Foreword

According to the MOHURD's notice about drawing up or revising constructional standards in the scope of constructional industry, issued in 2004 (assigned as JIANBIAO [2004] No. 99), this Code is drawing up referring to former standard worked up in 2008 and finalized through extensive investigation of advanced standards as well as user's comments.

This Code comprises 14 chapters, with the main technical contents as follows: general provisions; terms and symbols; basic requirements of structural design; loads and load combinations; types and arrangement of structure; calculation and analysis of structure; design of structural members; design of bracing; design of purlin and girt; design of connection and joint; design of enclosure system; protection of steel structure from corrosion; fabrication.

In this Code, the provision printed in bold type is compulsory and must be enforced strictly.

Ministry of Housing and Urban-Rural Development is in charge of administrating to this Code and explanation of the compulsory provision, and China Institute of Building Standard Design & Research Co., Ltd. is responsible for the explanation of specific technical contents. During the process of implementing this Code, suggestions and comments, if any, are kindly requested, and please post or pass to China Institute of Building Standard Design & Research Co., Ltd. (Address: No. 2 Interwest International Center, 9 Shouti South Road, Haidian District, 100048, Beijing)

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Contents

1 General Provisions

1.0.1 This Code is formulated with a view to normalizing the design, fabrication, installation and acceptance of steel structure of light-weight buildings with gabled frames, achieving safety and usability, advanced technology, economy and rationality and ensuring quality.

1.0.2 This Code is applicable to single-storey steel structure buildings with the height of building no greater than 18m, the height-width ratio less than 1, single-span or multi-span solid-web gabled frame as the load-bearing structure, light-weight roof, without bridge crane or with Class A1-A5 bridge crane at a lifting capacity of no greater than 20t or 3t suspension crane.

This Code is not applicable to buildings with high corrosive medium effect on the steel structure as specified in GB 50046 *Code for Anti-corrosion Design of Industrial Constructions*.

1.0.3 Design, fabrication, installation and acceptance of the structural steel light-weight buildings with gabled frames shall meet the provisions of both this Code and relevant national standards.

2 Terms and Symbols

2.1 Terms

2.1.1 Light-weight buildings with gabled frames

The single-storey building structure with tapered or constant section solid-web frame subject to loading with light-weight steel roof and light-weight exterior wall as enclosure system.

2.1.2 Height of building

The average height is taken from the outdoor ground to the roof. Where the roof slope angle is not greater than 10°, cornice height may be taken. Where the roof slope angle is greater than 10°, the average of the cornice height and the roof ridge height shall be taken. For single-slope building, where the roof slope angle is not greater than 10°, cornic height may be taken.

2.1.3 Mezzanine

An indoor platform connected at one side of the frame columns, which is generally arranged along the longitudinal direction of the building and a few is arranged along the gable.

2.1.4 Leaning stanchion

An axial compression member with the upper and lower ends hinged.

2.1.5 Diagonal brace

Bracing member for supporting the rafter and the compression flange of the column.

2.1.6 End wall column

The column arranged at the gable and for transferring the wind load on the gable to the horizontal brace of the roof.

2.1.7 Opening

Those areas in the building envelop (wall, roof surface) which do not have a permanently attached means for effective closure.

2.1.8 Open building

The buildings having all walls at least 80% openings.

2.1.9 Partially enclosed building

A building in which the total area of the openings in a wall that receives positive external pressure exceeds the sum of the areas of the openings of the building envelop (wall and roof) and 10% of the area of that wall, and the density of the opening does not exceed 20% of the remaining exterior surfaces.

2.1.10 Enclosed building

A building that encloses a space and does not have openings that qualify under the definitions of a partially enclosed or open building.

2.1.11 Edge strip

For the purpose of assigning coefficients for enclosure and cladding, the zones divided on the exterior wall and the roof surface at the building end and edge.

2.1.12 End zone

For the purpose of assigning coefficients for main framing, the zones divided on the exterior wall and the roof surface at the building end and edge.

2.1.13 Interior zones

All areas not within the end zone are considered interior zones.

2.1.14 Effective wind load area

The effective load area is considered for determination of the wind load.

2.2 Symbols

2.2.1 Action and effect

F——concentrated load borne by the upper flange;

M_{cr}——critical bending moment for elastic buckling of tapered beam;

M_f——bending moment resisted by flanges;

M_e——bending moment resisted by effective section of the member;

M_f^N——bending moment resisted by flanges when bearing the pressure N concurrently;

N——design value of axial tension or axial pressure;

N_{cr}——Euler critical force;

N_s——pressure induced by the tension field;

N_t——design value of the tensile capacity of one high-strength bolt;

N_{t2}——design value of axial tensile force of one bolt in the second row inside the flange;

R_d——design value of bearing capacity of structural member;

S_E——design value of load combined with seismic effect when considering frequent seismic action;

S_{Ehk}——effect of characteristic value of horizontal seismic action;

S_{Evk}——effect of characteristic value of vertical seismic action;

S_k——characteristic value of snow load;

S_0——reference snow pressure;

S_{Gk}——characteristic value of permanent load effect;

S_{Qk}——characteristic value of vertical variable load effect;

S_{wk}——characteristic value of wind load effect;

S_{GE}——effect of representative value of gravity load;

V_d——design value of shear capacity of the web;

V_{max}——maximum shear force of the purlin;

$V_{x',max}$, $V_{y',max}$——shear force induced by vertical and horizontal load respectively;

V_y——reaction of purlin at support;

W——design value of total vertical load on roof in load-carrying area borne by purlin brace between adjacent columns;

w_k——characteristic value of wind load;

w_0——reference wind pressure.

2.2.2 Material properties and resistance

E——modulus of elasticity of steel;

f——design value of strength of steel materials;

f_{v}——design value of shear strength of steel materials;

f_{t}——design value of tensile strength of steel materials for connected plate;

$f_{\mathrm{f}}^{\mathrm{w}}$——design value of strength of fillet weld;

G——modulus of steel on shear;

R_1——rigidity corresponding to shear deformation of panel zone;

R_2——rigidity of connection on bending.

2.2.3 Geometrical parameters

A_0, A_1——gross sectional area of the member's small end and large end respectively;

A_{e}——effective sectional area;

A_{e1}——effective sectional area of large end;

A_{f}——sectional area of member flange;

A_{k}——sectional area of diagonal bracing ;

A_{n1}——net sectional area of single member ;

A_{p}——sectional area of purlin;

A_{st}——total sectional area of two diagonal stiffening ribs;

d_{b}——depth of rafter end section or panel zone at joint;

e_1——distance from the shear center of the beam section to the centroid line of the purlin;

e_{w}, e_{f}——distance from center of bolted connection to web and flange plate surface respectively;

h_1——distance between center of both flanges at beam end;

h_{b}——depth of snow load determined according to the reference snow pressure on the roof;

h_{c}——width of compressional area of the web;

h_{d}——piled height of snow;

h_0——height of straight-part section of purlin web after deduction of cold-formed radius;

h_{r}——difference in height between high and low roofs;

h_{sT0}, h_{sB0}——distance from center of middle-plane of upper and lower flanges of small-end section to the shear center respectively;

h_{w}——height of web;

h_{w1}, h_{w0}——depth of web at the large and small ends of the tapered section respectively;

I_1——moment of inertia about minor axis of the flange supported by the diagonal brace;

I_2——moment of inertia about minor axis of the flange connected with purlin;

I_{p}——moment of inertia about major axis of the purlin section;

I_{w0}——moment of inertia of warping of the section at small end;

$I_{\mathrm{w}\eta}$——moment of inertia about equivalent to warping of the tapered beam;

i_{x1}——radius of curvature about major axis of the section at large end;

I_{y}——moment of inertia about minor axis of the tapered beam;

i_{y1}——radius of curvature about minor axis of the section at large end;

I_{yT}, I_{yB}——moment of inertia about minor axis of compressive and tensile flanges of the section where the bending moment reaching to maximun respectively;

J, I_y, I_w——free torsional constant, moment of inertia about minor axis and moment of inertia of warping of the section at large end respectively;

J_0——free torsional constant of the section on small end;

J_η——equivalent Saint-Venant torsional constant of the tapered beam;

W_e——sectional modulus at maximum compressive fiber in effective section of the member;

W_{el}——sectional modulus at maximum compressive fiber in effective section of the large end;

W_{enx}, W_{eny}——effective net section modulus or net section modulus about principal axes x and y of the section;

W_{n1x}——net area sectional modulus of the member;

W_{x1}——section modulus of the compressive edge of the section with relatively large bending moment;

γ——tapering ratio of the tapered beam;

γ_p——tapering ratio of the web section;

λ_s——normalized web slenderness ratio of web during buckling by shear;

η_i——ratio of moment of inertia.

