (41)
**8 Purlins and Side Rails** ····· (43)
8. 1 Calculation of Purlins ····· (43)
8. 2 Detailing of Purlins ····· (44)
8. 3 Calculation of Side Rails ····· (45)
8. 4 Detailing of Side Rails ····· (46)
**9 Roof Truss** ····· (47)
9. 1 Calculations of Roof truss ····· (47)
9. 2 Detailing requirements of Roof Truss ····· (48)
**10 Rigid Frame** ····· (49)
10. 1 Calculations of Rigid Frame ····· (49)

10.2 Detailing Requirements of Rigid Frame ..... (52)
**11 Fabrication, Erection and Anticorrosion** ..... (54)
11.1 Fabrication and Erection ..... (54)
11.2 Anticorrosion ..... (57)
**Appendix A Coefficients of Calculation** ..... (60)
**Appendix B Section Characteristics** ..... (67)
**Appendix C Calculation Method of Design Values of Strength Which Considering the Effect of Cold Forming** ..... (83)
**Appendix D Classification of Corrosive Effcct , Primer and Compatible Finish Coating, and Duration Cimit of Maintenance** ..... (84)
**Explanations of Wording in This Code** ..... (86)

# 1 General Principles

**1.0.1** This code intends to implement the technical and economical policies of the State in design and construction of cold-formed thin-walled steel structure, by using advanced technology and ensuring economy reasonableness, safety, usability for use and good quality of the structure.

**1.0.2** This code applies to the design and construction of cold-formed thin-walled steel structures for building engineering.

**1.0.3** This code does not cover the special requirements of cold-formed thin-walled steel structure which bearing dynamic loading directly and undergoing intense action of corrosion.

**1.0.4** The design principles of this code are based on the current National Standard "Unified Standard for Reliability Design of Building Structures" GB 50068.

**1.0.5** In designing cold-formed thin-walled 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.

**1.0.6** The design and construction of cold-formed thin-walled steel structure shall comply with this code, shall furthermore comply with the relevant current national codes.

# 2 Terms and Symbols

## 2.1 Terms

**2.1.1** Elements

The flatplate part between two adjacent longitudinal edges in thin-walled steel member.

**2.1.2** Stiffened elements

The elements, which both of the two longitudinal edges connect with other elements.

**2.1.3** Partially stiffened elements

The elements, which one longitudinal edge connects with other element, and another is stiffened by the border lip according with the requirements.

**2.1.4** Unstiffened elements

The elements, which one longitudinal edge connects with other element, and another is free.

**2.1.5** Uniformly compressed elements

The elements, subject to the uniform compression.

**2.1.6** Non-uniformly compressed elements

The elements, subject to the non-uniform compression which is linear distributed.

**2.1.7** Sub-elements

The elements, which one longitudinal edge connects with other element, and another is connected with intermediate stiffener complied the requirements, or which two longitudinal edges are both connected with intermediate stiffeners complied the requirements.

**2.1.8** Width-to-thickness ratio

The ratio of the width to the thickness of elements.

**2.1.9** Effective width-to-thickness ratio

To simplify calculation, the width of compressed elements is reduced for considering its post-bucking strength . The ratio of the reduced design width to the thickness of elements is fermed the effective width-to-thickness ratio.

**2.1.10** Effect of cold forming

The effect of characteristics of steel are changed due to cold forming.

**2.1.11** Stressed skin action

The profiled steel sheeting system which is connected reliably with supporting

members has the capability to resist shear deformation in the plane itself.

**2.1.12** Flare groove welds

The welded which connect fillet to fillet or plate.

## 2.2 Symbols

**2.2.1** Actions and effects of actions

$B$——Bimoment;

$F$——Concentrated load;

$M$——Bending moment;

$N$——Axial force;

$N_t$——Tension force tansmitted by each fastener;

$N_v$——Shear force transmitted by each fastener;

$P$——Pretension force of a high-strength bolt;

$V$——Shear force.

**2.2.2** Calculation indices

$E$——Modulus of elasticity of steel;

$G$——Shearing modulus of steel;

$N_v^s$——Design value of shear capacity of each welding spot in resistance spot welding;

$N_t^b$——Design value of tensile capacity per bolt;

$N_v^b$——Design value of shear capacity per bolt;

$N_c^b$——Design value of compressive capacity per bolt;

$N_t^f$——Design value of tensile capacity per tapping screw or cartridge-fired pin;

$N_v^f$——Design value of tensile capacity per connector;

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

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

$f_v$——Design value of shear strength of steel;

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

$f_c^b$, $f_t^b$, $f_v^b$——Design value of compressive, tensile and shear strength of bolts respectively;

$f_c^w$, $f_t^w$, $f_v^w$——Design value of compressive, tensile and shear strength of butt welds respectively;

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

$\sigma$——Normal stress;

$\tau$——Shear stress.

**2.2.3** Geometric parameters

$A$——Gross area of cross-section;

$A_n$——Net area of cross-section;

$A_e$——Effective sectional area;

$A_{en}$——Effective net sectional area;

$H$——Column height;

$H_0$——Effective column height;

$I$——Moment of inertia of gross section;

$I_n$——Moment of inertia of net section;

$I_t$——Torsional moment of inertia (St. Venant torsion constant) of the gross section;

$I\omega$——Sectorial moment of inertia of the gross section;

$I_{es}$——Moment of inertia of the edge stiffener of profiled steel sheeting;

$I_{is}$——Moment of inertia of the intermediate stiffener of profiled steel sheeting;

$S$——First moment of gross sectional area;

$W$——Gross section modulus;

$W_n$——Net section modulus;

$W_\omega$——Sectorial modulus of gross section;

$W_e$——Effective section modulus;

$W_{en}$——Effective net section modulus;

$a$——Height of lip; panel length of the top chord in a laced purlin; spacing of connector;

$a_{max}$——The maximum allowable spacing of connector;

$b$——Width of section or element;

$b_0$——Effective width (or height) of section;

$b_s$——Width of sub-element in profiled steel sheeting;

$b_e$——Effective width of element;

$c$——Width of element adjacent to the calculated element;

$d$——Diameter;

$d_0$——Diameter of hole in member;

$d_e$——Effective diameter of screw thread on bolt;

$e$——Eccentricity;

$e_a$——Distance between loading point and shear center;

$e_0$——Abscissa of the sectional shear center (the geometric center is considered as base point) on axis of symmetry;

$e_x$——Equivalent eccentricity;

$h$——Height of section of element;

$h_0$——Effective height of web;

$h_f$——Size of fillet weld leg;

$i$——Radius of gyration;

$l$——Length or span; spacing of lateral supports; midliue length of steel section;

$l_w$——Effective length of fillet weld;

$l_0$——Effective length;

$l_\omega$——Effective length for torsional buckling;

$r_i$——Inside bend radins of ith rounded corner for the steel section;

$t$——Thickness;

$\theta$——Angle;

$\lambda$——Slenderness ratio;

$\lambda_0$——Equivalent slenderness ratio;

$\lambda_\omega$——Equivalent slenderness ratio for torsional-flexural buckling;

**2.2.4** Coefficients of calculation

$k$——Stability coefficient of compressed element;

$k_1$——Restraint coefficient of element assemble;

$n$——Number of bolt at a joint; total number of panels of adjacent two lateral supports;

$n_e$——Number of panels with interal force being compression;

$n_v$——Number of shear planes of each bolt;

$n_1$——Number of connecters at the same section;

$\alpha$, $\beta$——Restraint coefficients of member;

$\beta_m$——Factor of equivalent moment;

$\gamma$——Ratio of the tensile strength to the yield strength of steel;

$\gamma_R$——Partial safety factor for strength;

$\xi_1$, $\xi_2$——Parameter for checking overall stability of theflexural members;

$\eta$——Parameter for checking overall stability of the flexural members; coefficient used in calculating the design value of strength when considering the effect of cold forming; sectional coefficient;

$\zeta$——Parameter for checking overall stability of the flexural members ;

$\mu$——Effective length factor of the column of gabled frames;

$\mu_b$——Lateral effective length factor of the girder;

$\rho$——Mass density; coefficient for calculating the effective width-to- thickness ratio of compression elements;

$\varphi$——Stability factor of axial compression member;

$\varphi_b$, $\varphi'_b$——Overall stability factor of the flexural members;

$\psi$——Nonuniformily distributed coefficient of compressive stress.

# 3 Materials

**3.0.1** Steel strip or steel plate used for load-carrying cold-formed thin-walled steel structures, shall be of steel grade Q235 which conform to the requirements of the current national standard "Carbon Structural Steel" GB/T 700 and steel grade Q345 which conform to the requirements of the current national standard "High Strength Low-alloy Structural Steel" GB/T 1591. When there are reliable basis, the steel steel of other grades may be adopted and shall also conform the requirements of the relevant national standards.

**3.0.2** Steel strip or steel plate used for load-carrying cold-formed thin-walled steel structures, shall meet the requirements of tensile strength, percentage of elongation, yield strength (or yield point), cold-bending test as well as proper sulfur and phosphorus contents; for welded structures the steel shall also be guaranteed for proper carbon content.

**3.0.3** Different grades of steel may be used in one member if technique and economy are reasonable.

**3.0.4** The welding materials 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 Electrode" GB/T 5117 or "Low-alloy Steel Covered Electrode" GB/T 5118. The type of selected electrodes shall match the base metal in mechanical properties.

**2** The wire used for automatic or semi-automatic welding shall meet the requirements of the current national standard "Steel Wire for Melt Welding" GB/T 14957. The selected wire and flux shall match the base metal.

**3** The wire used for carbon dioxide gas shielding welding, shall meet the requirements of the current national standard "Welding Wire for Gas Shielding Arc Welding of Carbon and Low-alloy Steel" GB/T 8810.

**4** When welding steel of grade Q235 with steel of grade Q345, it's recommended to use welding electrode or wire which matches the steel of grade Q235.

**3.0.5** Connector (connecting materials) shall comply with the following requirements:

**1** Ordinary bolt shall meet the requirements of the current national standard "Hexagon Head Bolts-product Grade C" GB/T 5780, Its mechanical properties shall meet with the requirements of the current national standard "Mechanical Properties of

Fasteners-Bolts, Screws and Studs" GB/T 3098. 1.

**2** High-strength bolts shall meet the requirements of the current national standard "High Strength Bolts with Large Hexagon Head, Large Hexagon Nut, Plain Washer for Steel Structure and Specification of High Strength Bolts with Large Hexagon Head, Large Hexagon Nut, Plain Washer for Steel Structure" GB/T 1228～1231 or "Set of Torshear Type High Strength Bolt, Hexagon Nut and Plain Washer for Steel Structure, Technical Requirements for Set of Torshear Type High Strength Bolt, Hexagon Nut and Plain Washer for Steel Structure" GB/T 3632～3633.

**3** The self-tapping screw used for connecting thin steel plates or other metal plates shall meet the requirements of the current national standard "Drilling Screw with Tapping Screw Thread" GB/T 15856. 1～4, GB/T 3098. 11 or "Tapping Screw" GB/T 5282～5285.

**3. 0. 6 In the design documents and material ordering documents of cold-formed thin-walled steel structure, it shall be indicated the grade, the quality level, the supplying conditions of steel etc. , as well as the category (or steel grade) of connecting materials. If necessary, it shall be indicated additional guarantee items of the the mechanical properties and chemical composition for the steel.**

# 4 Basic Design Stipulations

## 4.1 Design Principles

**4.1.1** For calculation, the limit state design method based on probabilistic theory is adopted, using design expressions with partial safety factors.

**4.1.2** Load-carrying structures of cold-formed thin-walled steel shall be designed according to ultimate limit states and serviceability limit states.

**4.1.3 The importance factor $\gamma_0$ in designing cold-formed thin-walled steel structures shall be determined according to the safety class, the design working life of structure.**

**Cold-formed thin-walled steel structures of industrial and civil buildings, in general, shall be taken as safety class 2, its importance factor shall not be less than 1.0 when design working life is 50 years; and the importance factor shall not be less than 0.95 when design working life is 25 years. Whereas for special cold-formed thin-walled steel structures of buildings, the safety class and the design working life shall be dealt with individually.**

**4.1.4** In designing cold-formed thin-walled steel structures according to 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, the design value of loads and design value of strengths shall be used for the calculation. The design value of load equals the characteristic value of load multiplied by partial safety factor for loads; the design value of strength equals the characteristic value of strength of materials divided by partial safety factor for strength. For cold-formed thin-walled steel structures, partial safety factor for strength shall be taken as $\gamma_R = 1.165$.

**4.1.5** In designing cold-formed thin-walled steel structures according to serviceability limit state, the normal combination of load effects shall be considered, the characteristic value of loads and allowable value of deformation shall be used for the calculation.

**4.1.6** In calculating structural members and connections, the characteristic value of loads, partial safety factor for loads, the load effect combination and the load combination coefficient, shall comply with requirements of current national standard "Load Code for the Design of Building Structures"GB 50009.

Note: For members or structures supporting light roofing (roof truss, frame, etc.), the characteristic value of uniform roof live load may be taken as 0.3kN/m$^2$ when only one variable load is acting on and that the horizontal projection of loaded area of the building exceeds 60m$^2$.

**4.1.7 In designing gable frames, roof trusses, purlins and side rails, the unfavorable effects of changing of internal force in members caused by the wind suction loads shall be considered, in this case the partial safety factor for dead loads shall be taken as 1.0.**

**4.1.8** The tensile strength of structure members shall be calculated according to the net section; the compressive strength shall be calculated according to the effective net section; the stability shall be calculated according to the effective section.

**4.1.9** The deformation of members and all kinds of stability factors may be calculated according to the gross section.

**4.1.10** When the profiled steel sheeting each other and with their supporting members are fastened by connectors which are unable to slide, to form the encosure diaphragm systems such as roof and wall etc., the stressed skin action may be considered in design of single storey buildings, but in this case the following requirements shall be all complied with:

1 The parameters of strength and stiffness for diaphragm a ssemblies shall be determined by testing or reliable analysis, the analysis and design of the building shall be conclucted by considering the complete;

2 In case of check for the carrying capacity of the purlins, side rails, studs and the connectors cocated at key positions such as ridge, eaves and end ealls, besides to consider the internal forces caused by direct action of loads on them, the additional internal forces calculated from the complete analysis shall also be considered;

3 The permanent signboard must be put on notable place of the built up building, which shall indicate that any removal of the profiled steel sheeting is prohibited unless other temporary bracing is provided during the utilization and maintenance of the building. The above notice shall also be specified in the design documents.

## 4.2 Design Indices

**4.2.1 The design value of steel strength shall be taken from Table 4.2.1.**

**Table 4.2.1 Design Value of Steel Strength ($N/mm^2$)**

| Steel grade | Tension, compression and bending $f$ | Shear $f_v$ | End bearing ( polished and closely fitted ) $f_{ce}$ |
|---|---|---|---|
| Q235 steel | 205 | 120 | 310 |
| Q345 steel | 300 | 175 | 400 |

**4.2.2** The strength of members which the section is fully effective and subjected to tension, compression or bending, may take the design value of strength considering the effect of cold forming which is calculated according to Appendix C of this code.

**4.2.3 The design value of strenth considering the effect of cold forming shall not be table account for design cold-formed thin-walled steel members, which had been hot treatment such as annealing, welding and hot galvanizing.**

**4.2.4 The design value of weld strength shall be taken from Table 4.2.4.**

Table 4. 2. 4 Design Value of Weld Strength (N/mm²)

| Steel grade of member | Butt weld | | | Fillet weld |
|---|---|---|---|---|
| | Compression $f_c^w$ | Tension $f_t^w$ | Shear $f_v^w$ | Compression, tension and shear $f_t^w$ |
| Q235 steel | 205 | 175 | 120 | 140 |
| Q345 steel | 300 | 255 | 175 | 195 |

Note: 1 When welding Q235 Steel and Q345 Steel, the design value of weld strength shall take the values at the row of Q235 Steel in Table 4. 2. 4.

2 The design value of tensile strength of butt weld, which are examined by X ray and accord with weld quality of grade One or Two, shall take the design value of compressive strength.

**4. 2. 5 The design value of bolted connection strength of ordinary bolts of grade C shall be taken from Table 4. 2. 5.**

Table 4. 2. 5 Design Value of Bolted Connection Strength of Ordinary Bolts of Grade C (N/mm²)

| Category | Property grade | Steel grade of member | |
|---|---|---|---|
| | Grade4. 6, Grade 4. 8 | Q235 Steel | Q345 Steel |
| Tension strength $f_t^b$ | 165 | — | — |
| Shear strength $f_v^b$ | 125 | — | — |
| Compression strength $f_c^b$ | — | 290 | 370 |

**4. 2. 6** The design value of shear capacity of each welding spot in resistance spot welding shall be taken from Table 4. 2. 6.

Table 4. 2. 6 Design Value of Shear Capacity of Resistance Spot Welding

| Thickness of the outer thinner element of the welding elements $t$ (mm) | Design value of shear capacity of each welding spot $N_v^s$ (kN) | Thickness of the outer thinner element of the welding elements $t$ (mm) | Design value of shear capacity of each welding spot $N_v^s$ (kN) |
|---|---|---|---|
| 0. 4 | 0. 6 | 2. 0 | 5. 9 |
| 0. 6 | 1. 1 | 2. 5 | 8. 0 |
| 0. 8 | 1. 7 | 3. 0 | 10. 2 |
| 1. 0 | 2. 3 | 3. 5 | 12. 6 |
| 1. 5 | 4. 0 | — | — |

**4. 2. 7 The design value of strength specified in Clauses from 4. 2. 1 to 4. 2. 6 in this code shall be multiplied by a relevant reduction factor in the following situations of structural member and connection calculation:**

**1 Main compressive web member on the ends of plane latticed purlin: 0. 85;**

**2 Single angle steel member connected by one leg:**

**1) For checking member and connection strength as axially loaded: 0. 85;**

**2) For checking stability as an axial compression member: $0.6+0.0014\lambda$.**

Note: For the single angle compressive steel member without intermediate brace, $\lambda$ is slenderness ratio of member determined by the least radius of gyration.

**3 Butt weld performed by welding from one side without backing plate: 0.85;**

**4 Welded erection connections made high above the ground in unfavorable conditions: 0.90;**

**5 Two members' connection which is lap jointed or the connection with backing plate between them or nonsymmetrical connection with single cover plate: 0.90.**

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

**4.2.8** The physical properties of steel shall be according to the requirements of Table 4.2.8.

**Table 4.2.8 Physical Properties of Steel**

| Modulus of elasticity $E$ ($N/mm^2$) | Shear deformation modulus $G$ ($N/mm^2$) | Coefficient of linear expansion $\alpha$ ( per ℃ ) | Density $\rho$ ($kg/m^3$) |
|---|---|---|---|
| 206×103 | 79×103 | $12\times10^{-6}$ | 7850 |

## 4.3 General Stipulations of Detailing Requirements

**4.3.1** The wall thickness of cold-formed thin-wall steel structures should not be thicker than 6mm, and not be thinner than 1.5mm (except the profiled steel sheeting). The wall thickness of members of main load-carrying structure should not be thinner than 2mm.

**4.3.2** The wall thickness of the compressive parts of members shall meet the following requirements:

**1** The maximum width-to-thickness ratio of the compressive elements in members shall conform the requirements in Table 4.3.2;

**Table 4.3.2 Limited Values of Width-to-thickness Ratio of Compression Element**

| Type of element \ Steel grade | Q235 steel | Q345 steel |
|---|---|---|
| Unstiffened element | 45 | 35 |
| Partially stiffened element | 60 | 50 |
| Stiffened element | 250 | 200 |

**2** The ratio of the outer diameter to the wall thickness of members of circular tube section, should not be greater than 100 for Q235 steel, and not be greater than 68 for Q345 steel.

**4.3.3** The slenderness ratio of members shall meet the following requirements:

**1** The slenderness ratio of compression members should not exceed the values listed in Table 4.3.3.

Table 4.3.3 Allowable Values of Slenderness Ratio for Compression Members

| Item No. | Type of members | Allowable slenderness ratio |
|---|---|---|
| 1 | Main member (such as main load-carrying column, gabled frame column, chord and end strut oftruss and latticed gabled frame, etc.) | 150 |
| 2 | Other members and bracings | 200 |

2 The slenderness ratio of tension members should not exceed 350, but slenderness ratio of pretensioned round bars is not limited. When tension members change into compression ones under the combined action of dead and wind loads, their slenderness ratio should not exceed 250; and under the action of crane load, their slenderness ratio should not exceed 200.

**4.3.4** For latticed columns connected by battened plate or lace bar, the transverse diaphragms should be provided, the spaces should not be larger than 2~3m, and at the ends of each transportation unit the transverse diaphragms shall be provided. At the two ends of solid-web beam and beam-column and at the acting points of larger concentrated loads, the transverse stiffener shall be provided, if the ratio of height-to-thickness of solid-web member is comparatively large, the transverse stiffener should also be provided for detailing requirement.

# 5 Calculation of Members

## 5.1 Axial Tension Members

**5.1.1** The strength of members subject to axial tension shall be checked as follows:

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

Where $\sigma$——Normal stress;

$N$——Axial force;

$A_n$——Net area of cross-section;

$f$——Design value of tensile, compressive and bending strength of steel.

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

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

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

Where $n_1$——Number of high-strength bolts on the calculated section (outmost line of bolts);

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

$A$——Gross area of cross-section.

**5.1.2** In calculating the strength of axial tension members with open section, account shall be taken of the effect of bimoment due to the axial load not passing through the shear center of cross-section (or not passing through the sectorial zero of $Z$-section).

Note: This provision also applies to the axial compression members and members subjected to combined axial load and bending.

## 5.2 Axial Compression Members

**5.2.1** The strength of members subjected to axial compression shall be checked as follows:

$$\sigma = \frac{N}{A_{en}} \leqslant f \tag{5.2.1}$$

Where $A_{en}$——Effective net sectional area.

**5.2.2** The stability of axial compression members shall be checked as follows:

$$\frac{N}{\varphi A_e} \leqslant f \tag{5.2.2}$$

Where $\varphi$——Stability factor of axial compression members, which shall be taken from Table A. 1. 1-1 or Table A. 1. 1-2, as applicable;

$A_e$——Effective sectional area.

**5. 2. 3** For axial compression members of closed section, doubly symmetric open section and single equal-leg angle section being fully effective, the stability factor shall be determined according to the larger of $\lambda_x$ and $\lambda_y$ calculated by following equations:

$$\lambda_x = \frac{l_{0x}}{i_x} \tag{5. 2. 3-1}$$

$$\lambda_y = \frac{l_{0y}}{i_y} \tag{5. 2. 3-2}$$

Where $\lambda_x$、$\lambda_y$——Slenderness ratio of members about the principal axis $x$ and $y$ respectively;

$l_{0x}$、$l_{0y}$——Effective length of members about the principal axis $x$ and $y$ respectively;

$i_x$、$i_y$——Radius of gyration of gross cross-section of members about the principal axis $x$ and $y$ respectively.

**5. 2. 4** For axial compression members of singly-symmetric open section (as shown in Fig. 5. 2. 4), the stability factor shall be determined in accordance with the larger of $\lambda_y$ given in Eq. 5. 2. 3-2 and $\lambda_\omega$ calculated as follows:

$$\lambda_\omega = \lambda_x \sqrt{\frac{s^2 + i_0^2}{2s^2}\sqrt{\left(\frac{s^2 + i_0^2}{2s^2}\right) - \frac{i_0^2 - \alpha e_0^2}{s^2}}} \tag{5. 2. 4-1}$$

With

$$s^2 = \frac{\lambda_x^2}{A}\left(\frac{I_\omega}{l_\omega^2} + 0.039 I_t\right) \tag{5. 2. 4-2}$$

$$i_0^2 = e_0^2 + i_x^2 + i_y^2 \tag{5. 2. 4-3}$$

Where $\lambda_\omega$——Equivalent slenderness ratio for torsional-flexural buckling of the member;

$I_\omega$——Sectorial moment of inertia of the gross section;

$I_t$——Torsional moment of inertia of the gross section;

$e_0$——Distance from the shear center to the centroid along the axis of symmetry, taken as negative;

$l_\omega$——Effective length for torsional buckling, $l_\omega = \beta \cdot l$;

$l$——Effective length, taken as either the geometric length of unbattened members or the largest spacing between centers of adjacent battens for battened members;

$\alpha$, $\beta$——Restraint coefficients given in Table 5. 2. 4.

**Table 5. 2. 4 Restraint Coefficients of Open Section Members Subjected to Axial Compression or to Combined Axial Compression and Bending**

| Item No. | Supporting conditions at ends of members | Without battens | | With battens | |
|---|---|---|---|---|---|
| | | $\alpha$ | $\beta$ | $\alpha$ | $\beta$ |
| 1 | Both ends hinged, end sections free to warp | 1. 00 | 1. 00 | — | — |
| 2 | Both ends fixed | 1. 00 | 0. 50 | 0. 80 | 1. 00 |
| 3 | Both ends hinged, but end warping fully restrained | 0. 72 | 0. 50 | 0. 80 | 1. 00 |

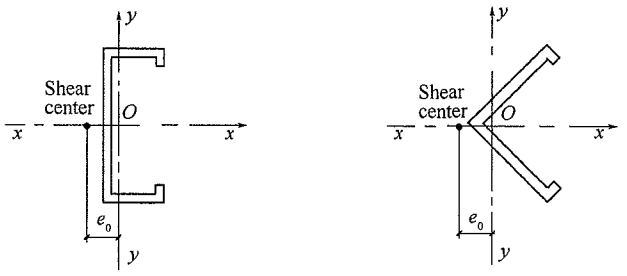



Fig. 5. 2. 4 Sketch of singly-symmetric open section

**5. 2. 5** For axial compression battened members with singly-symmetric open section, their equivalent slenderness ratio for torsional-flexural buckling may be computed by Eq. 5. 2. 4-1, in which the restraint coefficient $\alpha$ and $\beta$ may be obtained from Table 5. 2. 4 while the effective length for torsional buckling $l_\omega = \beta \cdot a$, where $a$ is the largest spacing of centers of adjacent battens.

The number of battens of a member between two supporteds shall not be less than 2 (excluding the battens or end covering plates located at the supporteds).