2.2.4 Calculation coefficient and others

k_τ——buckling coefficient of components on shear;

η_p——number of purlins in the load-carrying area of purlin brace;

β_{mx}, β_{tx}——equivalent bending moment coefficient;

$\beta_{x\eta}$——coefficent on unsymmetry of section;

γ_{Eh}——partial coefficient of horizontal seismic action;

γ_{Ev}——partial coefficient of vertical seismic action;

γ_G——partial coefficient of permanent or gravity load;

γ_0——coefficient on importance of structure;

γ_Q——partial coefficient of vertical variable load;

γ_{RE}——coefficient on adjustment of seismic resistance;

γ_W——partial coefficient of wind load;

γ_X——coefficient related to plastic deformatiom of the section;

λ_1——slenderness ratio based on large-end section when calculating with effective length of the member;

$\bar{\lambda}_1$——normalized slenderness ratio;

λ_p——parameter related to bending and compression of the plate;

λ_s——parameter related to shear of the plate;

λ_{1y}——slenderness ratio about the minor axis;

$\bar{\lambda}_{1y}$——normalized slenderness ratio about the minor axis;

λ_b——normalized slenderness ratio of the beam;

μ_r——distribution coefficient of snow load on roof;

μ_w——wind load coefficient;

μ_z——height variation coefficient of wind pressure;

ρ——effective width coefficient;

φ_{by}——overall stability coefficient of the beam;

φ_{min}——axial compression stability coefficient of the web member;

φ_s——shear and buckling stability coefficient of the web;

φ_x——axial compression stability coefficient of the bar;

χ_{tap}——tapering ratio reduction coefficient of shear strength while web buckled;

Ψ_Q, Ψ_w——combination value coefficients of the variable load and wind load respectively.

3 Basic Requirements of Structural Design

3.1 Design Principles

3.1.1 Probability-based ultimate state method is adopted in design of structural steel light-weight buildings with gabled frames. Reliability of the structural member is measured with reliability indexes. Expressions of partial coefficients are adopted in design.

3.1.2 Load-bearing members of structural steel light-weight buildings with gabled frames shall be designed according to ultimate limit state and conform to serviceability limit state as well.

3.1.3 When structural members are designed according to ultimate limit state method, requirements expressed in Formula(3.1.3)must be followed:

$$\gamma_0 S_d \leqslant R_d \tag{3.1.3}$$

where γ_0——coefficient on importance of structure. For members with safety Grade I, it is not less than 1.1; for members with safety Grade II, it is not less than 1.0; for members of structural steel buildings with gabled frames, safety Grade II is allowed; for members designed with service life of 25 years, γ_0 shall not be less than 0.95;

S_d——design value of load combination while no any seismic effect present, which shall meet the requirements of 4.5.2 in this Code;

R_d——design value of resistance of structural members.

3.1.4 In case of seismic fortification intensity 7 (0.15g) or above, combination of load and seismic effect must be checked with. Seismic design should meet the requirements of following formula:

$$S_E \leqslant R_d / \gamma_{RE} \tag{3.1.4}$$

where S_E——in case of considering frequent seismic action, design value of load and seismic effect combination shall meet the requirements of 4.5.4 in this Code;

γ_{RE}——coefficent on seismic adjustment of resistance.

3.1.5 Coefficient on seismic adjustment of resistance shall be adopted according to those specified in Table 3.1.5.

Table 3.1.5 Coefficient γ_{RE} on Seismic Adjustment of resistance

Member or connection	Stress state	γ_{RE}
Beam, column, brace, bolt, joint and weld	Strength	0.85
Column and brace	Stable	0.90

3.1.6 Where the structural members is designed according to serviceability limit state, deformation should be checked according to normal load combination stipulated in current national standard GB 50009 *Load Code for the Design of Building Structures*, and comforming requirements stated in section 3.3 in this Code.

3.1.7 Tensile strength of structural members shall be calculated with section in net area.

Compressional strength shall be calculated according to effective net area section. Stability of members shall be checked with effective section. Deformation and that with various stability coefficients may be checked with gross area section.

3.2 Choice of Materials

3.2.1 Choice of steels shall meet the following requirements:

1 For load-bearing cold-formed thin-wall steel, hot rolled section steel and steel plate, Q235 steel specified in the current national standard GB/T 700 *Carbon Structural Steels* and Q345 steel specified in GB/T 1591 *High Strength Low Alloy Structural Steels* shall be adopted.

2 For such members as gabled frame, crane beam and welded purlin and girt, Q235B or Q345A steel or above may be adopted. For such members as non-welded purlin and girt, Q235A steel may be adopted. Where there is a basis, other designation of steels may be adopted for the fabrication of the gabled frame, purlin and girt.

3 For the roofing and cladding for the enclosure system, steel plates in accordance with the requirements of the current national standard GB/T 2518 *Continuously Hot-dip Zinc-coated Steel Sheet and Strip*, GB/T 14978 *Continuously Hot-dip Aluminum-zinc Alloy Coated Steel Sheet and Strip*, GB/T 12754 *Prepainted Steel Sheet* and the profiled sheet adopted shall meet the requirements of the current national standard GB/T 12755 *Profiled Steel Sheet for Building*.

3.2.2 Connecting parts shall meet the following requirements:

1 Ordinary bolt shall meet the requirements of the current national standards GB/T 5780 *Hexagon Head Bolts-Product Grade C* and GB/T 5782 *Hexagon Head Bolts*, its mechanical property and dimensional specification shall meet the requirements of the current national standard GB/T 3098.1 *Mechanical Properties of Fasteners—Bolts, Screws and Studs*;

2 High-strength bolt shall meet the requirements of the current national standards GB/T 1228 *High Strength Bolts with Large Hexagon Head for Steel Structures*, GB/T 1229 *High Strength Large Hexagon Nuts for Steel Structures*, GB/T 1230 *High Strength Plain Washers for Steel Structures*, GB/T 1231 *Specifications of High Strength Bolts with Large Hexagon Head, Large Hexagon Nuts, Plain Washers for Steel Structures* or GB/T 3632 *Sets of Torshear Type High Strength Bolt Hexagon Nut and Plain Washer for Steel Structures*.

3 Self-drilling and self-tapping screw adopted for the connection of the roofing and cladding shall meet the requirements of the current national standards GB/T 15856.1 *Cross Recessed Pan Headed Self-drilling and Self-tapping Screw*, GB/T 15856.2 *Cross Recessed Countersunk Head Drilling Screws*, GB/T 15856.3 *Cross Recessed Raised Countersunk Head Drilling Screws with Tapping Screw Thread*, GB/T 15856.4 *Hexagon Flange Head Drilling Screws with Tapping Screw Thread*, GB/T 15856.5 *Hexagon Washer Head Drilling Screws with Tapping Screw Thread* or GB/T 5282 *Slotted Pan Head Tapping Screws* and GB/T 5283 *Slotted Countersunk Head Tapping Screws*, GB/T 5284 *Slotted Raised Countersunk Head Tapping Screws* and GB/T 5285 *Hexagon Head Tapping Screws*.

4 For the blind rivet with expandable shank, Grade BL2 or BL3 steel specified in the current professional standard YB/T 4155 *Hot-rolled Round Carbon Steel Bars and Rods for Standard Parts* shall be adopted for its fabrication and shall meet the requirements of the current national standards GB/T 12615.1～GB/T 12615.4 *Closed End Blind Rivets with Break Pull Mandrel and*

Protruding Head, GB/T 12616.1 *Closed End Blind Rivets with Break Pull Mandrel and Countersunk Head*, GB/T 12617.1～GB/T 12617.5 *Open End Blind Rivets with Break Pull Mandrel and Countersunk Head* and GB/T 12618.1～GB/T 12618.6 *Open End Blind Rivets with Break Pull Mandrel and Protruding Head*.

5 Fastener shall meet the requirements of the current national standard GB/T 18981 *Fastener*.

6 For the anchor bolt steel, Q235 steel in accordance with those specified in the current national standard GB/T 700 *Carbon Structural Steels* or Q345 steel in accordance with those specified in current national standard GB/T 1591 *High Strength Low Alloy Structural Steels* may be adopted.

3.2.3 Welding materials shall meet the following requirements:

1 Designation and property of manual welding electrode or automatic welding wire shall be appropriate to the property of the member steel. Where two strength grades of steels are welded, welding materials which are compatible with the steel with lower strength should be selected;

2 Material and property of the electrode shall meet the relevant requirements of the current national standards GB/T 5117 *Covered Electrodes for Manual Metal Arc Welding of Non-alloy and Fine Grain Steels* and GB/T 5118 *Covered Electrodes for Manual Metal Arc Welding of Creep-resisting Steels*;

3 Material and property of the welding wire shall meet the relevant requirements of the current national standards GB/T 14957 *Steel Wires for Melt Welding*, GB/T 8110 *Welding Electrodes and Rods for Gas Shielding Arc Welding of Carbon and Low Alloy Steel*, GB/T 10045 *Flux-cored Electrode Used for Non-alloy and Grain Refining Steel* and GB/T 17493 *Low Alloy Steel Flux Cored Electrodes for Arc Welding*;

4 Material and property of the submerged arc welding electrode and flux shall meet the relevant requirements of the current national standards GB/T 5293 *Carbon Steel Electrodes and Fluxes for Submerged Arc Welding* and GB/T 12470 *Low-alloy Steel Electrodes and Fluxes for Submerged Arc Welding*.

3.2.4 Design indexes of steels shall meet the following requirements:

1 Design strength value of various designations of steel shall be adopted according to those specified in Table 3.2.4-1.

Table 3.2.4-1 Design Strength Value of Steel (N/mm^2)

Designation	Steel thickness or diameter (mm)	Design value of tensile, compression and bending strength f	Design value of shear strength f_v	Minimum yield strength f_y	Design value of bearing strength of end surface (planed and topped) f_{ce}
Q235	≤6	215	125	235	320
	>6, ≤16	215	125		
	>16, ≤40	205	120	225	
Q345	≤6	305	175	345	400
	>6, ≤16	305	175		
	>16, ≤40	295	170	335	

Table 3.2.4-1 (continued)

Designation	Steel thickness or diameter (mm)	Design value of tensile, compression and bending strength f	Design value of shear strength f_v	Minimum yield strength f_y	Design value of bearing strength of end surface (planed and topped) f_{ce}
LQ550	≤0.6	455	260	530	—
	>0.6, ≤0.9	430	250	500	
	>0.9, ≤1.2	400	230	460	
	>1.2, ≤1.5	360	210	420	

Note: In this Code, Grade 550 steel is defined as LQ550, which is only used for the roofing and cladding.

2 Design value of the weld strength shall be adopted according to those specified in Table 3.2.4-2.

Table 3.2.4-2 Design Value of the Weld Strength (N/mm²)

Welding method and type of electrode	Designation	Thickness or diameter (mm)	Butt weld				Fillet weld
			Compression f_c^w	Tensile and bending f_t^w		Shear f_v^w	Tensile, compression and shear f_f^w
				Grades Ⅰ and Ⅱ weld	Grade Ⅲ weld		
Automatic welding, semi-automatic welding and manual welding with E43 electrode	Q235	≤5	215	215	185	125	160
		>6, ≤16	215	215	185	125	
		>16, ≤40	205	205	175	120	
Automatic welding, semi-automatic welding and manual welding with E50 electrode	Q345	≤6	305	305	260	175	200
		>6, ≤16	305	305	265	175	
		>16, ≤40	295	295	250	170	

Notes: 1 The weld quality grade shall meet the requirements of the current national standard GB 50205 *Code for Acceptance of Construction Quality of Steel Structures*. For the butt weld whose thickness is less than 8mm, ultrasonic testing should not be adopted for the determination of the weld quality grade.

2 f_c^w is taken as the design value of the strength of the butt weld in the bending and compression area, while f_t^w is taken as that in the bending and tensile areas.

3 The thickness in this table refers to the thickness of the steel at the calculation point and as for the axially loaded member, it refers to the thickness of the thicker plate in the section.

3 Design value of bolt connection strength shall be adopted according to those specified in Table 3.2.4-3.

Table 3.2.4-3 Design Value of Bolt Connection Strength (N/mm²)

Designation/or property grade of steel		Ordinary bolt						Anchor bolt		High-strength bolt in bearing-type connection		
		Grade C bolt			Grades A and B bolt							
		Tensile f_t^b	Shear f_v^b	Bearing f_c^b	Tensile f_t^b	Shear f_v^b	Bearing f_c^b	Tensile f_t^a	Shear f_v^a	Tensile f_t^b	Shear f_v^b	Bearing f_c^b
Ordinary bolt	Grade 4.6 Grade 4.8	170	140	—	—	—	—	—	—	—	—	—
	Grade 5.6	—	—	—	210	190	—	—	—	—	—	—
	Grade 8.8	—	—	—	400	320	—	—	—	—	—	—

Table 3.2.4-3 (continued)

Designation/or property grade of steel		Ordinary bolt						Anchor bolt		High-strength bolt in bearing-type connection		
		Grade C bolt			Grades A and B bolt							
		Tensile f_t^b	Shear f_v^b	Bearing f_c^b	Tensile f_t^b	Shear f_v^b	Bearing f_c^b	Tensile f_t^a	Shear f_v^a	Tensile f_t^b	Shear f_v^b	Bearing f_c^b
Anchor bolt	Q235	—	—	—	—	—	—	140	80	—	—	—
	Q345	—	—	—	—	—	—	180	105	—	—	—
High-strength bolt in bearing-type connection	Grade 8.8	—	—	—	—	—	—	—	—	400	250	—
	Grade 10.9	—	—	—	—	—	—	—	—	500	310	—
Member	Q235	—	—	305	—	—	405	—	—	—	—	470
	Q345	—	—	385	—	—	510	—	—	—	—	590

Notes: 1 Grade A bolt shall be used where d is less than or equal to 24mm and $10d$ or l is less than or equal to 150mm (whichever is smaller); Grade B bolt shall be used where d is greater than 24mm and $10d$ or l is greater than 150mm (whichever is smaller). Where, d is the nominal diameter and l is the nominal length of the screw rod.

2 Both the precision and hole wall surface roughness of Grades A and B bolt holes and the permissible deviation and hole wall surface roughness of Grade C bolt hole shall meet the requirements of the current national standard GB 50205 *Code for Acceptance of Construction Quality of Steel Structures*.

4 Where resistance spot welding is adopted for the cold-formed thin-wall section, the design value of the shear capacity of each welding spot shall meet the requirements of the current national standard GB 50018 *Technical Code of Cold-formed Thin-wall Steel Structures*. Where the whole section of the cold-formed thin-wall steel member is effective, the design value of strength when considering cold-forming effect as specified in the current national standard GB 50018 *Technical Code of Cold-formed Thin-wall Steel Structures* may be adopted for the calculation of the member strength. The members which are subjected to such heat treatment as annealing, welding and hot galvanizing are excluded from the consideration.

5 Physical property index of steel shall be adopted according to those specified in the current national standard GB 50017 *Standard for Design of Steel Structures*.

3.2.5 In calculation of the following structural members or connections, the design value of strength specified in 3.2.4 of this Code shall be multiplied by corresponding reduction coefficient. Where several of the following conditions coexist, corresponding reduction coefficient shall be multiplied continuously.

1 Angle steel in single-side connection:

1) In the calculation of the strength and connection according to axial load, the coefficient 0.85 shall be multiplied by.