**5. 2. 6** The stability of latticed members in axial compression shall be checked by Eq. 5. 2. 2, in which the relevant slenderness ratio shall be taken as the larger of $\lambda_{ox}$ and $\lambda_{oy}$ determined as follows:

**1** Battened members built-up of two components (see Fig. 5. 2. 6a).

$$\lambda_{ox} = \lambda_{ox} \tag{5. 2. 6-1}$$

$$\lambda_{oy} = \sqrt{\lambda_y^2 + \lambda_1^2} \tag{5. 2. 6-2}$$

**2** Laced members built-up of two components (see Fig. 5. 2. 6b).

$$\lambda_{0x} = \lambda_x$$

$$\lambda_{0y} = \sqrt{\lambda_y^2 + 27\frac{A}{A_1}} \tag{5. 2. 6-3}$$

**3** Laced members built-up of three components (see Fig. 5. 2. 6c).

$$\lambda_{0x} = \sqrt{\lambda_x^2 + \frac{42A}{A_1(1.5 - \cos^2\theta)}} \tag{5. 2. 6-4}$$

$$\lambda_{0y} = \sqrt{\lambda_y^2 + \frac{42A}{A_1 \cdot \cos^2\theta}} \tag{5. 2. 6-5}$$

Where $\lambda_{0x}$、$\lambda_{0y}$——Equivalent slenderness ratios of the latticed members;

$\lambda_x$——Slenderness ratio of the whole member about the $x$-axis;

$\lambda_y$——Slenderness ratio of the whole member about the virtual ($y$-)axis;

$\lambda_1$——Slenderness ratio of a component about its own principal axis(1-axis), the effective length taken equal to the clear spacing of adjacent battens;

$A$——Sum of the gross sectional area of all the components;

$A_1$——Sum of the gross sectional area of all the lacings in a section of the member;

$\theta$——Angel between a lacing plane and the $x$-axis in a section of the member.

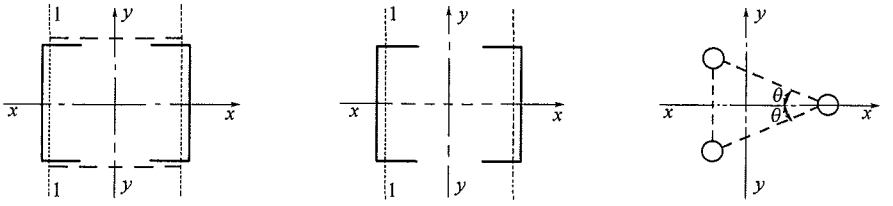



Fig. 5. 2. 6 Sketch of cross-sections of latticed members

The slenderness ratio of a component, $\lambda_1$, of the laced members in axial compression shall not exceed 0. 7 times $\lambda_{max}$, which is the largest of the slenderness ratios of the members about their principal axis; for battened members in axial compression, $\lambda_1$, shall not exceed 40 nor 0. $5\lambda_{max}$ ($\lambda_{max}$ is taken as equal to 50 when it is less than 50), then the strength and stability of a component may not be checked.

The angel between the lacing and the x-axis in a section of the member should be within the range of 40°～70°.

**5. 2. 7** The shear force of latticed members in axial compression shall be calculated according to the following formula:

$$V = \frac{fA}{80}\sqrt{\frac{f_y}{235}} \tag{5. 2. 7}$$

Where $V$——Shear force;

$A$——Sum of the gross sectional area of all the components;

$f_y$——Yield strength of steel, $f_y$=235N/mm$^2$ for Q235 steel, $f_y$=345N/mm$^2$ for Q345 steel.

The shear force $V$ may be considered as constant along the member length and shall be distributed to relevant lacings or batten plates.

## 5. 3 Flexural members

**5. 3. 1** For the flexural members subject to the lateral loads acting in the direction parallel to the principal axis and passing through the shear center of the cross-section (as shown in Fig. 5. 3. 1), the strength and stability shall be checked as follows:

For strength:

$$\sigma = \frac{M_{max}}{W_{enx}} \leqslant f \tag{5. 3. 1-1}$$

$$\tau = \frac{V_{max}S}{It} \leqslant f_v \tag{5. 3. 1-2}$$

For stability:

$$\frac{M_{max}}{\varphi_{bx} W_{ex}} \leqslant f \qquad (5.3.1\text{-}3)$$

Where $M_{max}$——Maximum bending moment about the principal $x$-axis in the span of a member;

$V_{max}$——Maximum shear force;

$W_{enx}$——Effective net section modulus about the principal x-axis, taken as a smaller one;

$\tau$——Shear stress;

$S$——First moment about neutral axis of that part of the gross sectional area above the location where shear stress is calculated;

$I$——Moment of inertia of gross section;

$t$——Sum of all web thickness;

$\varphi_{bx}$——Overall stability factor of the flexural members, determined according to Section A. 2 in Appendix A;

$W_{ex}$——Effective section modulus about the principal $x$-axis, with respect to the extreme compression fiber;

$f_v$——Design value of shear strength of steel.

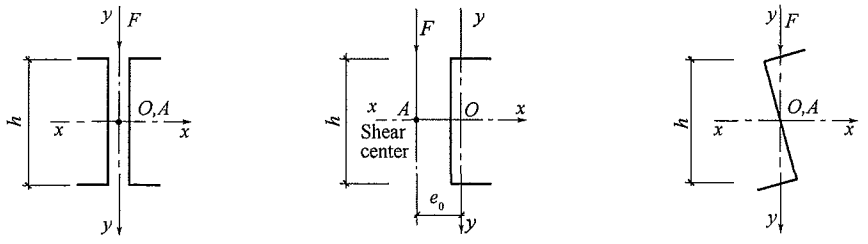



Fig. 5. 3. 1 Sketch of sections of flexural members under the lateral loads acting in the direction parallel to the principal axis and passing through the shear center of the section

**5. 3. 2** For the flexaral members subject to the lateral loads acting in the direction parallel to the principal axis and drifting off the shear center of the cross-section of the flexural member (see Fig. 5. 3. 2), the strength and stability shall be checked by following formulae:

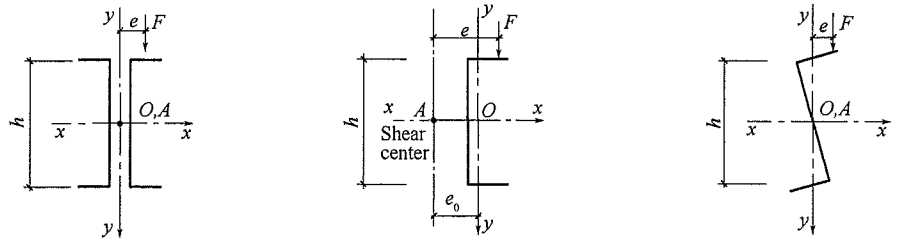



Fig. 5. 3. 2 Sketch of sections of flexural members under the lateral loads acting in the direction parallel to the principal axis and drifting off the shear center of the section

For strength:
$$\sigma = \frac{M}{W_{enx}} + \frac{B}{W_{\omega}} \leqslant f \qquad (5.3.2\text{-}1)$$

For stability:
$$\frac{M_{max}}{\varphi_{bx} W_{ex}} + \frac{B}{W_{\omega}} \leqslant f \qquad (5.3.2\text{-}2)$$

Where $M$——Design value of bending moment;

$B$——Bimoment corresponding to the given bending moment located at the same section, when a rigid decking is securely connected to the compression flange of the flexural members and capable to prevent its lateral deflection and rotation, $B=0$, then the overall stability of such flexural members may not be checked, otherwise, $B$ may be computed according to the provisions of Section A. 4 in Appendix A;

$W_{\omega}$——Sectorial modulus of gross section located at a same position of bending stress under consideration.

Shear stress may be checked by Eq. 5. 3. 1-2.

**5. 3. 3** For the flexural members xubject to the lateral loads acting in the direction inclined to the principal axis and drifting off the shear center of cross-section (see Fig. 5. 3. 3) the strength may be checked by Eq. 5. 3. 3-1, provided the detailing measures can be taken to ensure the overall stability of member:

$$\sigma = \frac{M_x}{W_{enx}} + \frac{M_y}{W_{eny}} + \frac{B}{W_{\omega}} \leqslant f \qquad (5.3.3\text{-}1)$$

Where $M_x$、$M_y$——Bending moments about the principal axis $x$ and $y$ of a cross-section respectively (in Fig. 5. 3. 3, $x$-axis is the strong axis and $y$-axis is the weak axis);

$W_{eny}$——Effective net section modulus about the principal $y$-axis.

The shear stress along the $x$- and $y$-axis may be checked by Eq. 5. 3. 1-2 respectively.

For above flexural members, when the detailing measures can not be used to ensure their overall stability, the stability may be checked according to Eq. 5. 3. 3-2:

$$\frac{M_x}{\varphi_{bx} W_{ex}} + \frac{M_y}{W_{ey}} + \frac{B}{W_{\omega}} \leqslant f \qquad (5.3.3\text{-}2)$$

Where $W_{ey}$——Effective section modulus about the principal $y$-axis with respect to the extreme compressive fiber.

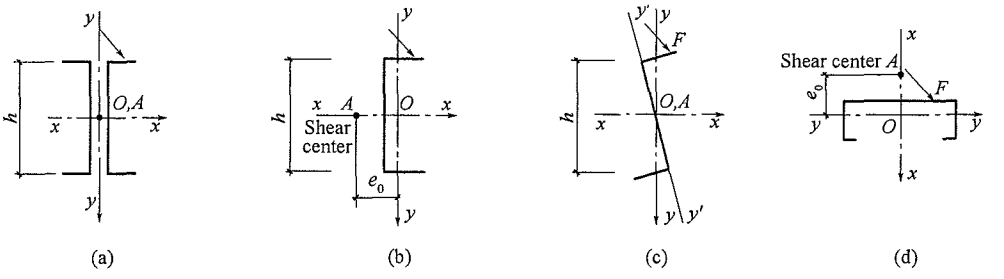



Fig. 5. 3. 3 Sketch of sections of flexural members under the lateral loads acting in the direction inclined to the principal axis and drifting off the shear center of the section

**5.3.4** Webs strengthened with bearing stiffeners at the supports of flexural members shall be checked for overall out-of-plane stability by Eq. 5.2.2, in which the effective length may be taken as the section depth of the flexural member, the cross-section of this strut shall be composed of the stiffener plus a portion of the web $15t\sqrt{235/f_y}$ in width on each side of the stiffener (where $t$ is web thickness).

If the bearing stiffers are not provided at supports, them the local bearing capacity of webs shall be checked according to provisions in Clause 7.1.7.

## 5.4 Members Subjected to Combined Axial Tension and Bending

**5.4.1** The strength of members subjected to combined axial tension and bending shall be checked as follows:

$$\sigma = \frac{N}{A_n} \pm \frac{M_x}{W_{nx}} \pm \frac{M_y}{W_{ny}} \leqslant f \tag{5.4.1}$$

Where $W_{nx}$、$W_{ny}$——Net section moduli about principal axis $x$ and $y$ of cross-section respectively.

In case the compression zone come into existence in the section of members and the width-to-thickness ratios of the compression elements are larger than the effective width-to-thickness ratio specified in Clause 5.6.1, then for computing its net sectional properties the excessive portion of compression elements shall be deducted according to the position as shown in Fig. 5.6.5.

## 5.5 Members Subjected to Combined Axial Compression and Bending

**5.5.1** The strength of members subjected to combined axial compression and bending shall be checked as follows:

$$\sigma = \frac{N}{A_{en}} \pm \frac{M_x}{W_{enx}} \pm \frac{M_y}{W_{eny}} \leqslant f \tag{5.5.1}$$

**5.5.2** For beam-columns of doubly-symmetric section bent in their plane of symmetry, the in-plane stability shall be checked by Eq. 5.5.2-1:

$$\frac{N}{\varphi A_e} + \frac{\beta_m M}{\left(1 - \frac{N}{N'_E}\varphi\right) W_e} \leqslant f \tag{5.5.2-1}$$

Where $M$——Effective bending moment, taken as the maximum bending moment in full length of member;

$\beta_m$——Coefficient of equivalent bending moment;

$N'_E$——Parameter, $N'_E = \frac{\pi^2 EA}{1.165\lambda^2}$;

$E$——Modulus of elastricity of steel;

$\lambda$——Slenderness ratio of a member in the plane of bending;

$W_e$——Effective section modulus for the extreme compression fiber.

Members bent in their principal plane of largest rigidity (see Fig. 5. 5. 2) shall also be checked for overall out-of plane stability by Eq. 5. 5. 2-2:

$$\frac{N}{\varphi_y A_e}+\frac{\eta M_x}{\varphi_{bx} W_{ex}} \leqslant f \tag{5. 5. 2-2}$$

Where $\eta$——Sectional factor, $\eta=0.7$ for closed section, $\eta=1.0$ for others;

$\varphi_y$——Stability factor of axial compression members about $y$-axis, in which the relevant slenderness ratio shall be determined from Eq. 5. 2. 3-2;

$\varphi_{bx}$——Overall stability factor of the flexural members bending in the plane of the largest rigidity, shall be computed in accordance with those specified in Section A. 2 of Appendix A, $\varphi_{bx}=1.0$ for closed section.

$M_x$ shall be taken as the maximum bending moment in the calculated portion of the member.

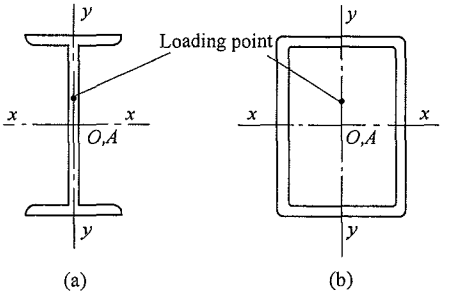



Fig. 5. 5. 2 Sketch of doubly-symmetric sections

**5. 5. 3** The coefficient of equivalent bending moment of beam-columns $\beta_m$ shall be adopted according to the following provisions:

**1** For members braced against lateral translation at two ends and no intermediate transverse loads:

$$\beta_m=0.6+0.4\frac{M_2}{M_1} \tag{5. 5. 3}$$

Where $M_1$、$M_2$——The larger and smaller end moment in absolute value respectively ($|M_1| \geqslant |M_2|$), $\frac{M_1}{M_2}$ is positive when the member is bent in single curvature and negative when it is bent in reverse curvature.

**2** For members braced against lateral translation at two ends and having intermediate transverse loads:

$$\beta_m=1.0$$

**3** For members whose ends having lateral translation:

$$\beta_m=1.0$$

**5.5.4** For beam-columns of singly-symmetric open section (as shown in Fig. 5.2.4) bent in their plane of symmetry, except that their in-plane stability shall be checked according to Clause 5.5.2, their overall out-of-plane stability shall also be verified by Eq. 5.2.2. In case the stability factor of axial compression members $\varphi$ in Eq. 5.2.2 shall be obtained from Table A.1.1-1 or Table A.1.1-2 in Appendis A, according to the equivalent slenderness ratio of members subject to torsional-flexural buckling, $\lambda_\omega$, calculated from Eq. 5.5.4-1.

$$\lambda_\omega = \lambda_x \sqrt{\frac{s^2+a^2}{2s^2} + \sqrt{\left(\frac{s^2+a^2}{2s^2}\right)^2 - \frac{a^2-\alpha(e_0-e_x)^2}{s^2}}} \qquad (5.5.4\text{-}1)$$

With

$$a^2 = e_0^2 + i_x^2 + i_y^2 + 2e_x\left(\frac{U_y}{2I_y} - e_0 - \xi_2 e_a\right) \qquad (5.5.4\text{-}2)$$

$$U_y = \int_A x(x^2+y^2)\,dA \qquad (5.5.4\text{-}3)$$

Where $e_x$——Equivalent eccentricity, $e_x = \pm\frac{\beta_m M}{N}$, in case $e_x$ is positive when it is located on the shear center side of the centroid and negative located on the another side opposite to the shear center. $M$ is taken as maximum moment in the calculated portion of the member;

$\xi_2$——Effect coefficient of transverse loading position, given in Table A.2.1;

$s$——Coefficient, computed by Eq. 5.2.4-2;

$e_a$——Distance from the transverse loading point to the shear center, for eccentric compression members and transverse load passing through the shear center, $e_a=0$; when the load is not at the shear center, $e_a$ is negative for load oriented to the shear center and positive for load departure from the shear center.

For beam-columns having $l_{0x} \leqslant l_{0y}$, when $\left(e_x + \frac{e_0}{2}\right) \leqslant 0$ or adopted the shapes given in Table B.1.1-3 or B.1.1-4 of this code, their overall out-of-plane stability may not be checked.

If the bending moment in the plane of symmetry (as shown in Fig. 5.2.4) causes compression on the shear center side of the centroid, beam-columns shall also be checked as follows:

$$\left|\frac{N}{A_e} - \frac{\beta_{my} M_y}{\left(1-\frac{N}{N'_{Ey}}\right)W'_{ey}}\right| \leqslant f \qquad (5.5.4\text{-}4)$$

Where $\beta_{my}$——Coefficient of equivalent moment about $y$-axis, shall be determined according to the provisions in Clause 5.5.3;

$W'_{ey}$——Smaller effective section modulus;

$N'_{Ey}$——Parameter, $N'_{Ey}=\dfrac{\pi^2 EA}{1.165\lambda_y^2}$.

**5.5.5** For beam-columns of singly-symmetric open sections bent in the plane of non-symmetry (see Fig. 5.5.5), in addition to check their in-plane stability by Eq. 5.5.5-1, their overall out-of-plane stability shall also be checked according to Eq. 5.5.5-2.

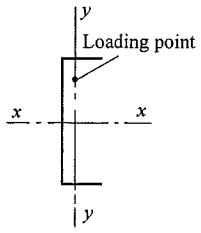



Fig. 5.5.5 Singly-symmetric open section bent about the symmetric axis

$$\frac{N}{\varphi_x A_e}+\frac{\beta_m M_x}{\left(1-\dfrac{N}{N'_{Ex}}\varphi_x\right)W_{ex}}+\frac{B}{W_\omega}\leqslant f \tag{5.5.5-1}$$

$$\frac{N}{\varphi_y A_e}+\frac{M_x}{\varphi_{bx}W_{ex}}+\frac{B}{W_\omega}\leqslant f \tag{5.5.5-2}$$

Where $\varphi_x$——stability factor of axial compression members about the $x$-axis, whose slenderness ratio shall be computed by Eq. 5.2.4-1;

$N'_{Ex}$——parameter, $N'_{Ex}=\dfrac{\pi^2 EA}{1.165\lambda_x^2}$.

**5.5.6** Doubly-symmetric section beam-columns bent in two principal planes shall be checked for stability by the following formulae:

$$\frac{N}{\varphi_x A_e}+\frac{\beta_{mx} M_x}{\left(1-\dfrac{N}{N'_{Ex}}\varphi_x\right)W_{ex}}+\frac{\eta M_y}{\varphi_{by}W_{ey}}\leqslant f \tag{5.5.6-1}$$

$$\frac{N}{\varphi_y A_e}+\frac{\eta M_x}{\varphi_{bx}W_{ex}}+\frac{\beta_{my} M_y}{\left(1-\dfrac{N}{N'_{Ey}}\varphi_y\right)W_{ey}}\leqslant f \tag{5.5.6-2}$$

Where $\varphi_{by}$——Overall stability factor of the flexural members bent in the principal plane of the smallest rigidity, shall be determined in accordance with Section A.2 of Appendix A;

$\beta_{mx}$——Coefficient of equivalent moment about $x$-axis, shall be determined according to the provisions in Clause 5.5.3.

**5.5.7** For the latticed beam-columns, in addition to the strength and overall stability of whole member shall be checked while each single component shall also be checked for its strength and stability.

The lace bars or battens of beam-columns shall be designed by using the actual shear force of member or the shear force given by Clause 5. 2. 7, whichever is larger.

**5. 5. 8** The latticed beam-columns bent about the solid axis ($x$-axis), their overall in-plane and out-of-plane stability shall be checked in the same way as solid web members, but the equivalent slenderness ratio $\lambda_{0y}$ given in Clause 5. 2. 6 shall be used for out-of-plane stability calculation to determine $\varphi_y$ in Eq. 5. 5. 2-2 and $\varphi_b$ taken as 1. 0; While bent about the virtual axis ($y$-axis), their in-plane stability shall be checked as follows:

$$\frac{N}{\varphi_y A_e}+\frac{\beta_{my}M_y}{\left(1-\frac{N}{N'_{Ey}}\varphi_y\right)W_{ey}}\leqslant f \tag{5. 5. 8}$$

Where $\varphi_y$、$N'_{Ey}$ shall be determined by using the equivalent slenderness ratio $\lambda_{0y}$, overall out-of-plane stability may not be checked, but stability of components shall be checked.

## 5. 6 Compressive Elements in Members

**5. 6. 1** Effective width-to-thickness ratios of stiffened-, partially stiffened-and unstiffened-elements shall be determined as follows:

For $\frac{b}{t}\leqslant 18\alpha\rho$:

$$\frac{b_e}{t}=\frac{b_c}{t} \tag{5. 6. 1-1}$$

For $18\alpha\rho<\frac{b}{t}<38\alpha\rho$:

$$\frac{b_e}{t}=\left(\sqrt{\frac{21.8\alpha\rho}{\frac{b}{t}}}-0.1\right)\frac{b_c}{t} \tag{5. 6. 1-2}$$

For $\frac{b}{t}\geqslant 38\alpha\rho$:

$$\frac{b_e}{t}=\frac{25\alpha\rho}{\frac{b}{t}}\cdot\frac{b_c}{t} \tag{5. 6. 1-3}$$

Where $b$——Width of element;

$t$——Thickness of element;

$b_e$——Effective width of element;

$\alpha$——Coefficient of calculation, $\alpha=1.15-0.15\psi$, for $\psi<0$, taken as $\alpha=1.15$;

$\psi$——Non-uniformly distributed coefficient of compressive stress, $\psi=\frac{\sigma_{min}}{\sigma_{max}}$;

$\sigma_{max}$——Maximum compressive stress at the edge of the compressive element ($N/mm^2$), taken as positive;

$\sigma_{min}$——Stress at the another edge of the compressive element ($N/mm^2$), compressive stress taken as positive while tensile stress taken as negative;

$b_c$——Width of compressive portion of element, for $\psi \geqslant 0$, $b_c = b$; for $\psi < 0$, $b_c = \frac{b}{1-\psi}$;

$\rho$——Coefficient, $\rho = \sqrt{\frac{205 k_1 k}{\sigma_1}}$, where $\sigma_1$ is determined according to the provisions in Clauses 5. 6. 7 and 5. 6. 8 of this section;

$k$——Buckling coefficient of compressive element, determined from Clause 5. 6. 2;

$k_1$——Restraint coefficient of plates assembly obtained from those specified in Clause 5. 6. 3, if the restraint effect of adjacent elements is disregarded, $k_1$ may be taken as 1. 0.

**5. 6. 2** Buckling coefficient of compressive elements may be calculated according to the following formule:

**1** Stiffened elements.

For $1 \geqslant \psi > 0$:

$$k = 7.8 - 8.15\psi + 4.35\psi^2 \qquad (5.6.2\text{-}1)$$

For $0 \geqslant \psi \geqslant -1$:

$$k = 7.8 - 6.29\psi + 9.78\psi^2 \qquad (5.6.2\text{-}2)$$

**2** Partially stiffened elements.

1) Maximum compressive stress located at the supported edge (as shown in Fig. 5. 6. 2a).

For $\psi \geqslant -1$:

$$k = 5.89 - 11.59\psi + 6.68\psi^2 \qquad (5.6.2\text{-}3)$$

2) Maximum compressive stress located at the partially stiffened edge (as shown in Fig. 5. 6. 2b).

For $\psi \geqslant -1$:

$$k = 1.15 - 0.22\psi + 0.045\psi^2 \qquad (5.6.2\text{-}4)$$

**3** Unstiffened elements.

1) Maximum compressive stress located at the supported edge (see Fig. 5. 6. 2c).

For $1 \geqslant \psi > 0$:

$$k = 1.70 - 3.025\psi + 1.75\psi^2 \qquad (5.6.2\text{-}5)$$

For $0 \geqslant \psi > -0.4$:

$$k = 1.70 - 1.75 - \psi + 55\psi^2 \qquad (5.6.2\text{-}6)$$

For $-0.4 \geqslant \psi \geqslant -1$:

$$k = 6.07 - 9.51\psi + 8.33\psi^2 \qquad (5.6.2\text{-}7)$$

2) Maximum compressive stress located at the free edge (see Fig. 5. 6. 2d).

For $\psi \geqslant -1$:

$$k=0.567-0.213\psi+0.071\psi^2 \qquad (5.6.2\text{-}8)$$

Note: When $\psi<-1$, $\psi$ need to be taken as $-1$ in computing $k$ of above equations.

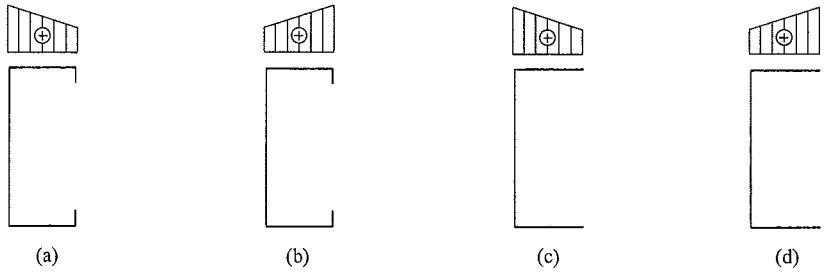



Fig. 5. 6. 2 Sketch of the stress distribution of the partially stiffened and unstiffened-elements

**5. 6. 3** The restraint coefficient of plates assembly of compressive elements shall be calculated by following formulae:

For $\xi \leqslant 1.1$:

$$k_1=\frac{1}{\sqrt{\xi}} \qquad (5.6.3\text{-}1)$$

For $\xi>1.1$:

$$k_1=0.11+\frac{0.93}{(\xi-0.05)^2} \qquad (5.6.3\text{-}2)$$

With

$$\xi=\frac{c}{b}\sqrt{\frac{k}{k_c}} \qquad (5.6.3\text{-}3)$$

Where $b$——Width of computed element;

$c$——Width of elements adjacent to the computed element, if the computed element is a stiffened element with both longitudinal edges connected to other elements, c shall be taken as the width of a adjacent element having larger compressive stress;

$k$——Buckling coefficient of the computed element determined according to Clause 5. 6. 2;

$k_c$——Buckling coefficient of the corresponding adjacent element, determined according to Clause 5. 6. 2.

When $k_1>k'_1$, $k_1$ is taken equal to $k'_1$, where $k'_1$ is the upper limit of $k_1$. $k'_1=1.7$ for stiffened elements; $k'_1=2.4$ for partially stiffened elements; $k'_1=3.0$ for unstiffened element.

When the computed element is a partially stiffened- or unstiffened- element, that is only one longitudinal edge connected to another element and this adjacent element is in

tension, then may take as $k_1=k'_1$.

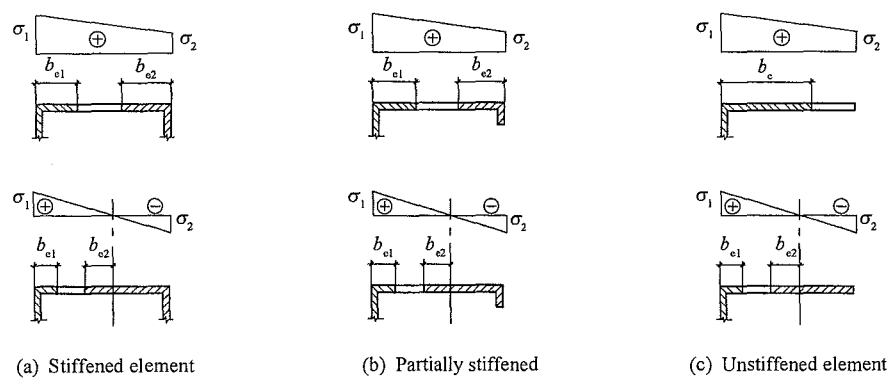
**5.6.4** Depth-to-thickness ratios of lips of partially stiffened elements should not be larger than 12, while the minimum depth-to-thickness ratios of lips shall be obtained from Table 5.6.4 according to the width-to-thickness ratios of partially stiffened elements.

**Table 5.6.4 Minimum Depth-to-thickness Ratios of Lips**

| $\frac{b}{t}$ | 15 | 20 | 25 | 30 | 35 | 40 | 45 | 50 | 55 | 60 |
|---|---|---|---|---|---|---|---|---|---|---|
| $\frac{a}{t}$ | 5.4 | 6.3 | 7.2 | 8.0 | 8.5 | 9.0 | 9.5 | 10.0 | 10.5 | 11.0 |

Note: $a$——Depth of lip;
$b$——Width of element having a simple lip;
$t$——Thickness of element.

**5.6.5** When the width-to-thickness ratios of compressive elements are larger than effective width-to-thickness ratios determined according to Clause 5.6.1, the effective section of compressive elements shall be obtained by deducting the excessive part (i.e. zone without shade lines in Fig. 5.6.5) according to the position as shown in Fig. 5.6.5 from the compressive region of section. The tensile portion of section is fully effective.




(a) Stiffened element (b) Partially stiffened (c) Unstiffened element

Fig. 5.6.5 Sketch of effective sections of compressive elements

In Fig. 5.6.5, $b_{e1}$ and $b_{e2}$ are calculated as follows:

For stiffened elements:

When $\psi \geqslant 0$:

$$b_{e1}=\frac{2b_e}{5-\psi}, \quad b_{e2}=b_e-b_{e1} \tag{5.6.5-1}$$

When $\psi<0$:

$$b_{e1}=0.4b_e, \quad b_{e2}=0.6b_e \tag{5.6.5-2}$$

For partially stiffened- and unstiffened elements:

$$b_{e1}=0.4b_e, \quad b_{e2}=0.6b_e \tag{5.6.5-3}$$

Where $b_e$ is determined according to Clause 5. 6. 1.

**5. 6. 6** When the ratios of external diameter to wall thickness of circular tubes members meet the requirements of Clause 4. 3. 2, their sections may be taken as fully effective in calculation.

**5. 6. 7** Effective width-to-thickness ratios of elements in axial compression members shall be calculated according to those specified in Clause 5. 6. 1, in which $\sigma_1$ shall be taken equal to $\varphi \cdot f$, where $\varphi$ is the stability factor of axial compression members and $f$ is the design value of strength of steel.

**5. 6. 8** Effective width-to-thickness ratios of elements in flexural members and members subjected to combined axial load and bending shall be determined as follows:

**1** For beam-columns, their effective width-to-thickness ratios of elements may be determined in accordance with Clause 5. 6. 1, in which the non-uniformly distributed coefficients $\psi$ of compressive stresses of all elements in section shall be calculated according to strength by gross section of members but the effect of bimoment is disregarded, $\sigma_1$ of the element having maximum compressive stress may be taken as the design value of strength of steel $f$, while the maximum compressive stresses of other elements in section may be computed from $\psi$.

**2** For beams and members subjected to combined axial tension and bending, the effective width-to-thickness ratios of elements may be calculated according to those specified in Clause 5. 6. 1, in which the non-uniformly distributed coefficient $\psi$ of compressive stresses and maximum compressive stresses of all elements in section shall be calculated according to strength by gross section of members while the effect of bimoment is neglected.

**3** The tensile portions of elements is fully effective.

# 6 Calculation and Detailing Requirements of Connections

## 6.1 Calculation of Connections

**6.1.1** The strength of butt weld and fillet weld shall be calculated according to the following formulae:

**1** For the butt weld subjected to axial tension.

$$\sigma=\frac{N}{l_w t}\leqslant f_t^w \tag{6.1.1-1}$$

**2** For the butt weld subjected to axial compression.

$$\sigma=\frac{N}{l_w t}\leqslant f_c^w \tag{6.1.1-2}$$

**3** For the butt weld subjected to both bending and shear.

The tensile stress:

$$\sigma=\frac{M}{W_f}\leqslant f_t^w \tag{6.1.1-3}$$

The shear stress:

$$\tau=\frac{VS_f}{I_f t}\leqslant f_v^w \tag{6.1.1-4}$$

At the location where combined the comparatively large shear stress $\tau$ and normal stress $\sigma$:

$$\sqrt{\sigma^2+3\tau^2}\leqslant 1.1 f_t^w \tag{6.1.1-5}$$

**4** For the transverse fillet weld subjected to shear force (perpendicular to the weld length).

$$\sigma_f=\frac{N}{0.7 h_f l_w}\leqslant 1.22 f_f^w \tag{6.1.1-6}$$

**5** For the longitudinal fillet weld subjected to shear force (parallel to the weld length).

$$\tau_f=\frac{N}{0.7 h_f l_w}\leqslant f_f^w \tag{6.1.1-7}$$

**6** At the location of fillet weld where combined the transverse stress $\sigma_f$ and longitudinal shear stress $\tau_f$.

$$\sqrt{\left(\frac{\sigma_f}{1.22}\right)^2+\tau_f^2}\leqslant f_f^w \tag{6.1.1-8}$$

Where $l_w$——Total effective length of the welds; for the butt weld applied end tabs, the effective length of each weld is taken as its actual weld length $l$; for the butt weld except that mentioned above, and all fillet weld, the effec-

tive length of each weld taken as its actual weld length $l$ minus $2h_f$;

$h_f$——Size of fillet weld leg;

$t$——Thickness of thinner plate in the connected joint;

$W_f$——Sectional modulus of weld section;

$S_f$——Maximum first moment of area of weld section;

$I_f$——Moment of inertia of weld section;

$\sigma_f$——Stress perpendicular to the weld axis, calculated based on effective weld area ($0.7h_f l_w$);

$\tau_f$——Shear stress parallel to the weld axis, calculated based on effective weld area ($0.7h_f l_w$);

$f_c^w$、$f_t^w$——Design value of compressive strength and tensile strength of butt welds respectively;

$f_v^w$——Design value of shear strength of butt weld;

$f_f^w$——Design value of compressive, tensile and shear strength of fillet weld.

**6.1.2** The strength of flare groove weld shall be calculated according to the following formulae:

**1** When the minimum thickness of connected plates is less than or equal to 4mm, the shear strength shall be determined as follows:

For the groove welds subjected to transverse load (see Fig. 6.1.2-1)

$$\tau=\frac{N}{l_w t}\leqslant 0.8f \qquad (6.1.2\text{-}1)$$

For the welds subjected to longitudinal load (see Fig. 6.1.2-2)

$$\tau=\frac{N}{l_w t}\leqslant 0.7f \qquad (6.1.2\text{-}2)$$

Where $t$——Minimum thickness of connected steel plates;

$l_w$——Total effective length of the welds, the effective length of each weld taken as actual weld length minus $2h_f$, the $h_f$ shall be determined according to Fig. 6.1.2-3;

$f$——Design value of tensile strength of connected steel plates.

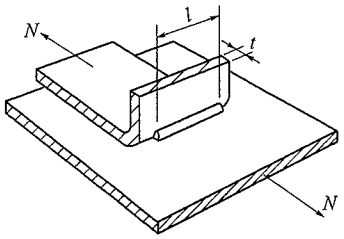
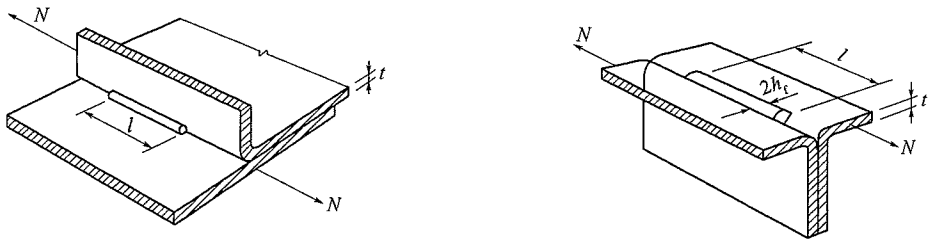



Fig. 6.1.2-1 Flare-bevel groove weld subjected to transverse load




(a) Flare-bevel groove weld　　(b) Flare V groove weld

Fig. 6. 1. 2-2　Flare groove weld subjected to longitudinal load

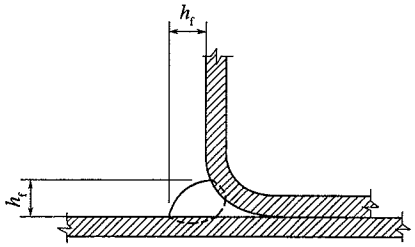



Fig. 6. 1. 2-3　Flare-bevel groove weld

**2** When the minimum thickness of connected plates is greater than 4mm, besides calculating according to formula 6. 1. 2-2, the shear strength for flare groove welds subjected to longitudinal load shall be checked supplementary according to formula 6. 1. 1-7, but the $h_f$ shall be determined according to Fig. 6. 1. 2-2b or Fig. 6. 1. 2-3.

**6. 1. 3** Resistance spot welding may be used for connecting sheets or composing build-up members. The maximum shear force transmitted by each welding spot shall not be larger than the design value of shear capacity given in Table 4. 2. 6 in this code.

**6. 1. 4** The strength of ordinary bolts shall be calculated according to the following requirements:

**1** The tensile force on each bolt in the ordinary bolted connection subjected to tension along bolt shank direction shall not be larger than the design value of tensile capacity $N_t^b$ calculated according to the following equation.

$$N_t^b = \frac{\pi d_e^2}{4} f_t^b \qquad (6.1.4\text{-}1)$$

Where $d_e$——Effective diameter of bolt at thread portion;

$f_t^b$——Design value of tensile strength of bolt.

**2** The shear force on each bolt in the ordinary bolted connection subjected to shear shall not be larger than the lesser design value of shear capacity $N_v^b$ and bearing capacity $N_c^b$ calculated according to the following formulae.

The design value of shear capacity:

$$N_v^b = n_v \frac{\pi d^2}{4} f_v^b \qquad (6.1.4\text{-}2)$$

The design value of bearing capacity:

$$N_c^b = d\Sigma t f_c^b \qquad (6.1.4\text{-}3)$$

Where $n_v$——Number of shear planes;

$d$——Shank diameter of bolt, for the full threaded bolt, taken as $d=d_e$;

$\Sigma t$——Lesser sum of thickness of connected plates bearing in the same direction;

$f_c^b$、$f_v^b$——Design value of bearing and shear strength of bolt respectively.

**3** The ordinary bolted connection subjected to combined effects of shear and tension in bolt shank direction shall satisfy the following formulae:

$$\sqrt{\left(\frac{N_v}{N_v^b}\right)^2 + \left(\frac{N_t}{N_t^b}\right)^2} \leqslant 1 \qquad (6.1.4\text{-}4)$$

$$N_v \leqslant N_c^b \qquad (6.1.4\text{-}5)$$

Where $N_v$、$N_t$——Shear and tensile force transmitted by each bolt respectively.

**6.1.5** The strength of high-strength bolts in the friction-type connections shall be calculated according to the following formulae:

**1** The shear force on each bolt shall not be larger than the design value of shear capacity $N_v^b$ calculated according to the following equation.

$$N_v^b = \alpha \cdot n_f \cdot \mu \cdot P \qquad (6.1.5\text{-}1)$$

Where $\alpha$——Coefficient, for minimum thickness of plate $t \leqslant 6$ mm, taken as 0.8, for minimum thickness of plate $t > 6$ mm, taken as 0.9;

$n_f$——Number of the friction surfaces transmitting force;

$\mu$——Slip coefficient taken from Table 6.1.5-1;

$P$——Pretension force of a high strength bolt taken from Table 6.1.5-2.

**Table 6.1.5-1 Value of Slip Coefficient $\mu$**

| Contact surface treatment of connected parts | Steel grade of connected parts | |
|---|---|---|
| | Q235 | Q345 |
| Blast-cleaned | 0.40 | 0.45 |
| Clean mill scale of hot rolled steel surface | 0.30 | 0.35 |
| Clean mill scale of cold rolled steel surface | 0.25 | — |
| Note: The direction of clean mill scale shall be perpendicular to the force direction. | | |

**Table 6.1.5-2 Value of Pretension Force of High-strength Bolt $P$ (kN)**

| Property grade of bolt | Nominal diameter of bolt (mm) | | |
|---|---|---|---|
| | M12 | M14 | M16 |
| Grade 8.8 | 45 | 60 | 80 |
| Grade 10.9 | 55 | 75 | 100 |

**2** The tensile force on each bolt in shank direction shall not be larger than the de-

sign value of tensile capacity $N_t^b$ calculated according to the following equation.

$$N_t^b = 0.8P \qquad (6.1.5\text{-}2)$$

**3** The high-strength bolt subjected to combined effects of shear $N_v$ in friction interface and tensile force $N_t$ in bolt shank direction shall satisfy the following formulae:

$$N_v \leqslant N_v^b = \alpha \cdot n_f \cdot \mu \cdot (P - 1.25N_t) \qquad (6.1.5\text{-}3)$$

$$N_t = 0.8P \qquad (6.1.5\text{-}4)$$

**6.1.6** When bolted joint length $l_b$ at the connection of members or at one side of members splice, measured in the direction of force transfer, is larger than $15d_0$, the design value of carrying capacity of all the bolts shall be reduced by multiplying a reduction factor $\left(1.1 - \frac{l_b}{150d_0}\right)$; when the $l_b$ is larger than $60d_0$, the reduction factor shall be taken as 0.7, where $d_0$ is the hole diameter for a bolt.

**6.1.7** The strength of blind rivets (pull-stem rivets), self-tapping screws and cartridge fired pins used to fasten profiled steel sheetings together or sheeting to other cold-formed steel members, in direct contact with each other may be calculated according to the following requirements:

**1** The tensile force on each self-tapping screw or cartridge fired pin in the connection with these fasteners subjected to tension along fastener shank direction shall not be larger than the design value of tensile capacity calculated according to the following formulae;

For static loads only:

$$N_t^f = 17tf \qquad (6.1.7\text{-}1)$$

For combination loads including wind load:

$$N_t^f = 8.5tf \qquad (6.1.7\text{-}2)$$

Where $N_t^f$——Design value of tensile capacity per self-tapping screw or cartridge fired pin (N);

$t$——Thickness of profiled steel sheeting next to the head of the fastener (mm), which should satisfy 0.5mm$\leqslant t \leqslant$1.5mm;

$f$——Design value of tensile strength of connected steel sheets (N/mm$^2$).

When the fastener is located at a quarter point in the trough of the sheeting (see Fig. 6.1.7 b), its design value of tensile capacity shall be reduced by multiplying reduction factor 0.9; when the fasteners are located at both quarter points (see Fig. 6.1.7 c), it shall be multiplied by reduction factor 0.7.

Penetration depth $t_c$ of self-tapping screw in the base material shall be larger than 0.9mm, and the tensile force on this screw shall not larger than the design value of tensile capacity calculated according to the following equation.

$$N_t^f = 0.75t_c df \qquad (6.1.7\text{-}3)$$

Where $d$——Nominal diameter of screw (mm);

$t_c$——Penetration depth of cylindrical thread part of the screw in the base material (mm);

$f$——Design value of tensile strength of the base material (N/mm$^2$).

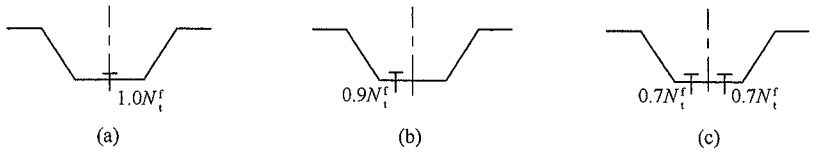



Fig. 6. 1. 7 Sketch for profiled steel sheeting fastening

**2** When the fasteners subjected to shear, the shear force on each fastener shall not be larger than the design value of shear capacity calculated according to the following formulae.

For blind rivets and self-tapping screws:

When $\frac{t_1}{t}=1$:

$$N_v^f=3.7\sqrt{t^3 d}f \qquad (6.1.7\text{-}4)$$

And

$$N_v^f \leqslant 2.4tdf \qquad (6.1.7\text{-}5)$$

When $\frac{t_1}{t}\geqslant 2.5$:

$$N_v^f=2.4tdf \qquad (6.1.7\text{-}6)$$

When $\frac{t_1}{t}$ is between 1 and 2.5, the $N_v^f$ may be determined by linear interpolation based on formulae 6.1.7-4 and 6.1.7-6.

Where $N_v^f$——Design value of shear capacity per fastener (N);

$d$——Nominal diameter of a rivet or a screw (mm);

$t$——Thickness of the thinner steel plate (in contact with the fastener head) (mm);

$t_1$——Thickness of the thicker steel plate(in contact with the site formed head or not in contact with the screw head) (mm);

$f$——Design value of tensile strength of connected steel sheets(N/mm$^2$).

For cartridge fired pins:

$$N_v^f = 3.7tdf \qquad (6.1.7\text{-}7)$$

Where $t$——Thickness of single connected steel sheet (mm);

$d$——Diameter of the cartridge fired pin (mm);

$f$——Design value of tensile strength of connected steel sheets (N/mm$^2$).

When the blind rivets or self-tapping screws are used to fasten the end of profiled steel sheeting on to the supporting members (e. g. purlins), its design value of shear ca-

pacity shall be multiplied by reduction factor 0.8.

**3** The screwed or pinned connection subjected to combined effects of shear and tension shall satisfy the following formula:

$$\sqrt{\left(\frac{N_v}{N_v^f}\right)^2+\left(\frac{N_t}{N_t^f}\right)^2}\leqslant 1 \qquad (6.1.7\text{-}8)$$

Where $N_v$、$N_t$——Shear and tensile forces transmitted by each fastener respectively;

$N_v^f$、$N_t^f$——Design values of shear and tensile capacity per fastener respectively.

**6.1.8** The maximum longitudinal spacing $\alpha_{max}$ of connectors (such as welds, spot weldings, bolts etc.), joining two channels (or C-sections) to form built-up I-section shall be determined according to the following requirements:

**1** For the members under combined compressive axial load and bending, it shall be taken as the lesser value calculated according to the following formulae.

$$\alpha_{max}=\frac{n_1 N_v^f I_y}{VS_y} \qquad (6.1.8\text{-}1)$$

$$\alpha_{max}=\frac{li_1}{2i_y} \qquad (6.1.8\text{-}2)$$

Where $n_1$——Number of connectors at the same section;

$N_v^f$——Design value of shear capacity per connector, for spot welding, it may be taken as $N_v^f = N_v^s$;

$I_y$——Moment of inertia of built-up I-section about its centroidal $y$-axis parallel to the web;

$V$——Shear force, taken as the larger value between real and calculated according to Clanse 5.2.7;

$S_y$——First moment of area of one channel about $y$-axis;

$l$——Unbraced length of member;

$i_1$——Radius of gyration of one channel about its central axis parallel to the web;

$i_y$——Radius of gyration of built-up I-section about its $y$-axis.

**2** For the flexural members:

$$\alpha_{max}=\frac{2N_t^f h_0}{dq_0} \qquad (6.1.8\text{-}3)$$

Where $N_t^f$——Design value of tensile capacity per connector, for spot welding, it may be taken as $N_t^f=0.3N_v^s$;

$h_0$——Vertical distance between the two rows of connectors nearest to the top and bottom flanges;

$d$——Distance from the shear center of one channel to the mid-plane of its web;

$q_0$——Equivalent load intensity.

The equivalent load intensity of flexural members shall be determined according to the following requirements: for the uniformly distributed load, it shall be taken equal to

3 times the uniformly distributed load; for the concentrated loads or reactions, it shall be taken as the larger value obtained by dividing above loads or reactions by their length of bearing or the longitudinal spacing of connectors.

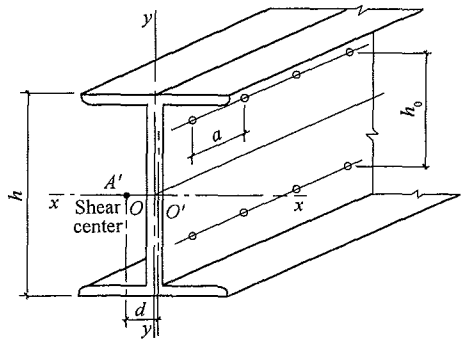



Fig. 6. 1. 8 Sketch of built-up I-section

Note: $A'$ is the shear center of one channel;

$O'$ is the intersection point of central line of web of one channel and symmetric $x$-axis.

## 6. 2 Detailing requirements of connections

**6. 2. 1** When thickness of connected plates $t \leqslant 6$mm, the effective length of weld shall not be less than 30mm; when $t > 6$mm, it shall not be less than 40mm. The size of fillet weld leg should not be larger than 1. $5t$ ( $t$ is the thickness of thinner plate in the connected parts). The size of fillet weld leg in the contoured steel tube joints may not be larger than 2. $0t$.

**6. 2. 2** When using the flare groove weld, the size of flare-bevel groove weld leg $h_f$ (see Fig. 6. 1. 2-3) shall not be less than 1. 4 times the minimum thickness of connected plates.

**6. 2. 3** The spot spacing of resistance spot welding should not be less than $15\sqrt{t}$ (mm), the edge distance of spot should not be less than $10\sqrt{t}$ (mm) ( $t$ is the thickness of thinner plate in the connecter parts).

**6. 2. 4** The spacing centre-to-centre of bolts shall not be less than 3 times $d_0$, the end distance shall not be less than 2 times $d_0$, and the edge distance shall not be less than 1. 5 times $d_0$, where $d_0$ is the diameter of bolthole (see Fig. 6. 2. 4). The edge distance of bolt hole close to the rounded corners shall also meet the installation requirements by using fastening tools.

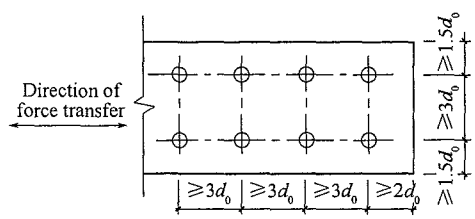



Fig. 6. 2. 4 Sketch of minimum spacing between bolts

**6.2.5** The thinner plate shall be next to the preformed head of the blind rivets (pull-stem rivets) or to the head of the self-tapping screws. The spacing centre-to-centre and the end distance of fasteners shall not be less than 3 times the diameter of fasteners, the edge distance shall not be less than 1.5 times the diameter of fasteners. The number of fasteners in the force transfer connections should not be less than 2.

**6.2.6** The applicable diameters for the blind rivets are 2.6～6.4mm, but it should not be less than 4mm for which used in stressed skin structures; the applicable diameters for the self-tapping screws are 3.0～8.0mm, but it should not be less than 5mm for which used in stressed skin structures.

**6.2.7** The diameter $d_0$ of pre-drilled holes in connected plates for self-tapping screws shall satisfy the following formulae:

$$d_0 = 0.7d + 0.2t_t \qquad (6.2.7\text{-}1)$$

And

$$d_0 \leqslant 0.9d \qquad (6.2.7\text{-}2)$$

Where $d$——Nominal diameter of self-tapping screws(mm);

$t_t$——Sum of thickness of connected plates(mm).

**6.2.8** The cartridge fired pins are only used to fasten sheets to supporting members (i.e. the base material, such as purlins). The end or edge distances and spacing between the pins shall not be less than 4.5 times the diameter of pins, and also the spacing center-to-center shall not be less than 20mm, the distances from the center of pins to the end and edge of the base material shall not be less than 15mm, the applicable diameters for the cartridge fired pins are 3.7～6.0mm.

The penetration depth of the pins (referred to as the depth from the point of pins to the surface of base material, as shown in Fig. 6.2.8) shall not be less than 10mm.

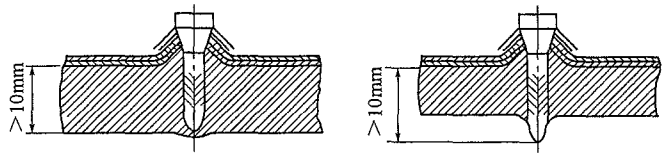



Fig. 6.2.8 Penetration depth of pins

The yield strength of base material shall not be less than 150N/mm², the maximum yield strength of connected steel sheets shall not be larger than 360N/mm². The thickness of the base material and the connected steel sheets shall meet the requirements of Table 6.2.8-1 and Table 6.2.8-2.