2) In the calculation of stability according to axial load:

For equal angle steel with one side connected, the coefficient $0.6+0.0015\lambda$ shall be multiplied by, which shall not be greater than 1.0. For the unequal angle steel with short side connected, the coefficient $0.5+0.0025\lambda$ shall be multiplied by, which shall not be greater than 1.0.

For the unequal angle steel with long side connected, the coefficient 0.70 shall be multiplied by.

Note: λ is slenderness ratio, as for the single-angle steel compressive rod without connection in middle, it shall be calculated according to the minimum radius of curvature. Where λ is less than 20, 20 is taken.

2 For the single-side butt weld without subplate, the coefficient 0.85 shall be multiplied by.

3 For the high altitude installation weld under poor construction conditions, the reduction coefficient 0.90 shall be multiplied by.

4 For two members, where lapping connection, or connection with filler plates between them as well as asymmetrical connection with single cover plate are adopted, the coefficient 0.90 shall be multiplied by.

5 For the main compression web rod at the planar truss purlin end, the coefficient 0.85 shall be multiplied by.

3.2.6 In connection with high-strength bolt, anti-slip coefficient μ among steel friction surface shall be adopted according to those specified in Table 3.2.6-1 and that of coating joint surface shall be adopted according to those specified in Table 3.2.6-2.

Table 3.2.6-1 Anti-slip Coefficient μ of Steel Friction Surface

Treatment method for contact surface of member at the connection		Steel grade of member	
		Q235	Q345
Ordinary steel structure	Shot (sand) blasting	0.35	0.40
	Red rust after shot (sand) blasting	0.45	0.45
	Removal of floating rust with a wire brush or untreated clean rolled surface	0.30	0.35
Cold-formed thin-wall steel structure	Shot (sand) blasting	0.35	0.40
	Removal of floating rust on hot rolled steel rolled surface	0.30	0.35
	Removal of floating rust on cold rolled steel rolled surface	0.25	—

Notes: 1 The direction of rust removal with wire brush shall be perpendicular to the force direction;

2 Where different steel grades are adopted for the connecting members, the μ value shall be taken in accordance with corresponding lower value;

3 Where other methods are adopted for the treatment, both the treatment process and the anti-slip coefficient shall be determined through test.

Table 3.2.6-2 Anti-slip Coefficient μ of Coating Joint Surface

Surface treatment requirement	Coating method and coating thickness	Coating category	Anti-slip coefficient μ
By shot blasting, realizing Grade Sa2$\frac{1}{2}$	Spraying or hand brushing, 50μm~75μm	Alkyd iron red	0.15
		Polyurethane zinc-rich	
		Epoxy zinc-rich	
	Spraying or hand brushing, 50μm~75μm	Inorganic zinc-rich	0.35
		Water-based inorganic zinc-rich	
	Spraying, 30μm~60μm	ZINA	0.45
	Spraying, 80μm~120μm	Anti-slip and anti-rust zinc silicate paint (HES-2)	

Note: Where other coatings (hot-spray aluminum and zinc-coated etc.) are required to be used in the design, their steel surface treatment requirement, coating thickness and anti-slip coefficient shall be determined through test.

3.2.7 Design value of pretension for single high-strength bolt shall be adopted according to those specified in Table 3.2.7.

Table 3.2.7 Design Value of Pretension for Single High-strength Bolt In P(kN)

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

3.3 Deformation Limits

3.3.1 Drift limit for column top of single-storey gabled frame under the action of characteristic value of wind load or frequent earthquakes shall not be greater than the limit specified in Table 3.3.1. Drift limit for the column top at the mezzanine should be $H/250$, where H is the height of column at the mezzanine.

Table 3.3.1 Drift Limit for Rigid Frame Column Top (mm)

Crane condition	Other condition	Displacement limit of column top
Without crane	Where light-weight steel cladding is adopted	$h/60$
	Where masonry wall is adopted	$h/240$
With bridge crane	Where the crane is provided with a cab	$h/400$
	Where the crane is operated on the ground	$h/180$

Note: h in this table is the height of the rigid frame column.

3.3.2 The deflection value of bending member of gabled frame shall not be greater than the limit specified in Table 3.3.2.

Table 3.3.2 Deflection and Span Ratio Limit of Bending Member (mm)

	Type of member		Deflection limit of member
Vertical deflection	Rafter of gabled frame	Only supporting profiled steel roof and cold-formed steel purlin	$L/180$
		with suspended ceiling	$L/240$
		With suspension crane	$L/400$
	Mezzanine	Main beam	$L/400$
		Secondary beam	$L/250$
	Purlin	Only supporting profiled steel roof	$L/150$
		with suspended ceiling too	$L/240$
	Profiled steel roofing		$L/150$
Horizontal deflection	Cladding		$L/100$
	End wall column or wind truss		$L/250$
	Girt	Only supporting profiled steel plate wall	$L/100$
		Supporting masonry wall	$L/180$, $\leqslant$50mm

Notes: 1 L in this table is the span of frame;

2 For rafter of gabled frame, full span is adopted for L;

3 For cantilever beam, the span of bending member shall be calculated according to two times of overhung length.

3.3.3 Roof slope changed value induced by column top displacement and member deflection shall not be greater than 1/3 the design value of the slope.

3.4 Structural Requirements

3.4.1 Wall thickness of the steel structural member and width-to-thickness ratio of the plate shall meet the following requirements:

1 For the cold-formed thin-wall steel used for the purlin and girt, the wall thickness should not be less than 1.5mm. For the steel plate used for web member in welded main rigid frame, the thickness should not be less than 4mm; where otherwise specified, web thickness may be not less than 3mm.

2 Width-to-thickness ratio of compressive plate in the member shall not be greater than the width-to-thickness ratio limit specified in the current national standard GB 50018 *Technical Code of Cold-formed Thin-wall Steel Structures*; in the compression plate in the main rigid frame member, For compressional flange plate of the member with I-shape section, ratio of the free extended length b to its thickness t shall not be greater than $15\sqrt{235/f_y}$; depth-to-thickness ratio of web plate used in welded I-shaped section of beam and column in main frame, h_w to t_w, shall not be greater than 250. Where critical stress of local stability of the compressive plate is less than the yield strength of the steel, stability of the plate shall be checked according to actual stress, or effective width is adopted for calculating the effective section of the member and checking the strength and stability of the member.

3.4.2 Slenderness ratio of the member shall meet the following requirements:

1 Slenderness ratio of the compressive member should not be greater than the limit specified in Table 3.4.2-1.

Table 3.4.2-1 Slenderness Ratio Limit of Compressive Member

Type of member	Slenderness ratio limit
Main member	180
Other member and brace	220

2 Slenderness ratio of tensile member should not be greater than the limit specified in Table 3.4.2-2.

Table 3.4.2-2 Slenderness Ratio Limit of Tensile Member

Type of member	Structure bearing static load or indirectly bearing dynamic load	Structure directly bearing dynamic load
Truss member	350	250
Column brace below crane beam or crane truss	300	—
Other brace except tensioned round steel or steel rope brace	—	—

Notes: 1 For the structure bearing static load, the slenderness ratio of the tensile member in vertical plane may be only calculated;

2 For the structure directly or indirectly subject to the dynamic load, in the calculation of the slenderness ratio of single-angle steel tensile member, minimum radius of curvature of the angle steel shall be adopted; in the calculation of the out-of-plane slenderness ratio of single-angle steel cross tensile member, the radius of curvature about the axis parallel to the angle leg shall be adopted;

3 In case of compression under the action of permanent load and wind load combinations, the slenderness ratio should not be greater than 250.