**Table 6.2.8-1 Maximum Thickness of Connected Steel Sheets (mm)**

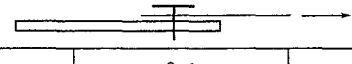
| Diameter of cartridge fired pins (mm) | ≥3.7 | ≥4.5 | ≥5.2 |
|---|---|---|---|
| In one direction |  | | |
| Maximum thickness of single fastened steel sheets | 1.0 | 2.0 | 3.0 |
| Maximum thickness of all fastened steel sheets | 1.4 | 2.5 | 3.5 |

**Table 6. 2. 8-1 (Continued)**

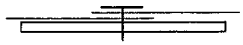
| Diameter of cartridge fired pins (mm) | ≥3. 7 | ≥4. 5 | ≥5. 2 |
|---|---|---|---|
| In opposite direction | ←  → | | |
| Maximum thickness of all fastened steel sheets | 2. 8 | 5. 0 | 7. 0 |

**Table 6. 2. 8-2 Minimum Thickness of Base Material (mm)**

| Diameter of cartridge fired pins (mm) | ≥3. 7 | ≥4. 5 | ≥5. 2 |
|---|---|---|---|
| Minimum thickness (mm) | 4. 0 | 6. 0 | 8. 0 |

**6. 2. 9** In the tensile connections, the diameter of head or washer for the self-tapping screws and cartridge fired pins shall not be less than 14mm; and it shall also ensure that the pull-out strength of fasteners from the base material determined by testing shall not be less than the design value of tensile capacity of the fasteners.

# 7 Profiled Steel Sheeting

## 7.1 Calculation of Profiled Steel Sheeting

**7.1.1** The requirements on calculation in this section only apply to the profiled steel sheeting used as the roof and wall cladding and the profiled steel sheeting designed as permanent formwork without composite effects in composite floor slab.

**7.1.2** The effective width-to-thickness ratio of the compression flange of profiled steel sheeting (as shown in Fig. 7.1.2) shall be adopted in accordance with following requirements:

**1** For the compression flange of which both longitudinal edges connected by webs, or one longitudinal edge connected by the web and the other connected by the intermediate stiffener which should satisfy the requirements of Clause 7.1.4, then its effective width-to-thickness ratio may be determined as a stiffened element in accordance with Clause 5.6.1;

**2** For the compression flange of which one longitudinal edge connected by the edge stiffener which should satisfy the requirements of Clause 7.1.4, web, and the other connected by its effective width-to-thickness ratio may be determined as a partially stiffened element in accordance with Clause 5.6.1.

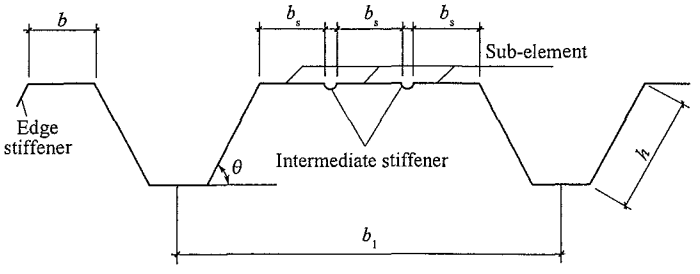



Fig. 7.1.2 Section of profiled steel sheeting

**7.1.3** The effective width-to-thickness ratio of the web of profiled steel sheeting shall be determined as a stiffened element in accordance with Clause 5.6.1.

**7.1.4** The longitudinal stiffener of the compression flange of profiled steel sheeting shall conform to the following requirements:

For edge stiffener:

$$I_{es} \geqslant 1.83t^4 \sqrt{(\frac{b}{t})^2 - \frac{27100}{f_y}} \qquad (7.1.4\text{-}1)$$

And

$$I_{es} \geqslant 9t^4$$

For intermediate stiffener:

$$I_{is} \geqslant 3.66t^4 \sqrt{(\frac{b_s}{t})^2 - \frac{27100}{f_y}} \qquad (7.1.4\text{-}2)$$

And $$I_{is} \geqslant 18t^4$$

Where $I_{es}$——Moment of inertia of the edge stiffener about its own centroidal axis parallel to the element to be stiffened;

$I_{is}$——Moment of inertia of the intermediate stiffener about its own centroidal axis parallel to the element to be stiffened;

$b_s$——Width of sub-element;

$b$——Width of the element connected with the edge stiffener;

$t$——Thickness of element.

**7.1.5** The calculation of the load-carrying capacity of profiled steel sheeting may be calculated as a flexural member by adopting either effective section of a single trough or the whole effective cross section of the profiled steel sheeting.

**7.1.6** The shear stress of the web of profiled steel sheeting shall be checked by the following formulase:

When $h/t<100$:

$$\tau \leqslant \tau_{cr} = \frac{8550}{(h/t)} \qquad (7.1.6\text{-}1)$$

$$\tau \leqslant f_v \qquad (7.1.6\text{-}2)$$

When $h/t \geqslant 100$:

$$\tau \leqslant \tau_{cr} = \frac{855000}{(h/t)^2} \qquad (7.1.6\text{-}3)$$

Where $\tau$——Average shear stress of the web ($N/mm^2$);

$\tau_{cr}$——Critical shear buckling stress of the web;

$h/t$——Height-to-thickness ratio of the web.

**7.1.7** For a web of profiled steel sheeting at support, the local crippling load carrying capacity of the web shall be checked by the following formulase:

$$R \leqslant R_w \qquad (7.1.7\text{-}1)$$

$$R_w = \alpha t^2 \sqrt{fE}(0.5 + \sqrt{0.02l_c/t})[2.4 + (\theta/90)^2] \qquad (7.1.7\text{-}2)$$

Where $R$——Reaction of the support applied on a web;

$R_w$——Design value of web crippling of a web;

$\alpha$——Coefficient, $\alpha=0.12$ for the internal support, $\alpha=0.06$ for the end support;

$t$——Thickness of the web;

$l_c$——Bearing length at support, $10mm<l_c<200mm$, $l_c=10mm$ for the end support;

$\theta$——Inclination of the web ($45° \leqslant \theta \leqslant 90°$).

**7.1.8** The cross sections of profiled steel sheeting subject to a combination of bending moment $M$ and support reaction $R$ shall be satisfied with the following requirements:

$$M/M_u \leqslant 1.0 \tag{7.1.8-1}$$

$$R/R_w \leqslant 1.0 \tag{7.1.8-2}$$

$$M/M_u + R/R_w \leqslant 1.25 \tag{7.1.8-3}$$

Where $M_u$——Design value of bending capacity of the cross section, $M_u = W_e f$.

**7.1.9** The cross sections of profiled steel sheeting subject to a combination of bending moment $M$ and shear force $V$ should be satisfied with the following requirement:

$$\left(\frac{M}{M_u}\right)^2 + \left(\frac{V}{V_u}\right)^2 \leqslant 1 \tag{7.1.9}$$

Where $V_u$——Design value of the shear force of the incline web, $V_u = (ht \cdot \sin\theta)\tau_{cr}$, $\tau_{cr}$ shall be calculated according to Clause 7.1.6.

**7.1.10** When a concentrated load $F$ act on a single trough of profiled steel sheeting, the concentrated load $F$ can be replaced by an equivalent uniform linear load $q_{re}$ along with the width of the profiled steel sheeting and calculated by the following formula. The profiled steel sheeting may be calculated as a flexural member under the uniform linear load $q_{re}$ at the mid-span by using the effective cross section of either a single trough or the whole cross section of the profiled steel sheeting.

$$q_{re} = \eta \frac{F}{b_1} \tag{7.1.10}$$

Where $F$——Concentrated load;

$b_1$——The width of a single trough of profiled steel sheeting;

$\eta$——Conversion coefficient, which is determined by test. In the absence of any test information, taken as $\eta = 0.5$.

The concentrated load $F$ act on the roof profiled steel sheeting during construction or maintenance should be taken as 1.0kN. When the construction loads exceed 1.0kN, the construction loads shall be taken according to the practical conditions at the work site.

**7.1.11** The ratio of deflection to span of profiled steel sheeting shall not exceed the following limitations:

| | | |
|---|---|---|
| For roof plate: | when the roof slope is less than 1/20 | 1/250; |
| | when the roof slope is more than or equal to 1/20 | 1/200; |
| For wall panel: | | 1/150; |
| For floor sheeting: | | 1/200. |

**7.1.12** The loads act on the profiled steel sheeting which only be used as permanent formwork, are the dead load, load of wet reinforced concrete slab and necessary construction loads. If the reliable measurement is made during construction, the impact

force of pouring concrete may not be considered, and the construction loads should not be included when checking the deflection of the profiled steel sheeting.

### 7.2 Detailing Requirements of Profiled Steel Sheeting

**7.2.1** The angle of inclination $\theta$ between the web and flange of profiled steel sheeting should not be less than 45°.

**7.2.2** The steel sheets to be rolled for profiled steel sheeting should be zinc coated, zinc-aluminum coated or preprinted steel sheets.

**7.2.3** For roof and wall claddings, the thickness of base steel sheet of profiled steel sheeting should be 0.4～1.6mm; for the profiled steel sheeting designed as permanent formwork of the composite floor slab it should not be less than 0.5mm. In order to reduce the end overlaps in the longitudinal direction , it will be better to choose the profile sheeting in longer length.

**7.2.4** End-overlaps in the longitudinal direction of sheets shall be firmly connected to the supporting member (such as purlin, side rail etc.). The sealant taps should be provided along location of end-overlaps for waterproof. The overlapping length of sheets should not be less than following values:

350mm for the roof sheeting with the higher depth $\geqslant$70mm;

250mm for the roof sheeting with the lower depth $<$70mm and roof slope $\leqslant$1/10;

200mm for the roof sheeting with the lower depth $<$70mm and roof slope $>$1/10;

120mm for the wall profiled steel sheeting.

**7.2.5** Side-overlaps in the lateral direction of roof sheets may adopt overlapping, seam-lock or standing seam system etc.. If the overlapping is used, one trough of profiled steel sheeting needs to be lapped; however in the special requirement cases may be two of it needed. The lapping part should be connected firmly by using fasteners at crest of the profile ribs. The fasteners should be proved waterproof sealant. The distance of fasteners is 700～800mm for higher-depth sheeting and 300～400mm for lower-depth sheeting.

For the side-overlaps by using seam-lock or standing seam system, a special clip support which is suitable for connecting the profiled sheeting shall be fixed on the purlin by the self-tapping screws or cartridge fired pins. The profiled roof sheeting should be laid on the clip support and shall be ensured that the side-overlaps by using seam-lock or standing seam are reliable under the action of wind up-lift suction force etc..

**7.2.6** Side-overlaps in the lateral direction of wall sheets may adopt overlapping and one trough of profiled steel sheeting needs to be lapped. The fasteners can be fixed at the crest or at the valley of the profiled steel sheeting. The fastener of self-tapping screws also should be proved waterproof sealant.

**7.2.7** For the roof by laying higher-depth profiled steel sheeting, the special fixed support should be assembled on purlin and the upper flange width of purlin should be 10mm wider than width of the special fixed support. The special fixed support is connected with purlin by using self-tapping screws or cartridge fired pins to be set at each trough of the support. However there is no special fixed support needed for lower-depth profiled steel sheeting, but the self-tapping screws or cartridge fired pins with waterproof sealant should be set for connecting profiled sheel sheetings with the purlin. The fastener may be fixed at the crest of the profiled steel sheeting and one for every trough or for every two trough. However a whole lower-depth profiled sheel shall not be less than 3 fasteners.

**7.2.8** When the profiled steel sheeting is used as permanent formwork of the composite floor slab and supported on steel beams, the supported length of profiled steel sheeting shall not be less than 50mm; when supported on other materials such as concrete, brick masonry etc. , the supported length should not be less than 75mm. Before pouring concrete, the deleterious matter on the profiled steel sheeting such as grease, dirt etc. should be cleaned up.

**7.2.9** When laying floor profiled steel sheeting, the unusual heavy construction concentrated loads should be avoided, otherwise the temporary bracing support should be set, if necessary.

# 8 Purlins and Side Rails

## 8.1 Calculation of Purlins

**8.1.1** When the roof can prevent the lateral buckling and twisting of a solid web purlin (as shown in Fig. 8.1.1), its strength may be checked as follows:

$$\sigma=\frac{M_x}{W_{enx}}+\frac{M_y}{W_{eny}}\leqslant f \tag{8.1.1-1}$$

Otherwise, if the roof can not prevent the lateral buckling and twisting of a solid web purlin, its stability may be checked as follows:

$$\frac{M_x}{\varphi_b W_{ex}}+\frac{M_y}{W_{ey}}\leqslant f \tag{8.1.1-2}$$

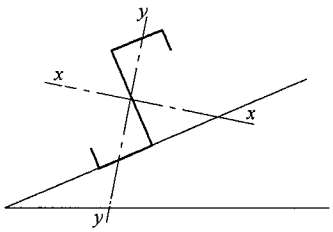



Fig. 8.1.1 Sketch of a solid web purlin

**8.1.2** When the bottom flange of a solid web purlin is in compression due to wind load, its stability may be checked by Eq. 8.1.1-2.

**8.1.3** The strength of top chord of a plane latticed purlin may be checked by Eq. 5.5.1, while its stability may be verified as follows:

$$\frac{N}{\varphi_{min}A_e}+\frac{M_x}{W_{ex}}+\frac{M_y}{W_{ey}}\leqslant f \tag{8.1.3-1}$$

Where $\varphi_{min}$——Stability factor of axial compression members may be obtained from Table A.1.1 in Appendix A according to the maximum slenderness ratio of the member;

$M_x$、$M_y$——Bending moment about the principal axis $x$ and $y$ of the top chord section of a purlin respectively, where $x$-axis is perpendicular to the roof slope.

The bending moment $M_x$ and $M_y$ in above equation may be adopted according to the following provisions:

**1** $M_x$ may be calculated by treating the tie rods as the lateral supports of a purlin.

In computing the strength of a purlin, $M_x$ at supports may be calculated by following equation:

$$M_x = \frac{q_y l_1^2}{10} \qquad (8.1.3\text{-}2)$$

While checking for the stability of the purlin, $M_x$ may be taken as the maximum bending moment along the whole unbraced length between the lateral supports.

**2** At joints and mid-span:

$$M_y = \frac{q_x a^2}{10} \qquad (8.1.3\text{-}3)$$

Where $l_1$——Distance between the lateral supports;

$a$——Panel length of the top chord;

$q_x$——Load component in the direction normal to the roof slope;

$q_y$——Load component in the direction parallel to the roof slope.

**8.1.4** When the bottom chord of a plane latticed purlin is in compression due to wind load, it shall be made of shapes, its strength and stability may be checked as follows:

For strength:

$$\sigma = \frac{N}{A_{en}} \leqslant f \qquad (8.1.4\text{-}1)$$

For stability:

$$\frac{N}{\varphi_{min} A_e} \leqslant f \qquad (8.1.4\text{-}2)$$

**8.1.5** For the compression chord of a plane latticed purlin, the in-plane effective length shall be equal to its panel length and the out-of-plane effective length shall be taken as the distance between the lateral supports (the tie rods provided in the chord may be treated as lateral supports). The effective length of web members in any plane may all be taken as their geometric length.

The slenderness ratios of end posts shall not exceed 150.

**8.1.6** For the deflection of a purlin in the direction normal to the roof slope, the ratio of its allowable value to the span length of the purlin may be taken as follows:

**1** Roofing of corrugated sheet: 1/150;

**2** Roofing of profiled steel sheeting, or wire-mesh cement or other cement products: 1/200.

## 8.2 Detailing of Purlins

**8.2.1** The solid web purlins may be connected to the trusses or rigid frames by using cleats (as shown in Fig. 8.2.1).

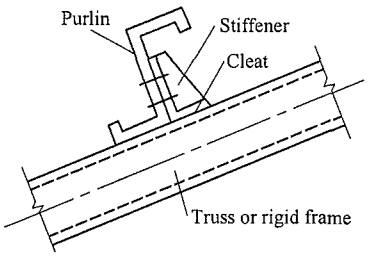



Fig. 8. 2. 1 End connection of solid web purlin

**8. 2. 2** The ratio of height to span for a plane latticed purlin may be taken as 1/12～1/20.

For the web members of a plane latticed purlin, the end posts shall be made of shapes.

If the bottom chord of a plane latticed purlin is in compression due to wind load, the tie rods and struts should be provided in the top and bottom chords of the purlin.

**8. 2. 3** When a solid-web purlin span exceeds 4m the tie rods or struts shall be provided in the compression flange and designed by calculation. The diameter of rods should not be less than 10mm while the slenderness ratio of struts should not be larger than 200.

When the profiled steel sheeting are provided in the surface of the top and bottom flanges of a purlin and securely connected to the flanges, so the tie rods and struts may not be supplied.

**8. 2. 4** In case a purlin is used as a compression member of the horizontal bracing, the slenderness ratio of this purlin shall not exceed 200 (tie rods and struts may be considered as lateral supports) while its strength and stability shall be checked in accordance with a beam-column.

## 8. 3 Calculation of Side Rails

**8. 3. 1** The strength of a simply supported side rails (as shown in Fig. 5. 3. 3d) shall be checked according to Eq. 5. 3. 3-1 and following formulae:

$$\tau_x = \frac{3V_{xmax}}{4b_0 t} \leqslant f_v \qquad (8.3.1\text{-}1)$$

$$\tau_y = \frac{3V_{ymax}}{2h_0 t} \leqslant f_v \qquad (8.3.1\text{-}2)$$

Where $V_{xmax}$、$V_{ymax}$——Maximum value of shear force due to the design values of vertical load ($q_x$) and horizontal wind load ($q_y$) respectively;

$b_0$、$h_0$——Effective depth of side rail section measured along the principal axis $x$ and $y$ respectively, taken as the width of flat portion of each relevant element;

$t$——Thickness of side rail section.

For the side rails with both side flanges attached to cladding and with one side

flange attached to cladding but another side flange having tie rods (struts) which may prevent its twisting, so the effect of torsional-flexural bimoment may be neglected (i. e. may take as $B=0$).

**8. 3. 2** If the overall stability of a side rail can not be ensured by taking the detailing measure, then it shall also be checked according to Eq. 5. 3. 3-2, but in which $\varphi_{bx}$ shall be calculated in accordance with those specified in Section A. 2 of Appendix A for only $M_x$ is applied in the side rail ($M_y$ and $B$ are neglected).

**8. 3. 3** The ratios of allowable deflection to span length of side rails may be determined as follows:

**1** Cladding of profiled steel sheeting and corrugated sheet (horizontal): 1/150;

**2** Side rail at the top of windows (vertical and horizontal): 1/200.

While the vertical deflection of the side rails shall not exceed 10mm.

## 8. 4 Detailing of Side Rails

**8. 4. 1** The side rails are mainly subjected to the horizontal wind load, so their principal plane of largest rigidity should be in the horizontal direction.

**8. 4. 2** One row of the tie rods should be positioned at mid-span for span length of a side rail exceeds 4m and two rows at one-third span points for span length exceeds 6m. The self weight of cladding may be carried by tie rods and transferred to the main columns or wall studs. Generally a pair of diagonal tie rods may be arranged at intervals of five lines of tie rods (as shown in Fig. 8. 4. 2) for transferring the self weight of the cladding in parts.

The diameter of tie rods should not be less than 10mm, the required sectional area shall be determined by calculation.

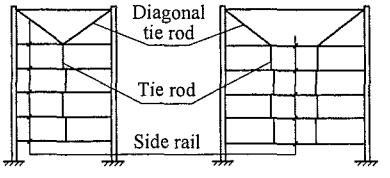



Fig. 8. 4. 2 Arrangement plan of tie rods

# 9 Roof Truss

## 9.1 Calculation of Roof Truss

**9.1.1** When calculating internal force of the members of roof truss, the truss joints are assumed as pinned joints and secondary stresses may not be taken into account, but the unfavorable effects such as wind suction loads on the roof, which may cause the truss member to change its internal force sign should be considered and the tensional resistance capacity of support anchors of roof truss shall be also checked.

**9.1.2** The effective length of the members of roof truss (as shown in Fig. 9.1.2) may be determined in accordance with following regulations:

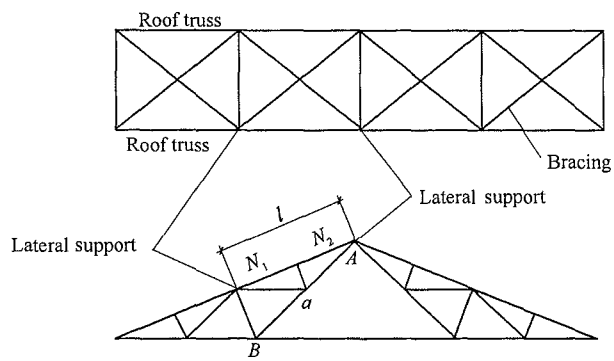



Fig. 9.1.2 Sketch for determining the effective length of roof truss members

**1** In roof truss plane, the effective length of the members may take the distance between panel joints;

**2** Out of roof truss plane, the chord members may take the distance between lateral supports; the web members may take the distance between panel joints (the web member a in Fig. 9.1.2 shall take the distance between $A$ and $B$); when the distance between lateral supports of compression chord or web member with equal panel joints is equal to two times the panel length and the two panels have unequal compression internal forces, the effective length shall be determined according to following formulae:

$$l_0=(0.75+0.25\frac{N_2}{N_1})l \qquad (9.1.2\text{-}1)$$

And

$$l_0\geqslant 0.5l \qquad (9.1.2\text{-}2)$$

Where $l_0$——Effective length of the member;

$l$——Distance between lateral supports of the member;

$N_1$——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 lateral supports of the roof truss are the points that can not move laterally (such as the points or joints of bracing). When the purlin, tie-rod or other members are not connected with the horizontal (or vertical) bracing joints or other points that cannot move laterally, in this case all of these points can not be treated as lateral supports.

## 9.2 Detailing of Requirements Roof Truss

**9.2.1** The arch camber of the roof truss should be given when the bottom chord of the truss is straight and the span is not less than 15m for simply supported triangular truss and 24m for the simply supported trapezoid or parallel-chord trusses. The depth of arch camber shall be 1/500 of span of the roof truss.

**9.2.2 The bracing system shall be arranged in roof system . When the round steel rods used as the bracing member, the adjusting tensioner for the steel rod must be installed.**

**9.2.3** The members of the roof truss should be thin-walled steel tube (such as square tube, rectangular tube or circular tube).

**9.2.4** In order to extend the length of truss member, the splice should be made by using welded or bolted connection. The splice should have the equal strength of the member to be connected and should be arranged in the panel point with the smaller internal force. The number and location of splice of the roof truss shall be determined in accordance with the construction and transportation conditions.

**9.2.5** The joint details of the roof truss should satisfy the following requirements:

**1** The centroidal axis of all joined members should intersect at the center of the joint;

**2** To strengthen the stiffness of the truss joints, the reinforcing plate or other measures should be added at the weaken positions;

**3** Be easy to weld, paint and clear up the dirt.

# 10 Rigid Frame

## 10.1 Calculations of Rigid Frame

**10.1.1** The strength and stability of beams and columns of rigid frame shall be checked as follows:

**1** The beams of rigid frame is allowed to be checked for in-plan strength solely as a member subjected to combined axial load and bending; the solid-web beams in rigid frame shall be checked for out-of-frame-plane stability as a member subjected to combined axial load and bending;

**2** The columns of solid-web frame shall be checked for strength and stability as a member subjected to combined axial load and bending;

**3** The columns of latticed rigid frame shall be checked for strength as well as for stability in bending plane as a member subjected to axial load and bending;

**4** The chord-members, web-members and lacing bars etc. of latticed beam-column shall be checked for strength and stability as a member subjected to axial tensile force or axial compressive force respectively;

**5** The stability of rigid frame columns with tapered member may be checked as for a uniform section member with an effective section equal to that subjected to maximum bending moment, while the axial force shall be taken as that acting at the same section where the maximum bending moment exists.

**10.1.2** The effective length $H_0$ of the column in a single-span gabled frame shall be determined as follows:

$$H_0=\mu H \tag{10.1.2-1}$$

Where $H$——The column's height taken as the distance from top surface of the foundation to the crossing point of member axes (as shown in Fig. 10.1.2);

$\mu$——Effective length factor of the column, taken as follows.

**1** For the rigid frame members with uniform section, $\mu$ may be taken as shown in Tables A.3.1 or A.3.2 in Appendix A;

**2** For rigid frame with tapered members, $\mu$ may be determined as follows:

$$\mu=\sqrt{\frac{24EI_1}{K\cdot H^3}} \tag{10.1.2-2}$$

$$K=\frac{1}{\Delta} \tag{10.1.2-3}$$

Where $K$——Side-sway stiffness of the frame under unit horizontal force applied on the column top;

$\Delta$——Lateral deflection of the frame at the column top calculated according to first order analysis under unit force applied on the column top;

$I_1$——Moment of inertia of the frame column larger end section.

**3** The effective length factor $\mu$ for above mentioned rigid frame is preferably multiplied by a regulation factor according to the details of the column base as follows:

0.85 for hinge connected base;

1.2 for rigidly connected base.

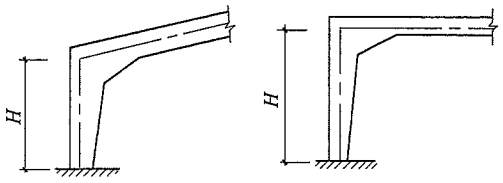



Fig. 10.1.2 Sketch of height of rigid frame column

**10.1.3** The effective length factor $\mu$ for exterior column of a multi-span gabled frame shall be calculated by the formula 10.1.2-1 with the effective length factor determined as follows:

**1** When the interior columns are hinged at both ends (i.e. being leaning column), the effective length factor $\mu_r$ of the exterior column shall be calculated by following formulae:

$$\mu_r=\eta\cdot\mu \tag{10.1.3-1}$$

$$\eta=\sqrt{1+\frac{\Sigma(N_{1i}/H_{1i})}{\Sigma(N_{fj}/H_{fj})}} \tag{10.1.3-2}$$

Where $\eta$——Amplification factor;

$\mu$——Effective length factor of columns of a single-span gabled frame calculated according to Clause 10.1.2;

$N_{1i}$——Axial force acting in the $i$ th interior leaning column;

$N_{fj}$——Axial force acting in the $j$ th exterior column;

$H_{1i}$——Height of the $i$ th interior leaning column;

$H_{fj}$——Height of the $j$ th exterior column.

The length of the rigid frame beam, used for consulting Tables A.3.1 or A.3.2 for $\mu$, shall be taken as twice of the beam's span (i.e. the distance from exterior column to the adjacent interior column).

The effective length of the leaning column shall be taken as 1.0.

**2** When the interior columns are non-leaning column, the effective length $\mu_i$ for all columns of the frame shall be calculated by following formulae:

$$\mu_i=\sqrt{\frac{1.2N_{Ei}}{K\cdot N_i}\cdot\Sigma\frac{N_i}{H_i}} \tag{10.1.3-3}$$

$$N_{Ei}=\frac{\pi^2 EI_i}{H_i^2} \qquad (10.1.3\text{-}4)$$

Where $\mu_i$——The effective length factor of $i$ th column of the frame, preferably to be multiplied by a regulation factor corresponding to Clause 10.1.2 item 3 according to details of the column base;

$N_{Ei}$——Euler's critical load for $i$ th column of the frame based on larger end section;

$H_i$、$N_i$——Height and axial compressive force respectively of $i$ th column of the frame;

$I_i$——Moment of inertia of larger end section of $i$ th column of the frame.

**10.1.4** The effective length of solid-web beam-columns out-of-frame-plane shall be taken as the distance between adjacent lateral supports. Lateral supports may be regarded as wherein diagonal bracing or bracing between columns attached. When the distance between adjacent lateral supports for both flanges of beam (or column) are different, the distance between lateral supports for flange subjected to maximum compressive force shall be taken into account.

**10.1.5** The effective length $l_0$ of chord and web members as well as lacing bars in latticed rigid frame (as shown in Fig. 10.1.4) shall be taken as follows:

**1** for in-plane stability the distance between adjacent joints;

**2** for out-of-plane stability, the distance between joints for web members and lacing bars; and the distance between lateral supports for chord members. If the internal force varied within the portion considered of the compressed chord, following stipulations shall be complied with:

1) When the internal forces are all compressive, it may be calculated by formulae 9.1.2-1 and 9.1.2-2, taken $N_1$ as largest compressive force, and $N_2$ as smallest one.

2) When the internal forces are compressive in several panels between the lateral supports considered and tensile in the rest panels, it is allowed to be calculated by the following formulae, however, the effective length of members shall not be less than total length of the compressed panels.

$$l_0=\left(1.5+0.5\frac{\overline{N}_t}{\overline{N}_c}\right)\cdot\frac{n_c}{n}\cdot l \qquad (10.1.5\text{-}1)$$

Furthermore $$l_0 \leqslant l \qquad (10.1.5\text{-}2)$$

Where $l$——Distance between lateral supports;

$\overline{N}_t$——Mean value of tensile forces, taken as negative in calculation;

$\overline{N}_c$——Mean value of compressive forces, taken as positive in calculation;

$n$——Number of panels between adjacent lateral supports;

$n_c$——Number of panels with internal force being compressive.

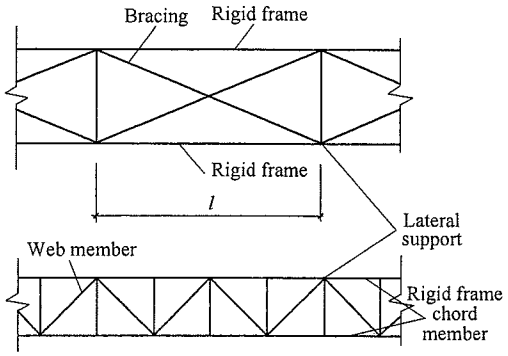



Fig. 10. 1. 5 Sketch of effective length for out-of-plane buckling of the chord members in latticed rigid frame

**10. 1. 6** The vertical deflection-to-span ratio of beams in rigid frame shall be preferably not greater than the limits listed in Table 10. 1. 6-2. The drift of columns in gabled frame under the action of characteristic value of wind load shall be preferably not greater than limits listed in Table 10. 1. 6-2, so as to ensure adequate stiffness of frame and normal service conditions of roofing and cladding.

**Table 10. 1. 6-1 Vertical Deflection Limit to Rigid Frame Beam**

| Load condition of roofing system | Deflection limit |
|---|---|
| Supporting roof panels and purlins only( with live load and snow load ) | $l/180$ |
| With additive ceiling | $l/240$ |
| With additive plaster ceiling | $l/360$ |
| Note: 1 For single-span gabled frames $l$ refers to the length of rafter on one of inclined sides of roof; for multiple-span gabled frames, $l$ refers to the length of rafter on one of inclined sides of roof between two adjacent columns;<br>2 For cantilever $l$ shall be taken in double length. | |

**Table 10. 1. 6-2 Lateral Drift Limit to Rigid Frame Column**

| Type of crane | Other conditions | Lateral drift limit on top of the column |
|---|---|---|
| Without crane | Metal cladding | $H/75$ |
| | Masonry wall | $H/100$ |
| With crane | Cab operated | H/400 |
| | Pendent operated | H/180 |
| Note: $H$refers to the height of the column. | | |

## 10. 2 Detailing Requirements of Rigid Frame

**10. 2. 1** The cold-formed thin-walled steel used for beams and columns of rigid frame shall have a thickness no less than 2mm.

**10. 2. 2** The minimum depth-to-span ratio of a frame girder shall be in the order of 1/15 to 1/25 for a latticed one and of 1/30 to 1/45 for a solid-web one.

**10. 2. 3 The gabled frame buildings should be arranged with bracing system. For a section of building between expansion points or a section of that constructed in a separated stage, the transverse bracing between upper flange(chord member) of the beams and inter-column bracing should be installed. At locations of frame corner parts (e. g. at eaves and ridges) as well as at top of certain interior columns in multi-span rigid frames, rigid members shall be arranged passing through the whole length in longitudinal direction of the buildings.**

**10. 2. 4** When lateral supports need to set at inner flange(or leg) of beams and columns of the rigid frames, the purlins and girts, served as lateral supports for outer flange (or leg) of the beams and columns, may be utilized to attach the diagonal brace (as shown in Fig. 10. 2. 4) for this purpose. The diagonal brace shall be checked as a compressive member.

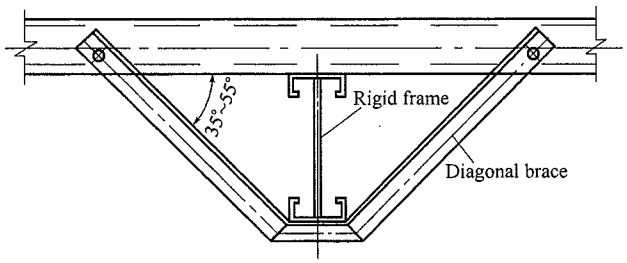



Fig. 10. 2. 4 Diagonal brace for beams and columns of rigid frame

**10. 2. 5** The beam of rigid frame shall be securely connected with purlins and other rigid members of the roof system.

# 11 Fabrication, Erection and Anticorrosion

## 11.1 Fabrication and Erection

**11.1.1** The members shall be protected from scoring. Layout and marking shall be made with a prescribed surplus according to the technological requirements for anticipated contraction induced by welding as well as certain surplus for cutting, trimming and chipping necessary during fabrication and erection.

**11.1.2** Precision of cutting in length and regularity of cut surface shall be ensured. Rust, dirt, contamination etc. shall be removed from the surface prior to cutting. Burr, flux slag, spatter etc. shall be removed after cutting.

**11.1.3** Straightening of the sections and members shall comply to the following requirements:

**1** Straightening of the sections shall conduct at ambient temperature with mechanical devices. When a square hollow section are used as main compressive cold-formed member in a structure, the local deformation measured on side of the section in longitudinal direction (as shown in Fig. 11.1.3) shall satisfy the stipulations expressed in following formula:

$$\delta \leqslant 0.01b \tag{11.1.3}$$

Where $\delta$——Local deflection measured in longitudinal direction;

$b$——Gauge length for measuring the local deformation, taken as the width of the section side face where the deformation occurs.

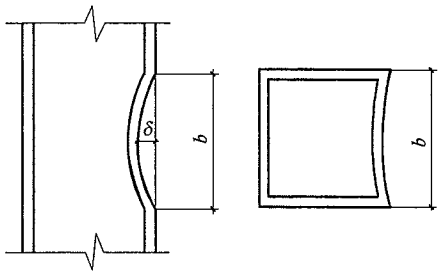



Fig. 11.1.3 Sketch of view for gauge length measurement

**2** Cold straightening and cold bending shall not be performed for the pieces of carbon steel at ambient temperature lower than −16℃ and for the pieces of low alloyed steel at ambient temperature lower than −12℃.

**3** Heating of carbon structural steel and low alloyed structural steel shall be performed up to a temperature of 900℃ according to the properties of the steel related.

Heat straightened low alloyed structural steel shall be cooled gradually in a natural way;

**4** After straightening, warping rise of arch should not exceed 1/1000 of member's length and should not be more than 10mm.

**11.1.4** The bolt hole in members shall be made in compliance with the following requirements:

**1** The hole for high strength bolts shall be made by drilling.

**2** Burr, crack, flare, uneven, scrap, etc. around the bolt hole shall be removed.

**11.1.5** Assembly and field splice of the members shall be completed as follows:

**1** Assembly of the members shall be performed on suitable platform and assembly jig. The platform and jig must be leveled with the structural members clamped, so that the central line of members placed on a same level with the deviation not exceeding 3mm.

**2** Dimensions of the structure shall be strictly controlled in accordance with the shop drawing. Working line of the members shall be intersected at the center of joints where the central lines pass through. The deviation between the working-line intersection and the center of joints shall not exceeding 3mm.

**3** Dimensions of the members shall keep surplus taking account for contraction induced by welding as well as shortening by other reasons during assembling. For members in need of cambering, the amount of rise shall be prescribed and made up in advance. Tolerance for deviation of the rise shall not exceed 1/1000 of the member length and 6mm simultaneously.

**4** Members shall be protected from distortion. The deviation of central line on surface of the member during assembling shall not exceed 3mm.

**5** Offset in the weld and that in the joint for lap and butt joints shall not exceed 0.5mm.

**6** Tack weld shall be placed where they will be incorporated into a final weld. Overwelding causing burning through of the workpeice is not allowed. The electrode in sense of metal and flux used for the tack weld shall be the same as that used in the final weld.

**7** The deviation in location of central line of the hole where connection with other members required shall not exceed 2mm.

**11.1.6** Cold-formed thin-walled steel shall be welded in compliance with the following provisions:

**1** The features of cold-formed thin-walled steel, the welding methods required by welding technology, the welding sequence and the technical measurements shall be well familiar with prior to welding. Prescribed parameters related to welding shall be verified by tests and adopted accordingly, so as to ensure the welding quality.

2 Prior to welding, rust, dirt, standing water etc. shall be removed from surface of the pieces to be welded. Electrodes and flux shall be baked out in advance.

3 The start and extinction of arc for butt weld or closed sectional weld shall not be overlapped at a same location. Arc shall be extinguished in certain distance beyond the location where the arc starting. Moreover, it shall not be allowed to start or extinct arc on non-welding position of the base metal.

4 After welding completed, flux slag and spatter on two sides shall be removed from the surface of the weld. The quality in appearance of weld shall also be inspected.

5 Presetting joints before welding may be considered to minimize deformation induced.

6 Proper measures according to the specific conditions must be taken (e. g. leaving space in advance, one side welding with backing, two sides welding) to ensure full penetration of the butt weld.

7 Selection of related technological parameters for the resistance spot welding (e. g. galvanizing duration, welding current, pressure on electrode, etc. ) shall be aimed at ensuring strength of the welding spot tested up to standard. All parameters related shall remain relatively stable during welding. Welded pieces shall be closely jointed at their contacting surface.

8 Resistance spot welding shall be made preferably with a conic head electrode, no less than $5\sqrt{t}$ in diameter ( $t$ refers to the thickness of thinner one of the plates welded). In the process of welding, diameter of the electrode shall not be changed in an extent of over 1/5 of it.

**11.1.7** Members of cold-formed thin-walled steel structure shall be packed after the painted coat having dried. Care shall be taken during packing so as to protect the painted coat from damage. Moreover, members shall handled with care to avoid distortion, harm and loosening in the process of transport, loading, unloading and storage.

**11.1.8** Cold-formed thin-walled steel structure shall be installed according to the following stipulations:

1 Quality check shall be performed prior to installation of the structure. Deformation, and defects exceeding the tolerance of standard, if any, shall be properly eliminated.

2 Proper measures shall be adopted to avoid permanent deformation of the structure during hoisting. The hoisted member shall be well cushioned beneath where the cable upholding.

3 It is not allowed to take advantage of installed members for hoisting other weights or to attach foreign pieces to mainly stressed members of the structure.

4 Prior to installation of the roof panels, effective measures shall be taken to en-

sure tightening of the sag tie and place the purlins correctly.

**5** During installation of roof panels, effective measures shall be taken to distribute construction load on larger roof area to prevent members from local buckling under concentrated load.

**11.1.9** Fabrication and installation of cold-formed thin-walled steel structure shall adhere to the provisions of the present code. Besides, provisions in current national standard "Code for Acceptance of Construction Quality of Steel Structure"GB 50205 shall also be satisfied. For construction work in regard to spaying fire resisting coats provisions of current national standard "Fire Resistive Coating for Steel Structure"GB14907 shall be complied with.

## 11.2 Anticorrosion

**11.2.1** Effective anticorrosive measures for cold-formed thin-walled steel structure must be adopted, easy to inspect, easy to surface cleaning and easy to painting shall be taken into account in detailing. Closed section shall be welded airtight in longitudinal direction and at both ends.

**11.2.2** Proper methods for the surface processing and anticorrosive measures shall be selected according to the service condition and ambient environment for the cold-formed thin-walled steel structure.

Classification of the corrosive action on cold-formed thin-walled steel structure can be referred to Table D.0.1 in Appendix D.

**11.2.3** Surface processing of cold-formed thin-walled steel structure shall be performed according to the design requirements. Methods for antirust and its gradation shall comply to the current national standard "Rust Grades and Preparation Grades of Steel Surfaces Before Application of Paints and Related Products"GB/T 8923.

**11.2.4** When a chemical antirust method is adopted for cold-formed thin-walled steel structure, a processing liquid having two or more functions, such as antirust, phosphorating and chromating, shall be selected. Its quality shall comply to the provisions of present national standard "General Specification for Multifunctional Solution of Iron and Steel Surface Treatment"GB/T 12612.

**11.2.5** For cold-formed thin-walled steel structure, related anticorrosion measures appropriate to following different conditions shall be selected as follows:

**1** Metal protection layer coating (surface alloyed galvanization, zinc-aluminum alloy coating, etc.);

**2** Anticorrosive painting:

1) In non-corrosive or weak corrosive condition, oil paints, phenolic paints and alkyd paints are suitably to use;

2) In moderate corrosive condition, epoxy paints, epoxy resin paints, perchloroethylene paints, chlorinated rubber paints and chloro-acetate paints are preferably to use;

3) Primer and finish coat of anticorrosive paints shall be compatible with each other.

**3** Composite protection:

1) For members fabricated with aluminum-zinc alloy coated sheet, degreasing, phosphorating and chromating (or phosphorating primer after degreasing) shall be done prior to painting;

2) For panels made with surface alloyed galvanized or galvanized steel plate (such as profiled steel sheeting, corrugated sheet etc.) the red lead antirust primer will not be suitable, instead, the H06-2 zinc yellow epoxy resin primer or other special paints are preferably to apply.

**11.2.6** The painting and finishing material adopted for cold-formed thin-walled steel structure shall be provided with certificate of maker and comply with the design requirements. Coating may be applied by hand brushing or mechanical spray apart from those regulated in design.

**11.2.7** Paints, number of coating layers and thickness of coating layers shall comply with the design requirements. When design documents haven't offer any definite regulations for painting and finishing, as a rule it shall apply 4～5 times and total thickness of dry film should be no less than 150$\mu$m for members outdoor and no less than 120$\mu$m for members indoor, with the tolerance of deviation of $\pm 25\mu$m.

**11.2.8** The ambient temperature and the relative humidity during painting should comply with the requirements of product specification. When no special requirements set forth, the ambient temperature shall be taken as 5～38℃, and the relative humidity should not be taken over 85%. Paints shall not be applied when moisture condensation present on the member surface. Rainfall should be avoided up to 4h after having painting and finish coats completed.

**11.2.9** Quality in appearance of painting and finish coat applied on cold-formed thin-walled steel structure should be uniform, delicate, no obvious chromatic difference, no downflow, no chill, no wrinkle, no pinhole, no bubble, no crack, no peeling off, no dirt to adhere, no neglect in painting etc. and should be well adhered ( to be checked by scratching method or cohesiometer). After painting film become dry enough, a dry film thickness meter to be used to measure thickness of the drying film and keep a record. Those which does not complied with the regulation in thickness should be painted supplementary. Those having painting and finish coat not met quality requirements at all should be processed again.

**11. 2. 10** Anticorrosive processing of cold-formed thin-walled steel structure shall comply with the following requirements:

**1** Anticorrosive paints shall be applied timely within 6h after completion of the surface processing in order to prevent from rusting again.

**2** Location on surface where no painting required according to the shop drawing, should not be painted. Nearby field welds 30～50mm around shall be left uncoated temporarily.

**3** After cold-formed thin-walled steel structure having been installed in place, location where painting coats wear down during transport and hoisting or remain uncoated nearby field welds shall be applied with paints supplementary to ensure a level of surface protection as that for the adjacent parts.

**4** Parts of cold-formed thin-walled steel structure to be encased with or embedded into concrete may not be painted or coated.

**5** For members easily subjected to moisture or standing water and difficult in maintenance in sense of newly painting, measures should be taken to seal.

**11. 2. 11** Termly inspection and maintenance for cold-formed thin-walled steel structure during the course of service are necessary. Duration limit of maintenance shall be determined according to Table D. 0. 2 in Appendix D in dependence on the service condition, surface processing method, types of paints adopted and thickness of paint film for the structure.

**11. 2. 12** Quality of newly applied painting on cold-formed thin-walled steel structure shall comply with the stipulations of current national standard "Code for Acceptance of Construction Quality of Steel Structure"GB 50205.

# Appendix A Coefficients of Calculation

## A. 1 Stability Factor of Axial Compression Members

**A. 1. 1** Stability factor of axial compression members may be taken from following tables in accordance with the grade of steel.

**Table A. 1. 1-1 Stability Factor $\varphi$ of Axial Compression Members for Q235 Steel**

| $\lambda$ | 0 | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 |
|---|---|---|---|---|---|---|---|---|---|---|
| 0 | 1. 000 | 0. 997 | 0. 995 | 0. 992 | 0. 989 | 0. 987 | 0. 984 | 0. 981 | 0. 979 | 0. 976 |
| 10 | 0. 974 | 0. 971 | 0. 968 | 0. 966 | 0. 963 | 0. 960 | 0. 958 | 0. 955 | 0. 952 | 0. 949 |
| 20 | 0. 947 | 0. 944 | 0. 941 | 0. 938 | 0. 936 | 0. 933 | 0. 930 | 0. 927 | 0. 924 | 0. 921 |
| 30 | 0. 918 | 0. 915 | 0. 912 | 0. 909 | 0. 906 | 0. 903 | 0. 899 | 0. 896 | 0. 893 | 0. 889 |
| 40 | 0. 886 | 0. 882 | 0. 879 | 0. 875 | 0. 872 | 0. 868 | 0. 864 | 0. 861 | 0. 858 | 0. 855 |
| 50 | 0. 852 | 0. 849 | 0. 846 | 0. 843 | 0. 839 | 0. 836 | 0. 832 | 0. 829 | 0. 825 | 0. 822 |
| 60 | 0. 818 | 0. 814 | 0. 810 | 0. 806 | 0. 802 | 0. 797 | 0. 793 | 0. 789 | 0. 784 | 0. 779 |
| 70 | 0. 775 | 0. 770 | 0. 765 | 0. 760 | 0. 755 | 0. 750 | 0. 744 | 0. 739 | 0. 733 | 0. 728 |
| 80 | 0. 722 | 0. 716 | 0. 710 | 0. 704 | 0. 698 | 0. 692 | 0. 686 | 0. 680 | 0. 673 | 0. 667 |
| 90 | 0. 661 | 0. 654 | 0. 648 | 0. 641 | 0. 634 | 0. 626 | 0. 618 | 0. 611 | 0. 603 | 0. 595 |
| 100 | 0. 588 | 0. 580 | 0. 573 | 0. 566 | 0. 558 | 0. 551 | 0. 544 | 0. 537 | 0. 530 | 0. 523 |
| 110 | 0. 516 | 0. 509 | 0. 502 | 0. 496 | 0. 489 | 0. 483 | 0. 476 | 0470 | 0. 464 | 0. 458 |
| 120 | 0. 452 | 0. 446 | 0. 440 | 0. 434 | 0. 428 | 0. 423 | 0. 417 | 0. 412 | 0. 406 | 0. 401 |
| 130 | 0. 396 | 0. 391 | 0. 386 | 0. 381 | 0. 376 | 0. 371 | 0. 367 | 0. 362 | 0. 357 | 0. 353 |
| 140 | 0. 349 | 0. 344 | 0. 340 | 0. 336 | 0. 332 | 0. 328 | 0. 324 | 0. 320 | 0. 316 | 0. 312 |
| 150 | 0. 308 | 0. 305 | 0. 301 | 0. 298 | 0. 294 | 0. 291 | 0. 287 | 0. 284 | 0. 281 | 0. 277 |
| 160 | 0. 274 | 0. 271 | 0. 268 | 0. 265 | 0. 262 | 0. 259 | 0. 256 | 0. 253 | 0. 251 | 0. 248 |
| 170 | 0. 245 | 0. 243 | 0. 240 | 0. 237 | 0. 235 | 0. 232 | 0. 230 | 0. 227 | 0. 225 | 0. 223 |
| 180 | 0. 220 | 0. 218 | 0. 216 | 0. 214 | 0. 211 | 0. 209 | 0. 207 | 0. 205 | 0. 203 | 0. 201 |
| 190 | 0. 199 | 0. 197 | 0. 195 | 0. 193 | 0. 191 | 0. 189 | 0. 188 | 0. 186 | 0. 184 | 0. 182 |
| 200 | 0. 180 | 0. 179 | 0. 177 | 0. 175 | 0. 174 | 0. 172 | 0. 171 | 0. 169 | 0. 167 | 0. 166 |
| 210 | 0. 164 | 0. 163 | 0. 161 | 0. 160 | 0. 159 | 0. 157 | 0. 156 | 0. 154 | 0. 153 | 0. 152 |
| 220 | 0. 150 | 0. 149 | 0. 148 | 0. 146 | 0. 145 | 0. 144 | 0. 143 | 0. 141 | 0. 140 | 0. 139 |
| 230 | 0. 138 | 0. 137 | 0. 136 | 0. 135 | 0. 133 | 0. 132 | 0. 131 | 0. 130 | 0. 129 | 0. 128 |
| 240 | 0. 127 | 0. 126 | 0. 125 | 0. 124 | 0. 123 | 0. 122 | 0. 121 | 0. 120 | 0. 119 | 0. 118 |
| 250 | 0. 117 | — | — | — | — | — | — | — | — | — |

**Table A. 1. 1-2 Stability Factor $\varphi$ of Axial Compression Members for Q345 Steel**

| $\lambda$ | 0 | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 |
|---|---|---|---|---|---|---|---|---|---|---|
| 0 | 1.000 | 0.997 | 0.994 | 0.991 | 0.988 | 0.985 | 0.982 | 0.979 | 0.976 | 0.973 |
| 10 | 0.971 | 0.968 | 0.965 | 0.962 | 0.959 | 0.956 | 0.952 | 0.949 | 0.946 | 0.943 |
| 20 | 0.940 | 0.937 | 0.934 | 0.930 | 0.927 | 0.924 | 0.920 | 0.917 | 0.913 | 0.909 |
| 30 | 0.906 | 0.902 | 0.898 | 0.894 | 0.890 | 0.886 | 0.882 | 0.878 | 0.874 | 0.870 |
| 40 | 0.867 | 0.864 | 0.860 | 0.857 | 0.853 | 0.849 | 0.845 | 0.841 | 0.837 | 0.833 |
| 50 | 0.829 | 0.824 | 0.819 | 0.815 | 0.810 | 0.805 | 0.800 | 0.794 | 0.789 | 0.783 |
| 60 | 0.777 | 0.771 | 0.765 | 0.759 | 0.752 | 0.746 | 0.739 | 0.732 | 0.725 | 0.718 |
| 70 | 0.710 | 0.703 | 0.695 | 0.688 | 0.680 | 0.672 | 0.664 | 0.656 | 0.648 | 0.640 |
| 80 | 0.632 | 0.623 | 0.615 | 0.607 | 0.599 | 0.591 | 0.583 | 0.574 | 0.566 | 0.558 |
| 90 | 0.550 | 0.542 | 0.535 | 0.527 | 0.519 | 0.512 | 0.504 | 0.497 | 0.489 | 0.482 |
| 100 | 0.475 | 0.467 | 0.460 | 0.452 | 0.445 | 0.438 | 0.431 | 0.424 | 0.418 | 0.411 |
| 110 | 0.405 | 0.398 | 0.392 | 0.386 | 0.380 | 0.375 | 0.369 | 0.363 | 0.358 | 0.352 |
| 120 | 0.347 | 0.342 | 0.337 | 0.332 | 0.327 | 0.322 | 0.318 | 0.313 | 0.309 | 0.304 |
| 130 | 0.300 | 0.296 | 0.292 | 0.288 | 0.284 | 0.280 | 0.276 | 0.272 | 0.269 | 0.265 |
| 140 | 0.261 | 0.258 | 0.255 | 0.251 | 0.248 | 0.245 | 0.242 | 0.238 | 0.235 | 0.232 |
| 150 | 0.229 | 0.227 | 0.224 | 0.221 | 0.218 | 0.216 | 0.213 | 0.210 | 0.208 | 0.205 |
| 160 | 0.203 | 0.201 | 0.198 | 0.196 | 0.194 | 0.191 | 0.189 | 0.187 | 0.185 | 0.183 |
| 170 | 0.181 | 0.179 | 0.177 | 0.175 | 0.173 | 0.171 | 0.169 | 0.167 | 0.165 | 0.163 |
| 180 | 0.162 | 0.160 | 0.158 | 0.157 | 0.155 | 0.153 | 0.152 | 0.150 | 0.149 | 0.147 |
| 190 | 0.146 | 0.144 | 0.143 | 0.141 | 0.140 | 0.138 | 0.137 | 0.136 | 0.134 | 0.133 |
| 200 | 0.132 | 0.130 | 0.129 | 0.128 | 0.127 | 0.126 | 0.124 | 0.123 | 0.122 | 0.121 |
| 210 | 0.120 | 0.119 | 0.118 | 0.116 | 0.115 | 0.114 | 0.113 | 0.112 | 0.111 | 0.110 |
| 220 | 0.109 | 0.108 | 0.107 | 0.106 | 0.106 | 0.105 | 0.104 | 0.103 | 0.101 | 0.101 |
| 230 | 0.100 | 0.099 | 0.098 | 0.098 | 0.097 | 0.096 | 0.095 | 0.094 | 0.094 | 0.093 |
| 240 | 0.092 | 0.091 | 0.091 | 0.090 | 0.089 | 0.088 | 0.088 | 0.087 | 0.086 | 0.086 |
| 250 | 0.085 | — | — | — | — | — | — | — | — | — |

## A. 2 Overall Stability Factor of Flexural Members

**A. 2. 1** For simply supported beams of singly- or doubly-symmetric sections (including anti-symmetric sections) as shown in Fig. 5. 3. 1 while bent about the symmetric axis ($x$-axis), their overall stability factor shall be calculated by following formulae:

$$\varphi_{bx}=\frac{4320Ah}{\lambda_y^2 W_x}\xi_1\left(\sqrt{\eta^2+\zeta}+\eta\right)\cdot\frac{235}{f_y} \tag{A. 2. 1-1}$$

With

$$\eta=2\xi_2 e_a/h \tag{A. 2. 1-2}$$

$$\zeta=\frac{4I_\omega}{h^2 I_y}+\frac{0.156I_t}{I_y}\left(\frac{l_0}{h}\right)^2 \tag{A. 2. 1-3}$$

Where $\lambda_y$——Slenderness ratio of beams buckling the out-of-plane of bending;

$A$——Gross sectional area;

$h$——Depth of section;

$l_0$——Lateral effective length of beams, $l_0=\mu_b l$;

$\mu_b$——Factor of lateral effective length of the beams obtained from Table A. 2. 1;

$l$——Span length of the beams;

$\xi_1$、$\xi_2$——Coefficient obtained from Table A. 2. 1;

$e_a$——Distance from the transverse loading point to the shear center, for eccentric compression members and transverse load passing through the shear center $e_a=0$; while a transverse load is not located at the shear center, $e_a$ is negative for load oriented to the shear center and positive for load departed from the shear center;

$W_x$——Gross section modulus about the $x$-axis with respect to the extreme compression fiber;

$I_\omega$——Sectorial moment of inertia of gross section;

$I_y$——Moment of inertia of gross section about $y$-axis;

$I_t$——Torsional moment of inertia of gross section.