3.4.3 Where the effect of the seismic action combination is in control of the structural design, seismic resistant measures for the steel structure of light-weight building with gabled frames shall meet the following requirements:

1 For the compressional flange plate of the I-shape section member, the ratio of the free extended width b to its thickness t shall not be greater than $13\sqrt{235/f_y}$; for I-shaped section beam and column member web, the ratio of the calculated height h_w to the thickness t_w shall not be greater than 160;

2 Within the range of three purlin spacings on both sides of the cornice or interior column, the roof beam at each purlin shall be arranged with double-side diagonal brace; cornice wall purlin of side column shall be arranged with diagonal brace on both sides;

3 Where the column base is rigidly connected, the sectional area of anchor bolts shall not be less than 0.15 times the sectional area of the column;

4 Where round steel or steel rope is adopted for the longitudinal brace, relative sliding connection shall not be adopted between the brace and the column web;

5 Slenderness ratio of the column shall not be greater than 150.

4 Loads and Load Combinations

4.1 General Requirements

4.1.1 Design loads adopted for the steel structure of light-weight buildings with gabled frames shall include permanent load, vertical variable load, wind load, thermal action and seismic action.

4.1.2 Suspended load should be considered as a live load. Where the suspended load is unmoved, it may also be considered as a dead load. Load of roof equipments shall be adopted according to actual conditions.

4.1.3 Where light-weight roof with profiled steel sheet is adopted, a normal value of load taken as $0.5\mathrm{kN/m^2}$ applied on the roof acting on horizontal projected area is considered as roof live load. For roof with horizontal projected area of loading greater than $60\mathrm{m^2}$, the value of vertical uniform live load on roof may be taken as not less than $0.3\mathrm{kN/m^2}$.

4.1.4 In the design of roofing and purlin, constructional and repair concentrated load shall also be taken into consideration and its normal value shall be taken as 1.0kN acting at most unfavorable position of the structure; when the constructional load may be exceeded, it shall be adopted according to actual conditions.

4.2 Wind Load

4.2.1 In calculation of the steel structure of light-weight buildings with gabled frames, the maximum projected area perpendicular to the wind direction shall be taken as the action area of wind load, the characteristic value of wind load per unit area perpendicular to building surface shall be calculated according to the following formula:

$$w_k = \beta\mu_w\mu_z w_0 \tag{4.2.1}$$

Where w_k — normal value of wind load ($\mathrm{kN/m^2}$);

w_0 — basic wind pressure ($\mathrm{kN/m^2}$), adopted according to the requirements of the current national standard GB 50009 *Load Code for the Design of building Seructures*;

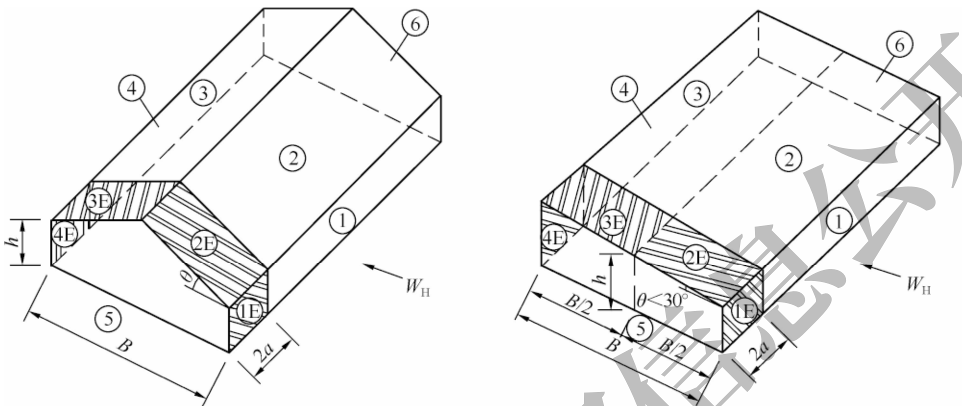
μ_z — height variation coefficient of wind pressure, which is adopted according to the requirements of the current national standard GB 50009 *Load Code for the Design of building Structures*; Where the height is less than 10m, it shall be adopted according to the value at 10m height;

μ_w — wind load coefficient, which is adopted according to the requirements of 4.2.2 in this Code when considering the combination of the maximum internal and external wind pressure;

β — coefficient, which is taken as 1.1 in the calculation of main frame and as 1.5 in the calculation of the purlin, girt, roofing and cladding and their connections.

4.2.2 For the light-weight buildings with gabled frames, where the height of building is not greater than 18m and the height-to-width ratio of the building is less than 1, the wind load coefficient μ_w shall meet the following requirements.

1 Transverse wind load coefficient of main frame shall be adopted according to those specified in Table 4.2.2-1 (Figures 4.2.2-1a and 4.2.2-1b);



(a) Transverse of Gable Roof (b) Transverse of Single-slope Roof

Figure 4.2.2-1 Transverse Wind Load Coefficient Zone of Main Frame

θ—roof slope angle, the included angle between the roof and the horizontal surface; B—width of building; h—average height from the roof to the outdoor ground, for gable roof eave height may be taken approximately, for single-slope roof mid-span height may be taken, a—width of the building edge strip in calculation of the enclosure structure member, for which 10% of the minimum horizontal dimension of the building or $0.4h$, whichever is smaller, but it shall not be less than 4% of the minimum dimension of the building or 1m. 1, 2, 3, 4, 5, 6, 1E, 2E, 3E and 4E are numbers of zones; W_H is the transverse wind.

Table 4.2.2-1 Transverse Wind Load Coefficient of Main Frame

Type of building	Roof slope angle θ	Load condition	Coefficient of end zone				Coefficient of middle zone				Gable
			1E	2E	3E	4E	1	2	3	4	5 and 6
Enclosed type	$0° \leqslant \theta \leqslant 5°$	(+i)	+0.43	−1.25	0.71	−0.60	+0.22	−0.87	−0.55	−0.47	−0.63
		(−i)	+0.79	−0.89	−0.35	−0.25	+0.58	−0.51	−0.19	−0.11	−0.27
	$\theta=10.5°$	(+i)	+0.49	−1.25	−0.76	−0.67	+0.26	−0.87	−0.58	−0.51	−0.63
		(−i)	+0.85	−0.89	−0.40	−0.31	+0.62	−0.51	−0.22	−0.15	−0.27
	$\theta=15.6°$	(+i)	+0.54	−1.25	−0.81	−0.74	+0.30	−0.87	−0.62	−0.55	−0.63
		(−i)	+0.90	−0.89	−0.45	−0.38	+0.66	−0.51	−0.26	−0.19	−0.27
	$\theta=20°$	(+i)	+0.62	−1.25	−0.87	−0.82	+0.35	−0.87	−0.66	−0.61	−0.63
		(−i)	+0.98	−0.89	−0.51	−0.46	+0.71	−0.51	−0.30	−0.25	−0.27
	$30° \leqslant \theta \leqslant 45°$	(+i)	+0.51	+0.09	−0.71	−0.66	+0.38	+0.03	−0.61	−0.55	−0.63
		(−i)	+0.87	+0.45	−0.35	−0.30	+0.74	+0.39	−0.25	−0.19	−0.27
Partially enclosed type	$0° \leqslant \theta \leqslant 5°$	(+i)	+0.06	−1.62	−1.08	−0.98	−0.15	−1.24	−0.92	−0.84	−1.00
		(−i)	+1.16	−0.52	+0.02	+0.12	+0.95	−0.14	+0.18	+0.26	+0.10
	$\theta=10.5°$	(+i)	+0.12	−1.62	−1.13	−1.04	−0.11	−1.24	−0.95	−0.88	−1.00
		(−i)	+1.22	−0.52	−0.03	+0.06	+0.99	−0.14	+0.15	+0.22	+0.10
	$\theta=15.6°$	(+i)	+0.17	−1.62	−1.20	−1.11	+0.07	−1.24	−0.99	−0.92	−1.00
		(−i)	+1.27	−0.52	−0.10	−0.01	+1.03	−0.14	+0.11	+0.18	+0.10
	$\theta=20°$	(+i)	+0.25	−1.62	−1.24	−1.19	−0.02	−0.24	−1.03	−0.98	−1.00
		(−i)	+1.35	−0.52	−0.14	−0.09	+1.08	−0.14	+0.07	+0.12	+0.10
	$30° \leqslant \theta \leqslant 45°$	(+i)	+0.14	−0.28	−1.08	−1.03	+0.01	−0.34	−0.98	−0.92	−1.00
		(−i)	+1.24	+0.82	+0.02	+0.07	+1.11	+0.76	+0.12	+0.18	+0.10