When the values of $\varphi_{bx}$ calculated from above formulae is larger than 0. 7, they shall be replaced by the corresponding $\varphi'_{bx}$ given by following formula:

$$\varphi'_{bx}=1.091-\frac{0.274}{\varphi_{bx}} \qquad \text{(A. 2. 1-4)}$$

**Table A. 2. 1 Values of $\xi_1$、$\xi_2$ and $\mu_b$ for Laterally Simply Supported Beams at Two Ends and in the Span**

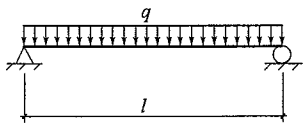
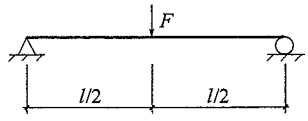
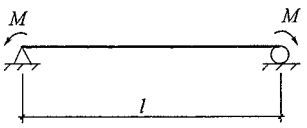
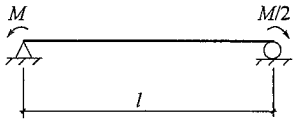
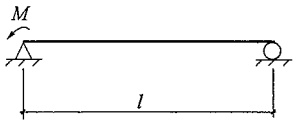
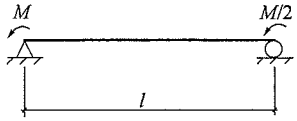
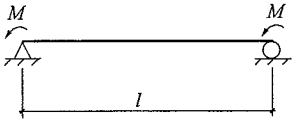
| Item No. | Load and supporting situations in the plane of bending | No intermediate lateral bracing in the span | | One lateral bracing at mid-span | | More than two equally spaced lateral bracing in the span | |
|---|---|---|---|---|---|---|---|
| | | $\mu_b=1.00$ | | $\mu_b=0.50$ | | $\mu_b=0.33$ | |
| | | $\xi_1$ | $\xi_2$ | $\xi_1$ | $\xi_2$ | $\xi_1$ | $\xi_2$ |
| 1 |  | 1. 13 | 0. 46 | 1. 35 | 0. 14 | 1. 37 | 0. 06 |
| 2 |  | 1. 35 | 0. 55 | 1. 83 | 0 | 1. 68 | 0. 08 |
| 3 |  | 1. 00 | 0 | 1. 00 | 0 | 1. 00 | 0 |

**Table A. 2. 1 (Continued)**

| Item No. | Load and supporting situations in the plane of bending | No intermediate lateral bracing in the span | | One lateral bracing at mid-span | | More than two equally spaced lateral bracing in the span | |
|---|---|---|---|---|---|---|---|
| | | $\mu_b$=1.00 | | $\mu_b$=0.50 | | $\mu_b$=0.33 | |
| | | $\xi_1$ | $\xi_2$ | $\xi_1$ | $\xi_2$ | $\xi_1$ | $\xi_2$ |
| 4 |  | 1.32 | 0 | 1.31 | 0 | 1.31 | 0 |
| 5 |  | 1.83 | 0 | 1.77 | 0 | 1.75 | 0 |
| 6 |  | 2.39 | 0 | 2.13 | 0 | 2.03 | 0 |
| 7 |  | 2.24 | 0 | 1.89 | 0 | 1.77 | 0 |

**A. 2. 2** For simply supported beams of singly-symmetric section, in which $x$-axis (strong axis) is a nonsymmetric axis as shown in Fig. A. 2. 2, while they bent about $x$-axis, then their overall stability factor may also be calculated from Eq. A. 2. 1-1, but Eq. A. 2. 1-2 need to substituted by following formulae:

$$\eta=2(\xi_2 e_a+\beta_y)/h \tag{A. 2. 2-1}$$

With

$$\beta_y=\frac{U_x}{2I_x}-e_{0y} \tag{A. 2. 2-2}$$

$$U_x=\int_A y(x^2+y^2)\,dA \tag{A. 2. 2-3}$$

Where $I_x$——Moment of inertia of gross section bent about $x$-axis;

$e_{0y}$——Coordinate of shear center in $y$-axis.

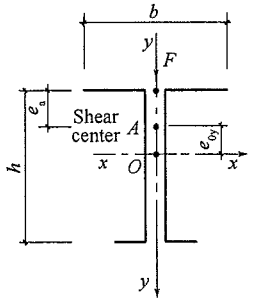



Fig. A. 2. 2 Singly-symmetric section

**A. 2. 3** For simply supported beams of singly- or doubly-symmetric section (as shown in Fig. 5. 3. 1), while they bent about $y$-axis (weak axis), if necessary, their overall stability factor $\varphi_{by}$ may be determined as follows:

$$\varphi_{by}=\frac{4320Ab}{\lambda_x^2 W_y}\xi_1\left(\sqrt{\eta^2+\zeta}+\eta\right)\left(\frac{235}{f_y}\right) \qquad \text{(A. 2. 3-1)}$$

With

$$\eta=2(\xi_2 e_a 9+\beta_x)/b \qquad \text{(A. 2. 3-2)}$$

$$\zeta=\frac{4I_\omega}{b^2 I_x}+\frac{0.156I_t}{I_x}\left(\frac{l_0}{b}\right)^2 \qquad \text{(A. 2. 3-3)}$$

For $y$-axis is a symmetric axis:

$$\beta_x = 0$$

For $y$-axis is a nonsymmetric axis:

$$\beta_x = \frac{U_y}{2I_y} - e_{0x} \qquad \text{(A. 2. 3-4)}$$

$$U_y = \int_A x(x^2+y^2)\,dA \qquad \text{(A. 2. 3-5)}$$

Where $b$——Width of section;

$\lambda_x$——Out-of-plane slenderness ratio of beams (about $x$-axis);

$W_y$——Gross section modulus about $y$-axis with respect to the extreme compression fiber;

$e_{0x}$——Coordinate of shear center in $x$-axis.

When $\varphi_{by}>0.7$, $\varphi_{by}$ shall be substituted by $\varphi'_{by}$ calculated from the following equation:

$$\varphi'_{by}=1.091-\frac{0.274}{\varphi_{by}} \qquad \text{(A. 2. 3-6)}$$

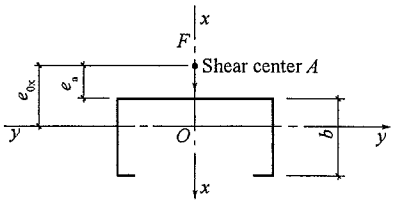



Fig. A. 2. 3 Lipped channel of singly-symmetric

## A. 3 Effective Length Factor for Frame Columns

**A. 3. 1** The effective length factor $\mu$ of uniform section framed columns may be given in Table A. 3. 1.

**Table. A. 3. 1 Effective length factor $\mu$ of uniform section frame columns**

| Types of column connected to the foundation \ $K_2/K_1$ | 0 | 0.2 | 0.3 | 0.5 | 1.0 | 2.0 | 3.0 | 4.0 | 7.0 | ≥10.0 |
|---|---|---|---|---|---|---|---|---|---|---|
| Rigidly fixed | 2.00 | 1.50 | 1.40 | 1.28 | 1.16 | 1.08 | 1.06 | 1.04 | 1.02 | 1.00 |

**Table. A. 3. 1 (Continued)**

| Types of column connected to the foundation \ $K_2/K_1$ | 0 | 0.2 | 0.3 | 0.5 | 1.0 | 2.0 | 3.0 | 4.0 | 7.0 | ≥10.0 |
|---|---|---|---|---|---|---|---|---|---|---|
| Hinged | ∞ | 3.42 | 3.00 | 2.63 | 2.33 | 2.17 | 2.11 | 2.08 | 2.05 | 2.00 |

Note: 1 $K_1=I_1/H, K_2=I_2/l$;

2 $I_1$ is the moment of inertia of section at the top of the column;
$I_2$ is the moment of inertia of the frame girder section;
$H$ is the height of the frame column;
$l$ is the length of the frame girder, taken as the developed slant length of the folded girder for gabled portal frame;

3 Take $K_2=0$ for column hinged to the girder.

**A. 3. 2** The effective length factor $\mu$ of the variable section frame columns may be taken from Table A. 3. 2.

**Table A. 3. 2 Effective Length Factor $\mu$ for Variable Section Frame Columns**

| Types connected column to the foundation | $I_0/I_1$ \ $K_2/K_1$ | 0.1 | 0.2 | 0.3 | 0.5 | 0.75 | 1.0 | 2.0 | ≥10.0 |
|---|---|---|---|---|---|---|---|---|---|
| Hinged | 0.01 | 5.03 | 4.33 | 4.10 | 3.89 | 3.77 | 3.74 | 3.70 | 3.65 |
| | 0.05 | 4.90 | 3.98 | 3.65 | 3.39 | 3.25 | 3.19 | 3.10 | 3.05 |
| | 0.10 | 4.66 | 3.82 | 3.48 | 3.19 | 3.04 | 2.98 | 2.94 | 2.75 |
| | 0.15 | 4.61 | 3.75 | 3.37 | 3.10 | 2.93 | 2.85 | 2.72 | 2.65 |
| | ≥0.20 | 4.59 | 3.67 | 3.30 | 3.00 | 2.84 | 2.75 | 2.63 | 2.55 |

Note: $I_0$ is the moment of inertia of section at the bottom of the column.

## A. 4 Calculation of Bimoment *B* for Simply Supported Beams

**A. 4. 1** The bimoment $B$ of the simply supported beams may be calculated by formulae given in Table A. 4. 1 according to the loading situation.

**Table A. 4. 1 Formulae of Computing Bimoment *B* of Simply Supported Beams**

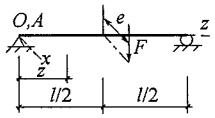
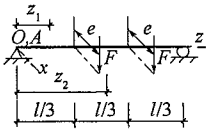
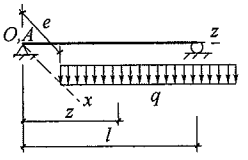
| Item No. | Ⅰ | Ⅱ | Ⅲ |
|---|---|---|---|
| Loading scheme |  |  |  |
| $B$ (at any point) | $\frac{F\cdot e}{2k}\cdot\frac{\mathrm{sh}kz}{\mathrm{ch}\frac{kl}{2}}$ | When $z=z_1$, $\frac{F\cdot e}{k}\cdot\frac{\mathrm{ch}\frac{kl}{6}}{\mathrm{ch}\frac{kl}{2}}\cdot\mathrm{sh}kz_1$<br>When $z=z_2$, $\frac{F\cdot e}{k}\cdot\frac{\mathrm{sh}\frac{kl}{3}}{\mathrm{ch}\frac{kl}{2}}\cdot\mathrm{ch}k\left(\frac{l}{2}-z_2\right)$ | $\frac{q\cdot e}{k^2}\left[1-\frac{\mathrm{ch}k\left(\frac{l}{2}-z\right)}{\mathrm{ch}\frac{kl}{2}}\right]$ |

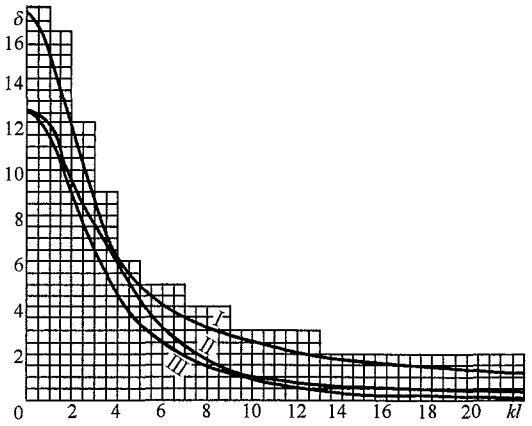
**Table A. 4. 1 (Continued)**

| Item No. | Ⅰ | Ⅱ | Ⅲ |
|---|---|---|---|
| $B_{max}$ (at mid-span) | $0.02\delta \cdot F \cdot e \cdot l$ | $0.02\delta \cdot F \cdot e \cdot l$ | $0.01\delta \cdot q \cdot e \cdot l^2$ |

Note: $k$——Torsional-flexural characteristic coefficient, $k=\sqrt{GI_t/EI_\omega}$;

$G$——Shear modulus of steel, $G=0.79\times10^5 N/mm^2$;

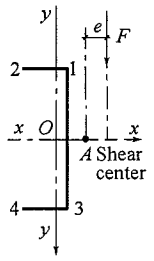
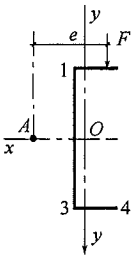
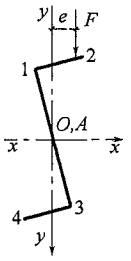
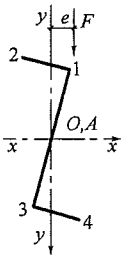
$\delta$——Coefficient of computing $B_{max}$, may be obtained from following diagram.




$\delta$—$kl$ diagram

**A. 4. 2** The sign of normal stresses due to bimoment $B$ may be determined according to Table A. 4. 2.

**Table A. 4. 2 Sign of Normal Stresses Due to Bimoment *B***

| Scheme of load and section / Points in the section |  |  |  |  |
|---|---|---|---|---|
| 1 | − | + | + | − |
| 2 | + | − | − | + |
| 3 | + | − | + | − |
| 4 | − | + | − | + |









Note: 1 The sign of normal stresses in this table, "+" represents compressive stress, while "−" represents tensile stress;

2 In this table the applied load $F$ produces a clockwise rotation around the shear center $A$ of section, otherwise if the outer applied load $F$ produces a counter clockwise rotation around the shear center $A$, then all signs of normal stresses in this table shall be reversed.

# Appendix B Section Characteristics

## B. 1 Table of Characteristics of Commonly Used Sections

**B. 1. 1** The characteristics of commonly used sections is list in Table B. 1. 1-1～ Table B. 1. 1-8.

**Table B. 1. 1-1 Square Tube of Steel**

| Dimension (mm) | | Area of section ($cm^2$) | Mass per meter (kg/m) | $I_x$ ($cm^4$) | $i_x$ (cm) | $W_x$ ($cm^3$) |
|---|---|---|---|---|---|---|
| $h$ | $t$ | | | | | |
| 25 | 1.5 | 1.31 | 1.03 | 1.16 | 0.94 | 0.92 |
| 30 | 1.5 | 1.61 | 1.27 | 2.11 | 1.14 | 1.40 |
| 40 | 1.5 | 2.21 | 1.74 | 5.33 | 1.55 | 2.67 |
| 40 | 2.0 | 2.87 | 2.25 | 6.66 | 1.52 | 3.33 |
| 50 | 1.5 | 2.81 | 2.21 | 10.82 | 1.96 | 4.33 |
| 50 | 2.0 | 3.67 | 2.88 | 13.71 | 1.93 | 5.48 |
| 60 | 2.0 | 4.47 | 3.51 | 24.51 | 2.34 | 8.17 |
| 60 | 2.5 | 5.48 | 4.30 | 29.36 | 2.31 | 9.79 |
| 80 | 2.0 | 6.07 | 4.76 | 60.58 | 3.16 | 15.15 |
| 80 | 2.5 | 7.48 | 5.87 | 73.40 | 3.13 | 18.35 |
| 100 | 2.5 | 9.48 | 7.44 | 147.91 | 3.95 | 29.58 |
| 100 | 3.0 | 11.25 | 8.83 | 173.12 | 3.92 | 34.62 |
| 120 | 2.5 | 11.48 | 9.01 | 260.88 | 4.77 | 43.48 |
| 120 | 3.0 | 13.65 | 10.72 | 306.71 | 4.74 | 51.12 |
| 140 | 3.0 | 16.05 | 12.60 | 495.68 | 5.56 | 70.81 |
| 140 | 3.5 | 18.58 | 14.59 | 568.22 | 5.53 | 81.17 |
| 140 | 4.0 | 21.07 | 16.44 | 637.97 | 5.50 | 91.14 |
| 160 | 3.0 | 18.45 | 14.49 | 749.64 | 6.37 | 93.71 |
| 160 | 3.5 | 21.38 | 16.77 | 861.34 | 6.35 | 107.67 |
| 160 | 4.0 | 24.27 | 19.05 | 969.35 | 6.32 | 121.17 |
| 160 | 4.5 | 27.12 | 21.05 | 1073.66 | 6.29 | 134.21 |
| 160 | 5.0 | 29.93 | 23.35 | 1174.44 | 6.26 | 146.81 |

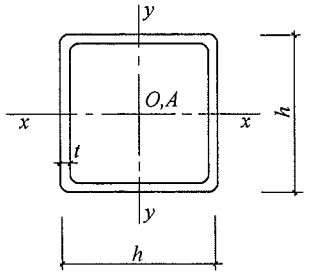


**Table B. 1. 1-2 Equal-leg Angle Steel**

| Dimension (mm) | | Area of section ($cm^2$) | Mass per meter (kg/m) | $y_0$ (cm) | $x_0-x_0$ | | | | $x-x$ | | $y-y$ | | $x_1-x_1$ | $e_0$ (cm) | $I_t$ ($cm^4$) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | $I_{x_0}$ ($cm^4$) | $i_{x_0}$ (cm) | $W_{x_0\max}$ ($cm^3$) | $W_{x_0\min}$ ($cm^3$) | $I_x$ ($cm^4$) | $i_x$ (cm) | $I_y$ ($cm^4$) | $i_y$ (cm) | $I_{x1}$ ($cm^4$) | | |
| $b$ | $t$ | | | | | | | | | | | | | | |
| 30 | 1.5 | 0.85 | 0.67 | 0.828 | 0.77 | 0.95 | 0.93 | 0.35 | 1.25 | 1.21 | 0.29 | 0.58 | 1.35 | 1.07 | 0.0064 |
| 30 | 2.0 | 1.12 | 0.88 | 0.855 | 0.99 | 0.94 | 1.16 | 0.46 | 1.63 | 1.21 | 0.36 | 0.57 | 1.81 | 1.07 | 0.0149 |
| 40 | 2.0 | 1.52 | 1.19 | 1.105 | 2.43 | 1.27 | 2.20 | 0.84 | 3.95 | 1.61 | 0.90 | 0.77 | 4.28 | 1.42 | 0.0203 |
| 40 | 2.5 | 1.87 | 1.47 | 1.132 | 2.96 | 1.26 | 2.62 | 1.03 | 4.85 | 1.61 | 1.07 | 0.76 | 5.36 | 1.42 | 0.0390 |
| 50 | 2.5 | 2.37 | 1.86 | 1.381 | 5.93 | 1.58 | 4.29 | 1.64 | 9.65 | 2.02 | 2.20 | 0.96 | 10.44 | 1.78 | 0.0494 |
| 50 | 3.0 | 2.81 | 2.21 | 1.408 | 6.97 | 1.57 | 4.95 | 1.94 | 11.40 | 2.01 | 2.54 | 0.95 | 12.55 | 1.78 | 0.0843 |
| 60 | 2.5 | 2.87 | 2.25 | 1.630 | 10.41 | 1.90 | 6.38 | 2.38 | 16.90 | 2.43 | 3.91 | 1.17 | 18.03 | 2.13 | 0.0598 |
| 60 | 3.0 | 3.41 | 2.68 | 1.657 | 12.29 | 1.90 | 7.42 | 2.83 | 20.02 | 2.42 | 4.56 | 1.16 | 21.66 | 2.13 | 0.1023 |
| 75 | 2.5 | 3.62 | 2.84 | 2.005 | 20.65 | 2.39 | 10.30 | 3.76 | 33.43 | 3.04 | 7.87 | 1.48 | 35.20 | 2.66 | 0.0755 |
| 75 | 3.0 | 4.31 | 3.39 | 2.031 | 24.47 | 2.38 | 12.05 | 4.47 | 39.70 | 3.03 | 9.23 | 1.46 | 42.26 | 2.66 | 0.1293 |

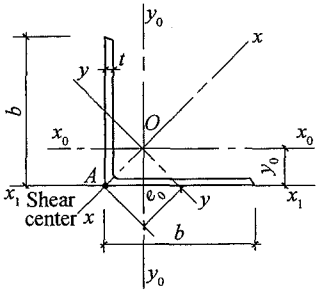


**Table B. 1. 1-3 Channel Steel**

| Dimension (mm) | | | Area of section ($cm^2$) | Mass per meter (kg/m) | $x_0$ (cm) | $x-x$ | | | $y-y$ | | | | $y_1-y_1$ | $e_0$ (cm) | $I_t$ ($cm^4$) | $I_\omega$ ($cm^6$) | $k$ ($cm^{-1}$) | $W_{\omega 1}$ ($cm^4$) | $W_{\omega 2}$ ($cm^4$) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| $h$ | $b$ | $t$ | | | | $I_x$ ($cm^4$) | $i_x$ (cm) | $W_x$ ($cm^3$) | $I_y$ ($cm^4$) | $i_y$ (cm) | $W_{ymax}$ ($cm^3$) | $W_{ymin}$ ($cm^3$) | $I_{y1}$ ($cm^4$) | | | | | | |
| 60 | 30 | 2.5 | 2.74 | 2.15 | 0.883 | 14.38 | 2.31 | 4.89 | 2.40 | 0.94 | 2.71 | 1.13 | 4.53 | 1.88 | 0.0571 | 12.21 | 0.0425 | 4.72 | 2.51 |
| 80 | 40 | 2.5 | 3.74 | 2.94 | 1.132 | 36.70 | 3.13 | 9.18 | 5.92 | 1.26 | 5.23 | 2.06 | 10.71 | 2.51 | 0.0779 | 57.36 | 0.0229 | 11.61 | 6.37 |
| 80 | 40 | 3.0 | 4.43 | 3.48 | 1.159 | 42.66 | 3.10 | 10.67 | 6.93 | 1.25 | 5.98 | 2.44 | 12.87 | 2.51 | 0.1328 | 64.58 | 0.0282 | 13.64 | 7.34 |
| 100 | 40 | 2.5 | 4.24 | 3.33 | 1.013 | 62.07 | 3.83 | 12.41 | 6.37 | 1.23 | 6.29 | 2.13 | 10.72 | 2.30 | 0.0884 | 99.70 | 0.0185 | 17.07 | 8.44 |
| 100 | 40 | 3.0 | 5.03 | 3.95 | 1.039 | 72.44 | 3.80 | 14.49 | 7.47 | 1.22 | 7.19 | 2.52 | 12.89 | 2.30 | 0.1508 | 113.23 | 0.0227 | 20.20 | 9.79 |
| 120 | 40 | 2.5 | 4.74 | 3.72 | 0.919 | 95.92 | 4.50 | 15.99 | 6.72 | 1.19 | 7.32 | 2.18 | 10.73 | 2.13 | 0.0988 | 156.19 | 0.0156 | 23.62 | 10.59 |
| 120 | 40 | 3.0 | 5.63 | 4.42 | 0.944 | 112.28 | 4.47 | 18.71 | 7.90 | 1.19 | 8.37 | 2.58 | 12.91 | 2.12 | 0.1688 | 178.49 | 0.0191 | 28.13 | 12.33 |
| 140 | 50 | 3.0 | 6.83 | 5.36 | 1.187 | 191.53 | 5.30 | 27.36 | 15.52 | 1.51 | 13.08 | 4.07 | 25.13 | 2.75 | 0.2048 | 487.60 | 0.0128 | 48.99 | 22.93 |
| 140 | 50 | 3.5 | 7.89 | 6.20 | 1.211 | 218.88 | 5.27 | 31.27 | 17.79 | 1.50 | 14.69 | 4.70 | 29.37 | 2.74 | 0.3223 | 546.44 | 0.0151 | 56.72 | 26.09 |
| 160 | 60 | 3.0 | 8.03 | 6.30 | 1.432 | 300.87 | 6.12 | 37.61 | 26.90 | 1.83 | 18.79 | 5.89 | 43.35 | 3.37 | 0.2408 | 1119.78 | 0.0091 | 78.25 | 38.21 |
| 160 | 60 | 3.5 | 9.29 | 7.20 | 1.456 | 344.94 | 6.09 | 43.12 | 30.92 | 1.82 | 21.23 | 6.81 | 50.63 | 3.37 | 0.3794 | 1264.16 | 0.0108 | 90.71 | 43.68 |