Table 4.2.2-1 (continued)

Type of building	Roof slope angle θ	Load condition	Coefficient of end zone				Coefficient of middle zone				Gable
			1E	2E	3E	4E	1	2	3	4	5 and 6
Open type	$0^\circ \leqslant \theta \leqslant 10^\circ$	Balanced	+0.75	−0.50	−0.50	−0.75	+0.75	−0.50	−0.50	−0.75	−0.75
		Unbalanced	+0.75	−0.20	−0.60	−0.75	+0.75	−0.20	−0.60	−0.75	−0.75
	$10^\circ < \theta \leqslant 25^\circ$	Balanced	+0.75	−0.50	−0.50	−0.75	+0.75	−0.50	−0.50	−0.75	−0.75
		Unbalanced	+0.75	+0.50	−0.50	−0.75	+0.75	+0.50	−0.50	−0.75	−0.75
		Unbalanced	+0.75	+0.15	−0.65	−0.75	+0.75	+0.15	−0.65	−0.75	−0.75
	$25^\circ < \theta \leqslant 45^\circ$	Balanced	+0.75	−0.50	−0.50	−0.75	+0.75	−0.50	−0.50	−0.75	−0.75
		Unbalanced	+0.75	+1.40	+0.20	−0.75	+0.75	+1.40	−0.20	−0.75	−0.75

Notes: 1 Load condition of the enclosed and partially enclosed buildings: (+i) represents that internal pressure being pressed, while (−i) represents that being a suction force. The balanced under the load condition of open building represents that Zones 2, 3, 2E and 3E have the same wind load conditions, while the unbalanced represents that they have different wind load conditions.

2 Plus and minus signs in this table respectively represents that the wind force is oriented toward and away from the plate surface.

3 Ungiven θ value coefficient may be linearly interpolated.

4 Where the roof pressure coefficient of Zone 2 is negative, this value is applicable to the range of Zone 2 that the extended width, calculated from the roof edge and vertical to the cornice direction, is 0.5 times the minimum horizontal dimension of the building or 2.5h, whichever is smaller. For the remaining area of Zone 2 till the ridge line, the coefficient of Zone 3 shall be adopted.

2 Longitudinal wind load coefficient of main frame shall be adopted according to those specified in Table 4.2.2-2 (Figures 4.2.2-2a, 4.2.2-2b and 4.2.2-2c);

3 Wind load coefficient of exterior wall shall be adopted according to those specified in Tables 4.2.2-3a and 4.2.2-3b (Figure 4.2.2-3);

4 Wind load coefficient of double-slope roof and cornice shall be adopted according to those specified in Tables 4.2.2-4a, 4.2.2-4b, 4.2.2-4c, 4.2.2-4d, 4.2.2-4e, 4.2.2-4f, 4.2.2-4g, 4.2.2-4h and 4.2.2-4i (Figures 4.2.2-4a, 4.2.2-4b and Figure 4.2.2-4c);

5 Wind load coefficient of multiple double-slope roofs and cornices shall be adopted according to those specified in Tables 4.2.2-5a, 4.2.2-5b, 4.2.2-5c and 4.2.2-5d (Figure 4.2.2-5);

6 Wind load coefficient of single-slope roof shall be adopted according to those specified in Tables 4.2.2-6a, 4.2.2-6b, 4.2.2-6c and 4.2.2-6d (Figures 4.2.2-6a and 4.2.2-6b);

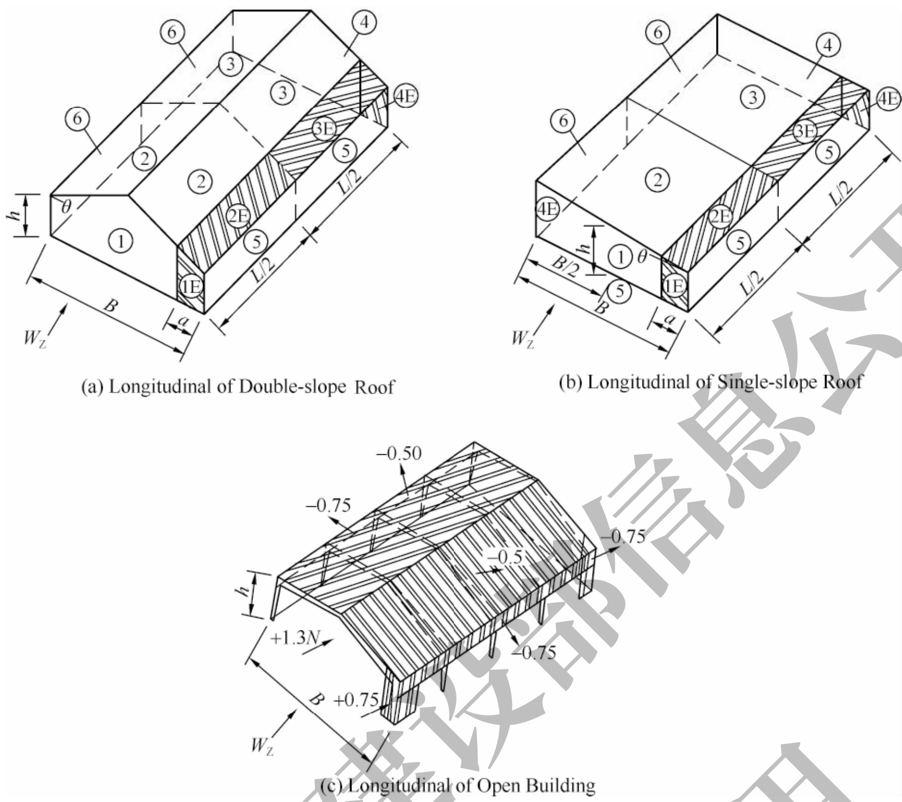
7 Wind load coefficient of saw-shaped roof shall be adopted according to those specified in Tables 4.2.2-7a and 4.2.2-7b (Figure 4.2.2-7).

Table 4.2.2-2 Longitudinal Wind Load Coefficient of Main Rigid Frame (Various Slope Angles θ)

Type of building	Load condition	Coefficient of end zone				Coefficient of middle zone				Sidewall
		1E	2E	3E	4E	1	2	3	4	5 and 6
Enclosed type	(+i)	+0.43	−1.25	−0.71	−0.61	+0.22	−0.87	−0.55	−0.47	−0.63
	(−i)	+0.79	−0.89	−0.35	−0.25	+0.58	−0.51	−0.19	−0.11	−0.27
Partially enclosed type	(+i)	+0.06	−1.62	−1.08	−0.98	−0.15	−1.24	−0.92	−0.84	−1.00
	(−i)	+1.16	−0.52	+0.02	+0.12	+0.95	−0.14	+0.18	+0.26	+0.10
Open type	The value is taken according to Figure4.2.2-2(c)									

Notes: 1 0.75 wind load coefficient in open building is applicable to any covering surface of building surface;

2 Where the open roof is at the plane vertical to the roof ridge, the maximum area of the projected solid web zone of the frame shall be multiplied by 1.3N coefficient; where this coefficient is adopted, it shall meet the following conditions: $0.1 \leqslant \varphi \leqslant 0.3$, $1/6 \leqslant h/B \leqslant 6$, $S/B \leqslant 0.5$. Where, φ is the ratio of the solid web part of the frame to the gross area of the gable and N is the number of the transverse frames.



(a) Longitudinal of Double-slope Roof

(b) Longitudinal of Single-slope Roof

(c) Longitudinal of Open Building

Figure 4. 2. 2-2 Longitudinal Wind Load Coefficient Zone of Main Frame

①、②、③、④、⑤、⑥、①E、②E、③E and ④E—numbers of zones; W_Z— longitudinal wind

Table 4. 2. 2-3a Wind Load Coefficient of Exterior Wall (Wind Suction)

Wind suction coefficient of exterior wall μ_w, for enclosing member and external wall board			
Zone	Effective wind load area A(m²)	Enclosed building	Partially enclosed building
Corner zone(5)	$A \leqslant 1$ $1 < A < 50$ $A \geqslant 50$	−1. 58 +0. 353log$A$-1. 58 −0. 98	−1. 95 +0. 353log$A$-1. 95 −1. 35
Middle zone(4)	$A \leqslant 1$ $1 < A < 50$ $A \geqslant 50$	−1. 28 +0. 176log$A$-1. 28 −0. 98	−1. 65 +0. 176log$A$-1. 65 −1. 35

Table 4. 2. 2-3b Wind Load Coefficient of Exterior Wall (Wind Pressure)

Wind pressure coefficient of exterior wall μ_w, for enclosing member and external wall board			
Zone	Effective wind load area A(m²)	Enclosed building	Partially enclosed building
All the zones	$A \leqslant 1$ $1 < A < 50$ $A \geqslant 50$	+1. 18 −0. 176log$A$+1. 18 +0. 88	+1. 55 −0. 176log$A$+1. 55 +1. 25

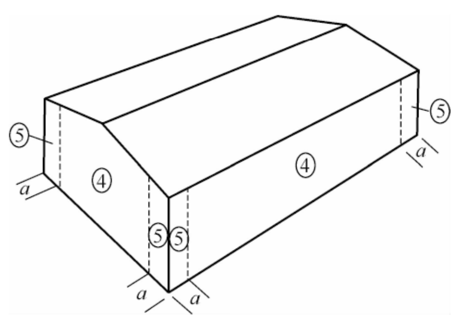


Figure 4. 2. 2-3 Wind Load Coefficient Zone of Exterior Wall

Table 4.2.2-4a Wind Load Coefficient of Gable Roof (Wind Suction)

($0^\circ \leqslant \theta \leqslant 10^\circ$)

Wind suction coefficient of roof μ_w, for enclosing member and roofing			
Zone	Effective wind load area A (m^2)	Enclosed building	Partially enclosed building
Corner zone (3)	$A \leqslant 1$ $1 < A < 10$ $A \geqslant 10$	-2.98 $+1.70\log A - 2.98$ -1.28	-3.35 $+1.70\log A - 3.35$ -1.65
Edge zone (2)	$A \leqslant 1$ $1 < A < 10$ $A \geqslant 10$	-1.98 $+0.70\log A - 1.98$ -1.28	-2.35 $+0.70\log A - 2.35$ -1.65
Middle zone (1)	$A \leqslant 1$ $1 < A < 10$ $A \geqslant 10$	-1.18 $+0.10\log A - 1.18$ -1.08	-1.55 $+0.10\log A - 1.55$ -1.45

Table 4.2.2-4b Wind Load Coefficient of Gable Roof (Wind Pressure)

($0^\circ \leqslant \theta \leqslant 10^\circ$)

Wind pressure coefficient of roof μ_w, for enclosing member and roofing			
Zone	Effective wind load area A (m^2)	Enclosed building	Partially enclosed building
All the zones	$A \leqslant 1$ $1 < A < 10$ $A \geqslant 10$	$+0.48$ $-0.10\log A + 0.48$ $+0.38$	$+0.85$ $-0.10\log A + 0.85$ $+0.75$

Table 4.2.2-4c Wind Load Coefficient of Cornice (Wind Suction)

($0^\circ \leqslant \theta \leqslant 10^\circ$)

Wind suction coefficient of cornice μ_w, for enclosing member and roofing		
Zone	Effective wind load area A (m^2)	Enclosed or partially enclosed building
Corner zone (3)	$A \leqslant 1$ $1 < A < 10$ $A \geqslant 10$	-2.80 $+2.00\log A - 2.80$ -0.80
Edge zone (2) Middle zone (1)	$A \leqslant 1$ $1 < A \leqslant 10$ $10 < A < 50$ $A \geqslant 50$	-1.70 $+0.10\log A - 1.70$ $+0.715\log A - 2.32$ -1.10

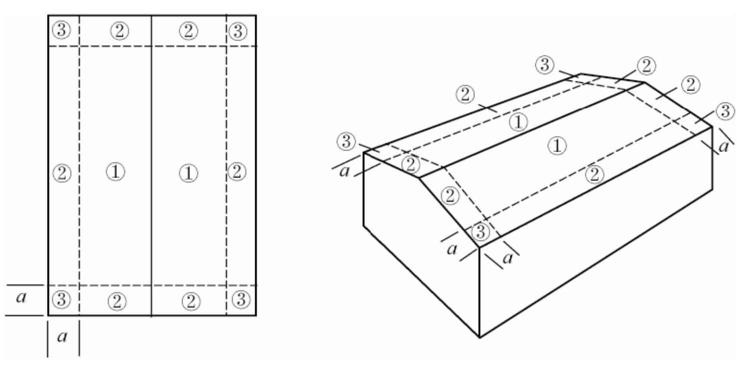


Figure 4.2.2-4a Wind Load Coefficient Zones of Gable Roof and Cornice ($0^\circ \leqslant \theta \leqslant 10^\circ$)

Table 4.2.2-4d　Wind Load Coefficient of Gable Roof (Wind Suction)

($10° \leqslant \theta \leqslant 30°$)

Wind suction coefficient of roof μ_w, for enclosing member and roofing			
Zone	Effective wind load area A (m²)	Enclosed building	Partially enclosed building
Corner zone (3) Edge zone (2)	$A \leqslant 1$ $1 < A < 10$ $A \geqslant 10$	-2.28 $+0.70\log A - 2.28$ -1.58	-2.65 $+0.70\log A - 2.65$ -1.95
Middle zone (1)	$A \leqslant 1$ $1 < A < 10$ $A \geqslant 10$	-1.08 $+0.10\log A - 1.08$ -0.98	-1.45 $+0.10\log A - 1.45$ -1.35

Table 4.2.2-4e　Wind Load Coefficient of Gable Roof (Wind Pressure)

($10° \leqslant \theta \leqslant 30°$)

Wind pressure coefficient of roof μ_w, for enclosing member and roofing			
Zone	Effective wind load area A (m²)	Enclosed building	Partially enclosed building
All the zones	$A \leqslant 1$ $1 < A < 10$ $A \geqslant 10$	$+0.68$ $-0.20\log A + 0.68$ $+0.48$	$+1.05$ $-0.20\log A + 1.05$ $+0.85$

Table 4.2.2-4f　Wind Load Coefficient of cornice (Wind Suction)

($10° \leqslant \theta \leqslant 30°$)

Wind suction coefficient of cornice μ_w, for enclosing member and roofing		
Zone	Effective wind load area A(m²)	Enclosed or partially enclosed building
Corner zone (3)	$A \leqslant 1$ $1 < A < 10$ $A \geqslant 10$	-3.70 $+1.20\log A - 3.70$ -2.50
Edge zone (2)	All the areas	-2.20

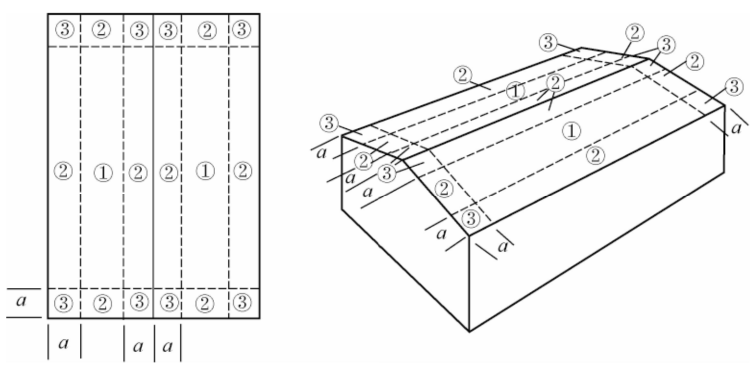


Figure 4.2.2-4b　Wind Load Coefficient Zones of Gable Roof and Eave

($10° \leqslant \theta \leqslant 30°$)

Table 4.2.2-4g Wind Load Coefficient of Gable Roof (Wind Suction)

($30° \leqslant \theta \leqslant 45°$)