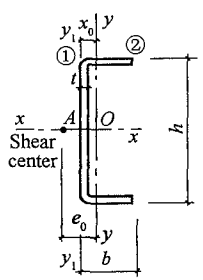


**Table B. 1. 1-4 Channel Steel With Lips**

| Dimension (mm) | | | | Area of section ($cm^2$) | Mass per meter (kg/m) | $x_0$ (cm) | $x-x$ | | | $y-y$ | | | | $y_1-y_1$ | $e_0$ (cm) | $I_t$ ($cm^4$) | $I_\omega$ ($cm^6$) | $k$ ($cm^{-1}$) | $W_{\omega 1}$ ($cm^4$) | $W_{\omega 2}$ ($cm^4$) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| $h$ | $b$ | $a$ | $t$ | | | | $I_x$ ($cm^4$) | $i_x$ (cm) | $W_x$ ($cm^3$) | $I_y$ ($cm^4$) | $i_y$ (cm) | $W_{ymax}$ ($cm^3$) | $W_{ymin}$ ($cm^3$) | $I_{y1}$ ($cm^4$) | | | | | | |
| 80 | 40 | 15 | 2.0 | 3.47 | 2.72 | 1.452 | 34.16 | 3.14 | 8.54 | 7.79 | 1.50 | 5.36 | 3.06 | 15.10 | 3.36 | 0.0462 | 112.9 | 0.0126 | 16.03 | 15.94 |
| 100 | 50 | 15 | 2.5 | 5.23 | 4.11 | 1.706 | 81.34 | 3.94 | 16.27 | 17.19 | 1.81 | 10.08 | 5.22 | 32.41 | 3.94 | 0.1090 | 352.8 | 0.0109 | 34.47 | 29.41 |
| 120 | 50 | 20 | 2.5 | 5.98 | 4.70 | 1.706 | 129.40 | 4.65 | 21.57 | 20.96 | 1.87 | 12.28 | 6.36 | 38.36 | 4.03 | 0.1246 | 660.9 | 0.0085 | 51.04 | 48.36 |
| 120 | 60 | 20 | 3.0 | 7.65 | 6.01 | 2.106 | 170.68 | 4.72 | 28.45 | 37.36 | 2.21 | 17.74 | 9.59 | 71.31 | 4.87 | 0.2296 | 1153.2 | 0.0087 | 75.68 | 68.84 |
| 140 | 50 | 20 | 2.0 | 5.27 | 4.14 | 1.590 | 154.03 | 5.41 | 22.00 | 18.56 | 1.88 | 11.68 | 5.44 | 31.86 | 3.87 | 0.0703 | 794.79 | 0.0058 | 51.44 | 52.22 |
| 140 | 50 | 20 | 2.2 | 5.76 | 4.52 | 1.590 | 167.40 | 5.39 | 23.91 | 20.03 | 1.87 | 12.62 | 5.87 | 34.53 | 3.84 | 0.0929 | 852.46 | 0.0065 | 55.98 | 56.84 |
| 140 | 50 | 20 | 2.5 | 6.48 | 5.09 | 1.580 | 186.78 | 5.39 | 26.68 | 22.11 | 1.85 | 13.96 | 6.47 | 38.38 | 3.80 | 0.1351 | 931.89 | 0.0075 | 62.56 | 63.56 |
| 140 | 60 | 20 | 3.0 | 8.25 | 6.48 | 1.964 | 245.42 | 5.45 | 35.06 | 39.49 | 2.19 | 20.11 | 9.79 | 71.33 | 4.61 | 0.2467 | 1589.8 | 0.0078 | 92.69 | 79.00 |
| 160 | 60 | 20 | 2.0 | 6.07 | 4.76 | 1.850 | 236.59 | 6.24 | 29.57 | 29.99 | 2.22 | 16.19 | 7.23 | 50.83 | 4.52 | 0.0809 | 1596.28 | 0.0044 | 76.92 | 71.30 |
| 160 | 60 | 20 | 2.2 | 6.64 | 5.21 | 1.850 | 257.57 | 6.23 | 32.20 | 32.45 | 2.21 | 17.53 | 7.82 | 55.19 | 4.50 | 0.1071 | 1717.82 | 0.0049 | 83.82 | 77.55 |
| 160 | 60 | 20 | 2.5 | 7.48 | 5.87 | 1.850 | 288.13 | 6.21 | 36.02 | 35.96 | 2.19 | 19.47 | 8.66 | 61.49 | 4.45 | 0.1559 | 1887.71 | 0.0056 | 93.87 | 86.63 |
| 160 | 70 | 20 | 3.0 | 9.45 | 7.42 | 2.224 | 373.64 | 6.29 | 46.71 | 60.42 | 2.53 | 27.17 | 12.65 | 107.20 | 5.25 | 0.2836 | 3070.5 | 0.0060 | 135.49 | 109.92 |
| 180 | 70 | 20 | 2.0 | 6.87 | 5.39 | 2.110 | 343.93 | 7.08 | 38.21 | 45.18 | 2.57 | 21.37 | 9.25 | 75.87 | 5.17 | 0.0916 | 2934.34 | 0.0035 | 109.50 | 95.22 |
| 180 | 70 | 20 | 2.2 | 7.52 | 5.90 | 2.110 | 374.90 | 7.06 | 41.66 | 48.97 | 2.55 | 23.19 | 10.02 | 82.49 | 5.14 | 0.1213 | 3165.62 | 0.0038 | 119.44 | 103.58 |
| 180 | 70 | 20 | 2.5 | 8.48 | 6.66 | 2.110 | 420.20 | 7.04 | 46.69 | 54.42 | 2.53 | 25.82 | 11.12 | 92.08 | 5.10 | 0.1767 | 3492.15 | 0.0044 | 133.99 | 115.73 |
| 200 | 70 | 20 | 2.0 | 7.27 | 5.71 | 2.000 | 440.04 | 7.78 | 44.00 | 46.71 | 2.54 | 23.32 | 9.35 | 75.88 | 4.96 | 0.0969 | 3672.33 | 0.0032 | 126.74 | 106.15 |
| 200 | 70 | 20 | 2.2 | 7.96 | 6.25 | 2.000 | 479.87 | 7.77 | 47.99 | 50.64 | 2.52 | 25.31 | 10.13 | 82.49 | 4.93 | 0.1284 | 3963.82 | 0.0035 | 138.26 | 115.74 |
| 200 | 70 | 20 | 2.5 | 8.98 | 7.05 | 2.000 | 538.21 | 7.74 | 53.82 | 56.27 | 2.50 | 28.18 | 11.25 | 92.09 | 4.89 | 0.1871 | 4376.18 | 0.0041 | 155.14 | 129.75 |
| 220 | 75 | 20 | 2.0 | 7.87 | 6.18 | 2.080 | 574.45 | 8.54 | 52.22 | 56.88 | 2.69 | 27.35 | 10.50 | 90.93 | 5.18 | 0.1049 | 5313.52 | 0.0028 | 158.43 | 127.32 |
| 220 | 75 | 20 | 2.2 | 8.62 | 6.77 | 2.080 | 626.85 | 8.53 | 56.99 | 61.71 | 2.68 | 29.70 | 11.38 | 98.91 | 5.15 | 0.1391 | 5742.07 | 0.0031 | 172.92 | 138.93 |
| 220 | 75 | 20 | 2.5 | 9.73 | 7.64 | 2.070 | 703.76 | 8.50 | 63.98 | 68.66 | 2.66 | 33.11 | 12.65 | 110.51 | 5.11 | 0.2028 | 6351.05 | 0.0035 | 194.18 | 155.94 |

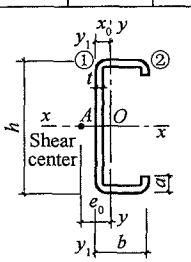
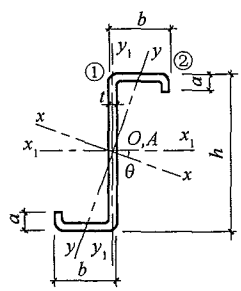


**Table B. 1. 1-5**

| Dimension (mm) | | | | Area of section ($cm^2$) | Mass per meter (kg/m) | $\theta$ (°) | $x_1-x_1$ | | | $y_1-y_1$ | | | $x-x$ | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| $h$ | $b$ | $a$ | $t$ | | | | $I_{x1}$ ($cm^4$) | $i_{x1}$ (cm) | $W_{x1}$ ($cm^3$) | $I_{y1}$ ($cm^4$) | $i_{y1}$ (cm) | $W_{y1}$ ($cm^3$) | $I_x$ ($cm^4$) | $i_x$ (cm) |
| 100 | 40 | 20 | 2.0 | 4.07 | 3.19 | 24.017 | 60.04 | 3.84 | 12.01 | 17.02 | 2.05 | 4.36 | 70.70 | 4.17 |
| 100 | 40 | 20 | 2.5 | 4.98 | 3.91 | 23.767 | 72.10 | 3.80 | 14.42 | 20.02 | 2.00 | 5.17 | 84.63 | 4.12 |
| 120 | 50 | 20 | 2.0 | 4.87 | 3.82 | 24.050 | 106.97 | 4.69 | 17.83 | 30.23 | 2.49 | 6.17 | 126.06 | 5.09 |
| 120 | 50 | 20 | 2.5 | 5.98 | 4.70 | 23.833 | 129.39 | 4.65 | 21.57 | 35.91 | 2.45 | 7.37 | 152.05 | 5.04 |
| 120 | 50 | 20 | 3.0 | 7.05 | 5.54 | 23.600 | 150.14 | 4.61 | 25.02 | 40.88 | 2.41 | 8.43 | 175.92 | 4.99 |
| 140 | 50 | 20 | 2.5 | 6.48 | 5.09 | 19.417 | 186.77 | 5.37 | 26.68 | 35.91 | 2.35 | 7.37 | 209.19 | 5.67 |
| 140 | 50 | 20 | 3.0 | 7.65 | 6.01 | 19.200 | 217.26 | 5.33 | 31.04 | 40.83 | 2.31 | 8.43 | 241.62 | 5.62 |
| 160 | 60 | 20 | 2.5 | 7.48 | 5.87 | 19.983 | 288.12 | 6.21 | 36.01 | 58.15 | 2.79 | 9.90 | 323.13 | 6.57 |
| 160 | 60 | 20 | 3.0 | 8.85 | 6.95 | 19.783 | 336.66 | 6.17 | 42.08 | 66.66 | 2.74 | 11.39 | 376.76 | 6.52 |
| 160 | 70 | 20 | 2.5 | 7.98 | 6.27 | 23.767 | 319.13 | 6.32 | 39.89 | 87.74 | 3.32 | 12.76 | 374.76 | 6.85 |
| 160 | 70 | 20 | 3.0 | 9.45 | 7.42 | 23.567 | 373.64 | 6.29 | 46.71 | 101.10 | 3.27 | 14.76 | 437.72 | 6.80 |
| 180 | 70 | 20 | 2.5 | 8.48 | 6.66 | 20.367 | 420.18 | 7.04 | 46.69 | 87.74 | 3.22 | 12.76 | 473.34 | 7.47 |
| 180 | 70 | 20 | 3.0 | 10.05 | 7.89 | 20.183 | 492.61 | 7.00 | 54.73 | 101.11 | 3.17 | 14.76 | 553.83 | 7.42 |

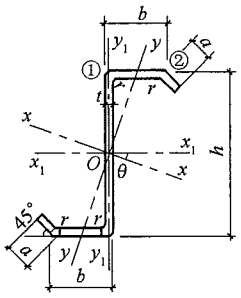


**Z-shape Steel With Lips**

| | | $y-y$ | | | | $I_{x1y1}$ | $I_t$ | $I_\omega$ | $k$ | $W_{\omega1}$ | $W_{\omega2}$ |
|---|---|---|---|---|---|---|---|---|---|---|---|
| $W_{x1}$ ($cm^3$) | $W_{x2}$ ($cm^3$) | $I_y$ ($cm^4$) | $i_y$ (cm) | $W_{y1}$ ($cm^3$) | $W_{y2}$ ($cm^3$) | ($cm^4$) | ($cm^4$) | ($cm^6$) | ($cm^{-1}$) | ($cm^4$) | ($cm^4$) |
| 15.93 | 11.94 | 6.36 | 1.25 | 3.36 | 4.42 | 23.93 | 0.0542 | 325.0 | 0.0081 | 49.97 | 29.16 |
| 19.18 | 14.47 | 7.49 | 1.23 | 4.07 | 5.28 | 28.45 | 0.1038 | 381.9 | 0.0102 | 62.25 | 35.03 |
| 23.55 | 17.40 | 11.14 | 1.51 | 4.83 | 5.74 | 42.77 | 0.0649 | 785.2 | 0.0057 | 84.05 | 43.96 |
| 28.55 | 21.21 | 13.25 | 1.49 | 5.89 | 6.89 | 51.30 | 0.1246 | 930.9 | 0.0072 | 104.68 | 52.94 |
| 33.18 | 24.80 | 15.11 | 1.46 | 6.89 | 7.92 | 58.99 | 0.2116 | 1058.9 | 0.0087 | 125.37 | 61.22 |
| 32.55 | 26.34 | 14.48 | 1.49 | 6.69 | 6.78 | 60.75 | 0.1350 | 1289.0 | 0.0064 | 137.04 | 60.03 |
| 37.76 | 30.70 | 16.52 | 1.47 | 7.84 | 7.81 | 69.93 | 0.2296 | 1468.2 | 0.0077 | 164.94 | 69.51 |
| 44.00 | 34.95 | 23.14 | 1.76 | 9.00 | 8.71 | 96.32 | 0.1559 | 2634.3 | 0.0048 | 205.98 | 86.28 |
| 51.48 | 41.08 | 26.56 | 1.73 | 10.58 | 10.07 | 111.51 | 0.2656 | 3019.4 | 0.0058 | 247.41 | 100.15 |
| 52.35 | 38.23 | 32.11 | 2.01 | 10.53 | 10.86 | 126.37 | 0.1663 | 3793.3 | 0.0041 | 238.87 | 106.91 |
| 61.33 | 45.01 | 37.03 | 1.98 | 12.39 | 12.58 | 146.86 | 0.2836 | 4365.0 | 0.0050 | 285.78 | 124.26 |
| 57.27 | 44.88 | 34.58 | 2.02 | 11.66 | 10.86 | 143.18 | 0.1767 | 4907.9 | 0.0037 | 294.53 | 119.41 |
| 67.22 | 52.89 | 39.89 | 1.99 | 13.72 | 12.59 | 166.47 | 0.3016 | 5652.2 | 0.0045 | 353.32 | 138.92 |

**Table B. 1. 1-6**

| Dimension (mm) | | | | Area of section ($cm^2$) | Mass per meter (kg/m) | $\theta$ (°) | $x_1-x_1$ | | | $y_1-y_1$ | | | $x-x$ | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| $h$ | $b$ | $a$ | $t$ | | | | $I_{x1}$ ($cm^4$) | $i_{x1}$ (cm) | $W_{x1}$ ($cm^3$) | $I_{y1}$ ($cm^4$) | $i_{y1}$ (cm) | $W_{y1}$ ($cm^3$) | $I_x$ ($cm^4$) | $i_x$ (cm) |
| 140 | 50 | 20 | 2.0 | 5.392 | 4.233 | 21.986 | 162.065 | 5.482 | 23.152 | 39.363 | 2.702 | 6.234 | 185.962 | 5.872 |
| 140 | 50 | 20 | 2.2 | 5.909 | 4.638 | 21.998 | 176.813 | 5.470 | 25.259 | 42.928 | 2.695 | 6.809 | 202.926 | 5.860 |
| 140 | 50 | 20 | 2.5 | 6.676 | 5.240 | 22.018 | 198.446 | 5.452 | 28.349 | 48.154 | 2.686 | 7.657 | 227.828 | 5.842 |
| 160 | 60 | 20 | 2.0 | 6.192 | 4.861 | 22.104 | 246.830 | 6.313 | 30.854 | 60.271 | 3.120 | 8.240 | 283.680 | 6.768 |
| 160 | 60 | 20 | 2.2 | 6.789 | 5.329 | 22.113 | 269.592 | 6.302 | 33.699 | 65.802 | 3.113 | 9.009 | 309.891 | 6.756 |
| 160 | 60 | 20 | 2.5 | 7.676 | 6.025 | 22.128 | 303.090 | 6.284 | 37.886 | 73.935 | 3.104 | 10.143 | 348.487 | 6.738 |
| 180 | 70 | 20 | 2.0 | 6.992 | 5.489 | 22.185 | 356.620 | 7.141 | 39.624 | 87.417 | 3.536 | 10.514 | 410.315 | 7.660 |
| 180 | 70 | 20 | 2.2 | 7.669 | 6.020 | 22.193 | 389.835 | 7.130 | 43.315 | 95.518 | 3.529 | 11.502 | 448.592 | 7.648 |
| 180 | 70 | 20 | 2.5 | 8.676 | 6.810 | 22.205 | 438.835 | 7.112 | 48.759 | 107.460 | 3.519 | 12.964 | 505.087 | 7.630 |
| 200 | 70 | 20 | 2.0 | 7.392 | 5.803 | 19.305 | 455.430 | 7.849 | 45.543 | 87.418 | 3.439 | 10.514 | 506.903 | 8.281 |
| 200 | 70 | 20 | 2.2 | 8.109 | 6.365 | 19.309 | 498.023 | 7.837 | 49.802 | 95.520 | 3.432 | 11.503 | 554.346 | 8.268 |
| 200 | 70 | 20 | 2.5 | 9.176 | 7.203 | 19.314 | 560.921 | 7.819 | 56.092 | 107.462 | 3.422 | 12.964 | 624.421 | 8.249 |
| 220 | 75 | 20 | 2.0 | 7.992 | 6.274 | 18.300 | 592.787 | 8.612 | 53.890 | 103.580 | 3.600 | 11.751 | 652.866 | 9.038 |
| 220 | 75 | 20 | 2.2 | 8.769 | 6.884 | 18.302 | 648.520 | 8.600 | 58.956 | 113.220 | 3.593 | 12.860 | 714.276 | 9.025 |
| 220 | 75 | 20 | 2.5 | 9.926 | 7.792 | 18.305 | 730.926 | 8.581 | 66.448 | 127.443 | 3.583 | 14.500 | 805.086 | 9.006 |
| 250 | 75 | 20 | 2.0 | 8.592 | 6.745 | 15.389 | 799.640 | 9.647 | 63.791 | 103.580 | 3.472 | 11.752 | 856.690 | 9.985 |
| 250 | 75 | 20 | 2.2 | 9.429 | 7.402 | 15.387 | 875.145 | 9.634 | 70.012 | 113.223 | 3.465 | 12.860 | 937.579 | 9.972 |
| 250 | 75 | 20 | 2.5 | 10.676 | 8.380 | 15.385 | 986.898 | 9.615 | 78.952 | 127.447 | 3.455 | 14.500 | 1057.30 | 9.952 |



**Z—shape Steel With Inclined Lips**

| | | $y-y$ | | | | $I_{x1y1}$ | $I_t$ | $I_\omega$ | $k$ | $W_{\omega1}$ | $W_{\omega2}$ |
|---|---|---|---|---|---|---|---|---|---|---|---|
| $W_{x1}$ ($cm^3$) | $W_{x2}$ ($cm^3$) | $I_y$ ($cm^4$) | $i_y$ (cm) | $W_{y1}$ ($cm^3$) | $W_{y2}$ ($cm^3$) | ($cm^4$) | ($cm^4$) | ($cm^6$) | ($cm^{-1}$) | ($cm^4$) | ($cm^4$) |
| 30.377 | 22.470 | 15.466 | 1.694 | 6.107 | 8.067 | 59.189 | 0.0719 | 1298.621 | 0.0046 | 118.281 | 59.185 |
| 33.352 | 24.544 | 16.814 | 1.687 | 6.659 | 8.823 | 64.638 | 0.0953 | 1407.575 | 0.0051 | 130.014 | 64.382 |
| 37.792 | 27.598 | 18.771 | 1.667 | 7.468 | 9.941 | 72.659 | 0.1391 | 1563.520 | 0.0058 | 147.558 | 71.926 |
| 40.271 | 29.603 | 23.422 | 1.945 | 8.018 | 9.554 | 90.733 | 0.0826 | 2559.036 | 0.0035 | 175.940 | 82.223 |
| 44.225 | 32.367 | 25.503 | 1.938 | 8.753 | 10.450 | 99.179 | 0.1095 | 2779.796 | 0.0039 | 193.430 | 89.569 |
| 50.132 | 36.445 | 28.537 | 1.928 | 9.834 | 11.775 | 111.642 | 0.1599 | 3098.400 | 0.0044 | 219.605 | 100.26 |
| 51.502 | 37.679 | 33.722 | 2.196 | 10.191 | 11.289 | 131.674 | 0.0932 | 4643.994 | 0.0028 | 249.609 | 111.10 |
| 56.570 | 41.226 | 36.761 | 2.189 | 11.136 | 12.351 | 144.034 | 0.1237 | 5052.769 | 0.0031 | 274.455 | 121.13 |
| 64.143 | 46.471 | 41.208 | 2.179 | 12.528 | 13.923 | 162.307 | 0.1807 | 5654.157 | 0.0035 | 311.661 | 135.81 |
| 56.094 | 43.435 | 35.944 | 2.205 | 11.109 | 11.339 | 146.944 | 0.0986 | 5882.294 | 0.0025 | 302.430 | 123.44 |
| 61.618 | 47.533 | 39.197 | 2.200 | 12.138 | 12.419 | 160.756 | 0.1308 | 6403.010 | 0.0028 | 332.826 | 134.66 |
| 69.876 | 53.596 | 43.962 | 2.189 | 13.654 | 14.021 | 181.182 | 0.1912 | 7160.113 | 0.0032 | 378.452 | 151.08 |
| 65.085 | 51.328 | 43.500 | 2.333 | 12.829 | 12.343 | 181.661 | 0.1066 | 8483.854 | 0.0022 | 383.110 | 148.38 |
| 71.501 | 56.190 | 47.465 | 2.327 | 14.023 | 13.524 | 198.803 | 0.1415 | 9242.136 | 0.0024 | 421.750 | 161.95 |
| 81.096 | 63.392 | 53.283 | 2.317 | 15.783 | 15.278 | 224.175 | 0.2068 | 10347.65 | 0.0028 | 479.804 | 181.87 |
| 71.976 | 61.841 | 46.532 | 2.327 | 14.553 | 12.090 | 207.280 | 0.1146 | 11298.92 | 0.0020 | 485.919 | 169.98 |
| 78.870 | 67.773 | 50.789 | 2.321 | 15.946 | 14.211 | 226.864 | 0.1521 | 12314.34 | 0.0022 | 535.491 | 184.53 |
| 89.108 | 76.584 | 57.044 | 2.312 | 18.014 | 16.169 | 255.870 | 0.2224 | 13797.02 | 0.0025 | 610.188 | 207.38 |

**Table B. 1. 1-7 Equal-leg Angle Steel With Lips**

| Dimension (mm) | | | Area of section ($cm^2$) | Mass per meter (kg/m) | $y_0$ (cm) | $x_0-x_0$ | | | | $x-x$ | | $y-y$ | | $x_1-x_1$ | $e_0$ (cm) | $I_t$ ($cm^4$) | $I_\omega$ ($cm^6$) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | $I_{x_0}$ ($cm^4$) | $i_{x_0}$ (cm) | $W_{x_0\max}$ ($cm^3$) | $W_{x_0\min}$ ($cm^3$) | $I_x$ ($cm^4$) | $i_x$ (cm) | $I_y$ ($cm^4$) | $i_y$ (cm) | $I_{x1}$ ($cm^4$) | | | |
| $b$ | $a$ | $t$ | | | | | | | | | | | | | | | |
| 40 | 15 | 2.0 | 1.95 | 1.53 | 1.404 | 3.93 | 1.42 | 2.80 | 1.51 | 5.74 | 1.72 | 2.12 | 1.04 | 7.78 | 2.37 | 0.0260 | 3.88 |
| 60 | 20 | 2.0 | 2.95 | 2.32 | 2.026 | 13.83 | 2.17 | 6.83 | 3.48 | 20.56 | 2.64 | 7.11 | 1.55 | 25.94 | 3.38 | 0.0394 | 22.64 |
| 75 | 20 | 2.0 | 3.55 | 2.79 | 2.396 | 25.60 | 2.69 | 10.68 | 5.02 | 39.01 | 3.31 | 12.19 | 1.85 | 45.99 | 3.82 | 0.0473 | 36.55 |
| 75 | 20 | 2.5 | 4.36 | 3.42 | 2.401 | 30.76 | 2.66 | 12.81 | 6.03 | 46.91 | 3.28 | 14.60 | 11.83 | 55.90 | 3.80 | 0.0909 | 43.33 |

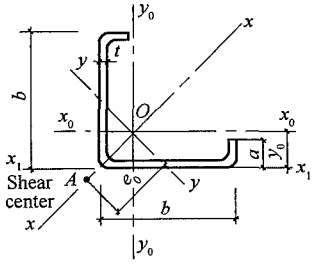
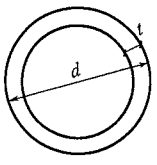


**Table B. 1. 1-8 Welded Thin-wall Circular Steel Tube**

| Dimension (mm) | | Area of section ($cm^2$) | Mass per meter (kg/m) | $I$ ($cm^4$) | $i$ (cm) | $W$ ($cm^3$) |
|---|---|---|---|---|---|---|
| $d$ | $t$ | | | | | |
| 25 | 1.5 | 1.11 | 0.87 | 0.77 | 0.83 | 0.61 |
| 30 | 1.5 | 1.34 | 1.05 | 1.37 | 1.01 | 0.91 |
| 30 | 2.0 | 1.76 | 1.38 | 1.73 | 0.99 | 1.16 |
| 40 | 1.5 | 1.81 | 1.42 | 3.37 | 1.36 | 1.68 |
| 40 | 2.0 | 2.39 | 1.88 | 4.32 | 1.35 | 2.16 |
| 51 | 2.0 | 3.08 | 2.42 | 9.26 | 1.73 | 3.63 |
| 57 | 2.0 | 3.46 | 2.71 | 13.08 | 1.95 | 4.59 |
| 60 | 2.0 | 3.64 | 2.86 | 15.34 | 2.05 | 5.10 |
| 70 | 2.0 | 4.27 | 3.35 | 24.72 | 2.41 | 7.06 |
| 76 | 2.0 | 4.65 | 3.65 | 31.85 | 2.62 | 8.38 |
| 83 | 2.0 | 5.09 | 4.00 | 41.76 | 2.87 | 10.06 |
| 83 | 2.5 | 6.32 | 4.96 | 51.26 | 2.85 | 12.35 |
| 89 | 2.0 | 5.47 | 4.29 | 51.74 | 3.08 | 11.63 |
| 89 | 2.5 | 6.79 | 5.33 | 63.59 | 3.06 | 14.29 |
| 95 | 2.0 | 5.84 | 4.59 | 63.20 | 3.29 | 13.31 |
| 95 | 2.5 | 7.26 | 5.70 | 77.76 | 3.27 | 16.37 |
| 102 | 2.0 | 6.28 | 4.93 | 78.55 | 3.54 | 15.40 |
| 102 | 2.5 | 7.81 | 6.14 | 96.76 | 3.52 | 18.97 |
| 102 | 3.0 | 9.33 | 7.33 | 114.40 | 3.50 | 22.43 |
| 108 | 2.0 | 6.66 | 5.23 | 93.60 | 3.75 | 17.33 |
| 108 | 2.5 | 8.29 | 6.51 | 115.40 | 3.73 | 21.37 |
| 108 | 3.0 | 9.90 | 7.77 | 136.50 | 3.72 | 25.28 |
| 114 | 2.0 | 7.04 | 5.52 | 110.40 | 3.96 | 19.37 |
| 114 | 2.5 | 8.76 | 6.87 | 136.20 | 3.94 | 23.89 |
| 114 | 3.0 | 10.46 | 8.21 | 161.30 | 3.93 | 28.30 |
| 121 | 2.0 | 7.48 | 5.87 | 132.40 | 4.21 | 21.88 |
| 121 | 2.5 | 9.31 | 7.31 | 163.50 | 4.19 | 27.02 |
| 121 | 3.0 | 11.12 | 8.73 | 193.70 | 4.17 | 32.02 |
| 127 | 2.0 | 7.85 | 6.17 | 153.40 | 4.42 | 24.16 |
| 127 | 2.5 | 9.78 | 7.68 | 189.50 | 4.40 | 29.84 |
| 127 | 3.0 | 11.69 | 9.18 | 224.70 | 4.39 | 35.39 |
| 133 | 2.5 | 10.25 | 8.05 | 218.20 | 4.62 | 32.81 |
| 133 | 3.0 | 12.25 | 9.62 | 259.00 | 4.60 | 38.95 |
| 133 | 3.5 | 14.24 | 11.18 | 298.70 | 4.58 | 44.92 |
| 140 | 2.5 | 10.80 | 8.48 | 255.30 | 4.86 | 36.47 |

**Table B. 1. 1-8 (Continued)**

| Dimension (mm) | | Area of section ($cm^2$) | Mass per meter (kg/m) | $I$ ($cm^4$) | $i$ (cm) | $W$ ($cm^3$) |
|---|---|---|---|---|---|---|
| $d$ | $t$ | | | | | |
| 140 | 3.0 | 12.91 | 10.13 | 303.10 | 4.85 | 43.29 |
| 140 | 3.5 | 15.01 | 11.78 | 349.80 | 4.83 | 49.97 |
| 152 | 3.0 | 14.04 | 11.02 | 389.90 | 5.27 | 51.30 |
| 152 | 3.5 | 16.33 | 12.82 | 450.30 | 5.25 | 59.25 |
| 152 | 4.0 | 18.60 | 14.60 | 509.60 | 5.24 | 67.05 |
| 159 | 3.0 | 14.70 | 11.54 | 447.40 | 5.52 | 56.27 |
| 159 | 3.5 | 17.10 | 13.42 | 517.00 | 5.50 | 65.02 |
| 159 | 4.0 | 19.48 | 15.29 | 585.30 | 5.48 | 73.62 |
| 168 | 3.0 | 15.55 | 12.21 | 529.40 | 5.84 | 63.02 |
| 168 | 3.5 | 18.09 | 14.20 | 612.10 | 5.82 | 72.87 |
| 168 | 4.0 | 20.61 | 16.18 | 693.30 | 5.80 | 82.53 |
| 180 | 3.0 | 16.68 | 13.09 | 653.50 | 6.26 | 72.61 |
| 180 | 3.5 | 19.41 | 15.24 | 756.00 | 6.24 | 84.00 |
| 180 | 4.0 | 22.12 | 17.36 | 856.80 | 6.22 | 95.20 |
| 194 | 3.0 | 18.00 | 14.13 | 821.10 | 6.75 | 84.64 |
| 194 | 3.5 | 20.95 | 16.45 | 950.50 | 6.74 | 97.99 |
| 194 | 4.0 | 23.88 | 18.75 | 1078.00 | 6.72 | 111.10 |
| 203 | 3.0 | 18.85 | 15.00 | 943.00 | 7.07 | 92.87 |
| 203 | 3.5 | 21.94 | 17.22 | 1092.00 | 7.06 | 107.55 |
| 203 | 4.0 | 25.01 | 19.63 | 1238.00 | 7.04 | 122.01 |
| 219 | 3.0 | 20.36 | 15.98 | 1187.00 | 7.64 | 108.44 |
| 219 | 3.5 | 23.70 | 18.61 | 1376.00 | 7.62 | 125.65 |
| 219 | 4.0 | 27.02 | 21.81 | 1562.00 | 7.60 | 142.62 |
| 245 | 3.0 | 22.81 | 17.91 | 1670.00 | 8.56 | 136.30 |
| 245 | 3.5 | 26.55 | 20.84 | 1936.00 | 8.54 | 158.10 |
| 245 | 4.0 | 30.28 | 23.77 | 2199.00 | 8.52 | 179.50 |

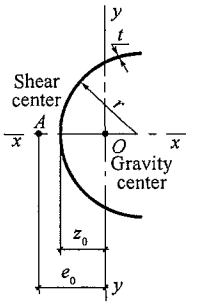


## B. 2 Equivalent Calculation Formulae of Section Characteristics

The following equivalent calculation formulae are all based on the central lines of sections.

Rightward $x$-axis is positive, upward $y$-axis is positive.

**B. 2. 1** The semi-circular steel tube.




$$A=\pi rt$$

$$z_0=0.363r$$

$$I_x=1.571r^3t$$

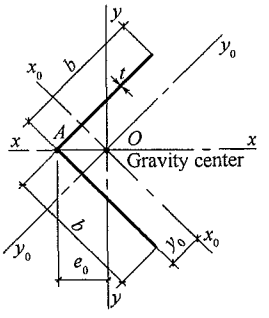
$$I_y=0.298r^3t$$

$$I_t=1.047rt^3$$

$$I_\omega=0.0374r^5t$$

$$e_0=0.636r$$

**B. 2. 2** The equal-leg angle steel.




$$A=2bt$$

$$e_0=\frac{b}{2\sqrt{2}}$$

$$I_x=\frac{1}{3}b^3t$$

$$I_y=\frac{1}{12}b^3t$$

$$I_t=\frac{2}{3}bt^3$$

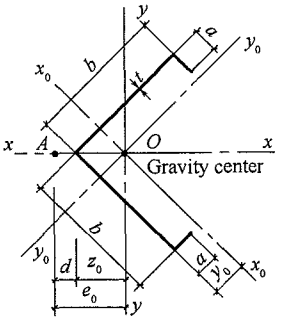
$$I_\omega=0$$

$$I_{x_0}=I_{y_0}=\frac{5}{24}b^3t$$

$$y_0=\frac{b}{4}$$

$$U_y=\frac{b^4t}{12\sqrt{2}}$$

**B. 2. 3** The equal-leg angle steel with lips.




$$A=2(b+a)t$$

$$z_0=\frac{b+a}{2\sqrt{2}}$$

$$I_x=\frac{1}{3}(b^3+a^3)t+ba(b-a)t$$

$$I_y=\frac{1}{12}(b+a)^3t$$

$$I_t=\frac{2}{3}(b+a)t^3$$

$$I_\omega=d^2b^2\left(\frac{b}{3}+\frac{a}{4}\right)t+\frac{2}{3}a\left[\frac{d}{\sqrt{2}}\left(\frac{3}{2}b-a\right)-ba\right]^2t$$

$$d=\frac{ba^2(3b-2a)}{3\sqrt{2}\cdot I_x}\cdot t$$

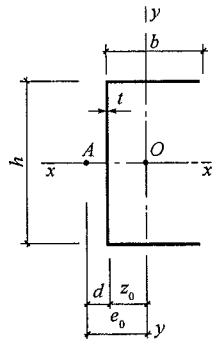
$$e_0=d+z_0$$

$$y_0=\frac{a+b}{4}$$

$$I_{x_0}=I_{y_0}=\frac{5}{24}(a-b)^3t+\frac{a^2bt}{4}+\frac{5}{12}b^3t$$

$$U_y=\frac{t}{12\sqrt{2}}(b^4+4b^3a-6b^2a^2+a^4)$$

**B. 2. 4** The channel steel.




$$A=(2b+h)t$$

$$z_0=\frac{b^2}{2b+h}$$

$$I_x=\frac{1}{12}h^3t+\frac{1}{2}bh^2t$$

$$I_y=hz_0^2t+\frac{1}{6}b^3t+2b\cdot\left(\frac{b}{2}-z_0\right)^2t$$

$$I_t=\frac{1}{3}(2b+h)t^3$$

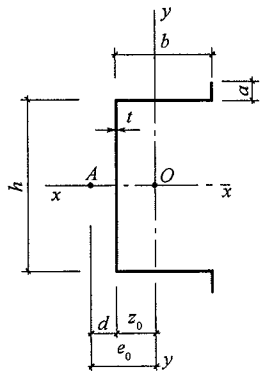
$$I_\omega=\frac{b^3h^2t}{12}\cdot\frac{2h+3b}{6b+h}$$

$$e_0=d+z_0$$

$$d=\frac{3b^2}{6b+h}$$

$$U_y=\frac{1}{2}(b-z_0)^4t-\frac{1}{2}z_0^4t-z_0^3ht+\frac{1}{4}(b-z_0)^2h^2t-\frac{1}{4}z_0^2h^2t-\frac{1}{12}z_0h^3t$$

**B. 2. 5** The channel steel with outward lips.




$$A=(h+2b+2a)t$$

$$z_0=\frac{b(b+2a)}{h+2b+2a}$$

$$I_x=\frac{1}{12}h^3t+\frac{1}{2}bh^2t+\frac{1}{6}a^3t+\frac{1}{2}a(h+a)^2t$$

$$I_y=hz_0^2t+\frac{1}{6}b^3t+2b\cdot\left(\frac{b}{2}-z_0\right)^2t+2a(b-z_0)^2t$$

$$I_t=\frac{1}{3}(h+2b+2a)t^3$$

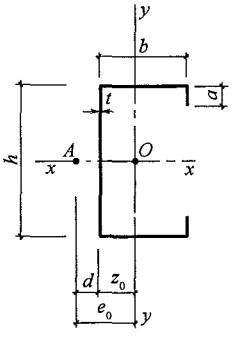
$$I_\omega=\frac{d^2h^3t}{12}+\frac{h^2}{6}[d^3+(b-d)^3]t+\frac{a}{6}[3h^2(d-b)^2+6ha(d^2-b^2)+4a^2(d+b)^2]t$$

$$d=\frac{b}{I_x}\left(\frac{1}{4}bh^2+\frac{1}{2}ah^2-\frac{2}{3}a^3\right)t$$

$$e_0=d+z_0$$

$$U_y=t\left[\frac{(b-z_0)^4}{2}-\frac{z_0^4}{2}-z_0^3h+\frac{(b-z_0)^2h^2}{4}-\frac{z_0^2h^2}{4}-\frac{z_0h^3}{12}+2a(b-z_0)^3+2(b-z_0)\left(\frac{a^3}{3}+\frac{a^2h}{2}+\frac{ah^2}{4}\right)\right]$$

**B. 2. 6** The channel steel with lips.




$$A=(h+2b+2a)t$$

$$z_0=\frac{b(b+2a)}{h+2b+2a}$$

$$I_x=\frac{1}{12}h^3t+\frac{1}{2}bh^2t+\frac{1}{6}a^3t+\frac{1}{2}a(h-a)^2t$$

$$I_y=hz_0^2t+\frac{1}{6}b^3t+2b\cdot\left(\frac{b}{2}-z_0\right)^2t+2a(b-z_0)^2t$$

$$I_t=\frac{1}{3}(h+2b+2a)t^3$$

$$I_\omega=\frac{d^2h^3t}{12}+\frac{h^2}{6}[d^3+(b-d)^3]t+\frac{a}{6}[3h^2(d-b)^2-6ha(d^2-b^2)+4a^2(d+b)^2]t$$

$$d=\frac{b}{I_x}\left(\frac{1}{4}bh^2+\frac{1}{2}ah^2-\frac{2}{3}a^3\right)t$$

$$e_0=d+z_0$$

$$U_y=t\left[\frac{(b-z_0)^4}{2}-\frac{z_0^4}{2}-z_0^3h+\frac{(b-z_0)^2h^2}{4}-\frac{z_0^2h^2}{4}-\frac{z_0h^3}{12}+2a(b-z_0)^3+2(b-z_0)\left(\frac{a^3}{3}-\frac{a^2h}{2}+\frac{ah^2}{4}\right)\right]$$

**B. 2. 7** The Z-shape steel.

$$A=(h+2b)t$$

$$I_{x1}=\frac{1}{12}h^3t+\frac{1}{2}bh^2t$$

$$I_{y1}=\frac{2}{3}b^3t$$

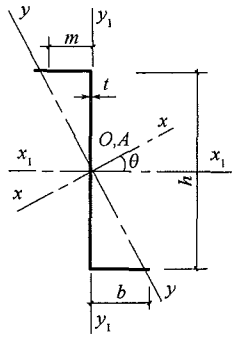
$$I_t=\frac{1}{3}(h+2b)t^3$$

$$I_{x1y1}=-\frac{1}{2}b^2ht$$

$$\mathrm{tg}2\theta=\frac{2I_{x1y1}}{I_{y1}-I_{x1}}$$

$$I_x=I_{x1}\cos^2\theta+I_{y1}\sin^2\theta-2I_{x1y1}\sin\theta\cos\theta$$

$$I_y=I_{x1}\sin^2\theta+I_{y1}\cos^2\theta+2I_{x1y1}\sin\theta\cos\theta$$




$$I_\omega=\frac{b^3h^2t}{12}\cdot\frac{b+2h}{h+2b}$$

$$m=\frac{b^2}{h+2b}$$

**B. 2. 8** The Z-shape steel with lips.

$$A=(h+2b+2a)t$$

$$I_{x1}=\frac{1}{12}h^3t+\frac{1}{2}bh^2t+\frac{1}{6}a^3t+\frac{1}{2}at(h-a)^2$$

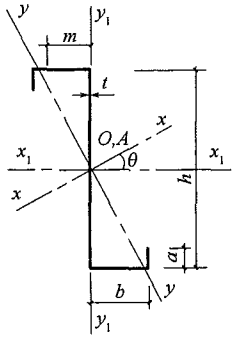
$$I_{y1}=b^2t\left(\frac{2}{3}b+2a\right)$$

$$I_{x1y1}=-\frac{1}{2}bt[bh+2a(h-a)]$$

$$\mathrm{tg}2\theta=\frac{2I_{x1y1}}{I_{y1}-I_{x1}}$$

$$I_x=I_{x1}\cos^2\theta+I_{y1}\sin^2\theta-2I_{x1y1}\sin\theta\cos\theta$$

$$I_y=I_{x1}\sin^2\theta+I_{y1}\cos^2\theta+2I_{x1y1}\sin\theta\cos\theta$$




$$I_t=\frac{1}{3}(h+2b+2a)t^3$$

$$I_\omega=\frac{b^2t}{12(h+2b+2a)}[h^2b(2h+b)+2ah(3h^2+6ah+4a^2)+4abh(h+3a)+4a^3(4b+a)]$$

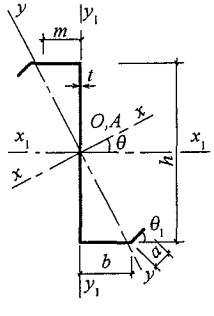
$$m=\frac{2ab(h+a)+b^2h}{(h+2b+2a)h}$$

**B. 2. 9** The Z-shape steel with inclined lips.

$$A=(h+2b+2a)t$$

$$I_{x1}=\frac{1}{12}h^3t+\frac{1}{2}h^2t(a+b)-a^2ht\sin\theta_1+\frac{2}{3}a^3t\sin^2\theta_1$$

$$I_{y1}=\frac{2}{3}b^3t+2ab^2t+2a^2bt\cos\theta_1$$



$$+\frac{2}{3}a^3t\cos^2\theta_1$$

$$I_{x1y1}=-\frac{1}{2}hb^2t-habt+a^2bt\sin\theta_1-\frac{1}{2}ha^2t\cos\theta_1+\frac{2}{3}a^3t\sin\theta_1\cos\theta_1$$

$$\mathrm{tg}2\theta=\frac{2I_{x1y1}}{I_{y1}-I_{x1}}$$

$$I_x=I_{x1}\cos^2\theta+I_{y1}\sin^2\theta-2I_{x1y1}\sin\theta\cos\theta$$

$$I_y=I_{x1}\sin^2\theta+I_{y1}\cos^2\theta+2I_{x1y1}\sin\theta\cos\theta$$

$$I_t=\frac{1}{3}(h+2b+2a)t^3$$

$$I_\omega=\frac{t}{12}[2h^2m^3+3h^3m^2+2h^2(b-m)^3+6ah^2(b-m)^2+$$

$$6a^2h(b-m)n+2a^3n^2]$$

$$m=\frac{bh(b+2a)+a^2n}{(h+2b+2a)h}$$

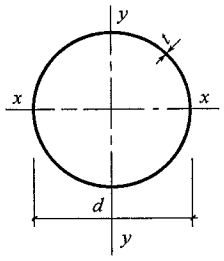
$$n=2b\sin\theta_1+h\cos\theta_1$$

**B. 2. 10** The circular steel tube.

$$A=\pi dt$$

$$I_x=I_y=\frac{1}{8}\pi td^3$$

$$i_x=\frac{d}{2\sqrt{2}}$$



# Appendix C Calculation Method of Design Value of Strength Considering the Effect of Cold Forming

**C. 0. 1** The design value $f'$ of strength considering the effect of cold forming may be calculated according to the following equation:

$$f'=\left[1+\frac{\eta(12\gamma-10)t}{l}\sum_{i=1}^{n}\frac{\theta_i}{2\pi}\right]f \qquad \text{(C. 0. 1-1)}$$

Where $\eta$——Coefficient depending on the type of forming, for cold-formed square or rectangular tubes (squaring from high frequency welded circular tubes), taken as $\eta=1.7$, for circular tubes, square or rectangular tubes formed in other operation and open section members, taken as $\eta=1.0$;

$\gamma$——Ratio of the tensile strength to the yield strength of steel, for steel Q235, taken as $\gamma=1.58$, for steel Q345, taken as $\gamma=1.48$;

$n$——Number of corners for a steel section;

$\theta_i$——Bend angle of rounded corner element $i$ for the steel section (see Fig. C. 0. 1), expressed in radians;

$l$——Midline length of steel section, it may be taken as the ratio of its area to thickness.

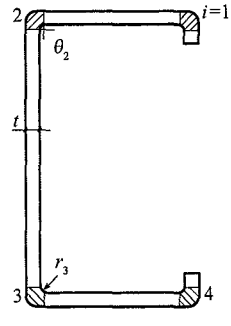



Fig. C. 0. 1 Sketch of cold-formed thin-walled steel section

The midline length of steel section may also be calculated according to the following equation:

$$l=l'+\frac{1}{2}\sum_{i=1}^{n}\theta_i(2r_i+t) \qquad \text{(C. 0. 1-2)}$$

Where $l'$——Sum of flat widths for a steel section;

$r_i$——Inside bend radius of rounded corner element $i$ for the steel section;

$t$——Wall thickness of steel section.

# Appendix D Classification of Corrosive Effect, Primer and Compatible Finish Coating, and Curation Limit of Maintenance

**D. 0. 1** The corrosive effect on cold-formed thin-walled steel structure can be classified according to Table D. 0. 1.

**Table D. 0. 1 Classification of ambient corrosive effect for cold-formed thin-walled steel structure**

| Serial number | Region | Relative humidity (%) | Corrosive effect classification for structure | | |
|---|---|---|---|---|---|
| | | | Indoor (with heating) | Indoor ( without heating) | Open air |
| 1 | Shopping center and residence in rural and in city | Dry, <60 | Non-corrosive | Non-corrosive | Weakly corrosive |
| 2 | | Generally, 60~70 | Non-corrosive | Weak corrosive | Moderately corrosive |
| 3 | | Moist, >75 | Weakly corrosive | Weak corrosive | Moderately corrosive |
| 4 | Industrial area and coastal area | Dry, <60 | Weakly corrosive | Moderately corrosive | Moderately corrosive |
| 5 | | Generally, 60~70 | Weakly corrosive | Moderately corrosive | Moderately corrosive |
| 6 | | Moist, >75 | Moderately corrosive | Moderately corrosive | Moderately corrosive |

Note: 1 Relative humidity in the table refers to local annual average relative humidity, For buildings with assigned relative humidity or constant temperature and humidity taken as indoor relative humidity;

2 Shopping center and residence in ordinary city refers to area where without corrosive medium, Industrial area refers to area where affected by corrosive medium and area emitting slightly corrosive medium.

**D. 0. 2** Frequently used primer and compatible finish coats and corresponding duration limit of maintenance can be taken as shown in Table D. 0. 2.

**Table D. 0. 2 Frequently Used Primer and Compatible Finish Coat and Corresponding Curability for Maintenance**

| Category of corrosive effect | | Surface treatment | Sort of paint | Primer and compatible finish coat | | | | | | Durability for maintenance (years) |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | Primer | Number of layers | Thickness of film ($\mu$) | Finish coat | Number of layers | Thickness of film ($\mu$) | |
| Indoor | Non-Corrosive | Antirust by blast, pickling, manual or semi-mechanical processing | Sort 1 | Y53-31 oil red lead antirust primer | 2 | 60 | C04-2 alkyd enamel chromates | 2 | 60 | 15～20 |
| | | | | Y53-32 oil iron red antirust primer | 2 | 60 | | | | |
| | Weakly corrosive | | | F53-31 red lead phenolic antirust primer | 2 | 60 | C0-45 grey alyd enamel | 2 | 60 | 10～15 |
| | | | | F53-33 iron red phenolic antirust primer | 2 | 60 | | | | |
| | | | | C53-31 red lead alkyd antirust primer | 2 | 60 | | | | |
| Outdoor | Weakly corrosive | | | C06-1 iron red alkyd primer | 2 | 60 | C04-5 grey micaceous iron alkyd ename | 2 | 60 | 8～10 |
| | | | | F53-40 micaceous iron alkyd antirust primer | 2 | 60 | | | | |
| Indoor | Moderately Corrosive | Antirust by pickling, phosphorating and blast | Sort 2 | H06-2iron red epoxy resin primer | 2 | 60 | Grey alkyd-modified perchlorinated Ethylene Enamel | 2 | 60 | 10～15 |
| Outdoor | | | | iron-red epoxy modified M-resin primer | 2 | 60 | alkyd-modified chlorinated rubber enamel | 2 | 60 | 5～7 |
| | | | | H53-30 micaceous iron epoxy resin primer | 2 | 60 | alkyd-modified chloro-acetous enamel | 2 | 60 | |
| | | | | | | | polyester-Modified chloro-acetous enamel | 2 | 60 | |
| | | | | Chlorosulfonated polyethylene anticorrosive primer | 2 | 60 | Chlorosul-fonated polyethylene Primer anticorrosion finish coat | 2 | 60 | 5～7 |

Note: Any primer (chlorosul-fonated polyethylene anticorrosion primer is excluded) of sort 1or sort 2 can be used with any finish paint of the same sort listed compatibly.

## Explanations of Wording in This Code

**1** Words denoting the different degrees of strictness are explained as follows in order that they can be distinguished from each other during the implementation of this code:

1) Words denoting a very strict or a 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 first choice when conditions allow:

"Should" or "may" are used for affirmation;

"Should not" is used for negation.

**2** "Be in accordance with" or "be in compliance with" are used in the text of this Code for those items that name the related standards or codes as a must requirement to implement this code.