Wind suction coefficient of roof μ_w, for enclosing member and roofing			
Zone	Effective wind load area A (m^2)	Enclosed building	Partially enclosed building
Corner zone (3) Edge zone (2)	$A \leqslant 1$ $1 < A < 10$ $A \geqslant 10$	-1.38 $+0.20\log A-1.38$ -1.18	-1.75 $+0.20\log A-1.75$ -1.55
Middle zone(1)	$A \leqslant 1$ $1 < A < 10$ $A \geqslant 10$	-1.18 $+0.20\log A-1.18$ -0.98	-1.55 $+0.20\log A-1.55$ -1.35

Table 4.2.2-4h Wind Load Coefficient of Gable Roof (Wind Pressure)

($30° \leqslant \theta \leqslant 45°$)

Wind pressure coefficient of roof μ_w, for enclosing member and roofing			
Zone	Effective wind load area A (m^2)	Enclosed building	Partially enclosed building
All the zones	$A \leqslant 1$ $1 < A < 10$ $A \geqslant 10$	$+1.08$ $-0.10\log A+1.08$ $+0.98$	$+1.45$ $-0.10\log A+1.45$ $+1.35$

Table 4.2.2-4i Wind Load Coefficient of cornice (Wind Suction)

($30° \leqslant \theta \leqslant 45°$)

Wind suction coefficient of cornice μ_w, for enclosing member and roofing		
Zone	Effective wind load area A (m^2)	Enclosed or partially enclosed building
Corner zone(3) Edge zone (2)	$A \leqslant 1$ $1 < A < 10$ $A \geqslant 10$	-2.00 $+0.20\log A-2.00$ -1.80

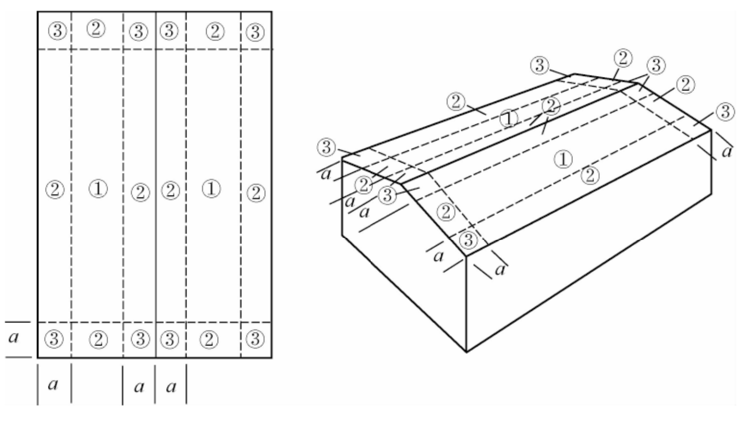


Figure 4.2.2-4c Wind Load Coefficient Zones of Gable Roof and Cornice

($30° \leqslant \theta \leqslant 45°$)

Table 4.2.2-5a Wind Load Coefficient of Multi-span Gable Roof (Wind Suction)

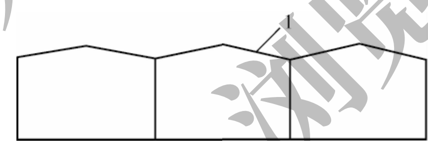
($10° < \theta \leqslant 30°$)

Wind suction coefficient of roof μ_w, for enclosing member and roofing			
Zone	Effective wind load area A (m^2)	Enclosed building	Partially enclosed building
Corner zone(3)	$A \leqslant 1$ $1 < A < 10$ $A \geqslant 10$	-2.88 $+1.00\log A - 2.88$ -1.88	-3.25 $+1.00\log A - 3.25$ -2.25
Edge zone (2)	$A \leqslant 1$ $1 < A < 10$ $A \geqslant 10$	-2.38 $+0.50\log A - 2.38$ -1.88	-2.75 $+0.50\log A - 2.75$ -2.25
Middle zone (1)	$A \leqslant 1$ $1 < A < 10$ $A \geqslant 10$	-1.78 $+0.20\log A - 1.78$ -1.58	-2.15 $+0.20\log A - 2.15$ -1.95

Table 4.2.2-5b Wind Load Coefficient of Multi-span Gable Roof (Wind Pressure)

($10° < \theta \leqslant 30°$)

Wind pressure coefficient of roof μ_w, for enclosing member and roofing			
Zone	Effective wind load area A (m^2)	Enclosed building	Partially enclosed building
All the zones	$A \leqslant 1$ $1 < A < 10$ $A \geqslant 10$	$+0.78$ $-0.20\log A + 0.78$ $+0.58$	$+1.15$ $-0.20\log A + 1.15$ $+0.95$



1—Zone of each double-slope roof is in accordance with Figure4.2.2-4c.

Figure 4.2.2-5 Wind Load Coefficient Zone of Multi-span Gable Roof

Table 4.2.2-5c Wind Load Coefficient of Multi-span Gable Roof (Wind Suction)

($30° < \theta \leqslant 45°$)

Wind suction coefficient of roof μ_w, for enclosing member and roofing			
Zone	Effective wind load area A (m^2)	Enclosed building	Partially enclosed building
Corner zone (3)	$A \leqslant 1$ $1 < A < 10$ $A \geqslant 10$	-2.78 $+0.90\log A - 2.78$ -1.88	-3.15 $+0.90\log A - 3.15$ -2.25
Edge zone (2)	$A \leqslant 1$ $1 < A < 10$ $A \geqslant 10$	-2.68 $+0.80\log A - 2.68$ -1.88	-3.05 $+0.80\log A - 3.05$ -2.25
Middle zone (1)	$A \leqslant 1$ $1 < A < 10$ $A \geqslant 10$	-2.18 $+0.90\log A - 2.18$ -1.28	-2.55 $+0.90\log A - 2.55$ -1.65

Table 4. 2. 2-5d　Wind Load Coefficient of Multi-span Gable Roof (Wind Pressure)

($30^\circ < \theta \leqslant 45^\circ$)

Wind pressure coefficient of roof μ_w, for enclosing member and roofing			
Zone	Effective wind load area A (m^2)	Enclosed building	Partially enclosed building
All the zones	$A \leqslant 1$ $1 < A < 10$ $A \geqslant 10$	$+1.18$ $-0.20\log A + 1.18$ $+0.98$	$+1.55$ $-0.20\log A + 1.55$ $+1.35$

Table 4. 2. 2-6a　Wind Load Coefficient of Single-slope Roof (Wind Suction)

($3^\circ < \theta \leqslant 10^\circ$)

Wind suction coefficient of roof μ_w, for enclosing member and roofing			
Zone	Effective wind load area A(m^2)	Enclosed building	Partially enclosed building
High zone Corner zone (3′)	$A \leqslant 1$ $1 < A < 10$ $A \geqslant 10$	-2.78 $+1.0\log A - 2.78$ -1.78	-3.15 $+1.0\log A - 3.15$ -2.15
Low zone Corner zone (3)	$A \leqslant 1$ $1 < A < 10$ $A \geqslant 10$	-1.98 $+0.60\log A - 1.98$ -1.38	-2.35 $+0.60\log A - 2.35$ -1.75
High zone Edge zone (2′)	$A \leqslant 1$ $1 < A < 10$ $A \geqslant 10$	-1.78 $+0.10\log A - 1.78$ -1.68	-2.15 $+0.10\log A - 2.15$ -2.05
Low zone Edge zone (2)	$A \leqslant 1$ $1 < A < 10$ $A \geqslant 10$	-1.48 $+0.10\log A - 1.48$ -1.38	-1.85 $+0.10\log A - 1.85$ -1.75
Middle zone(1)	All the areas	-1.28	-1.65

Table 4. 2. 2-6b　Wind Load Coefficient of Single-slope Roof (Wind Pressure)

($3^\circ < \theta \leqslant 10^\circ$)

Wind pressure coefficient of roof μ_w, for enclosing member and roofing			
Zone	Effective wind load area A (m^2)	Enclosed building	Partially enclosed building
All the zones	$A \leqslant 1$ $1 < A < 10$ $A \geqslant 10$	$+0.48$ $-0.10\log A + 0.48$ $+0.38$	$+0.85$ $-0.10\log A + 0.85$ $+0.75$

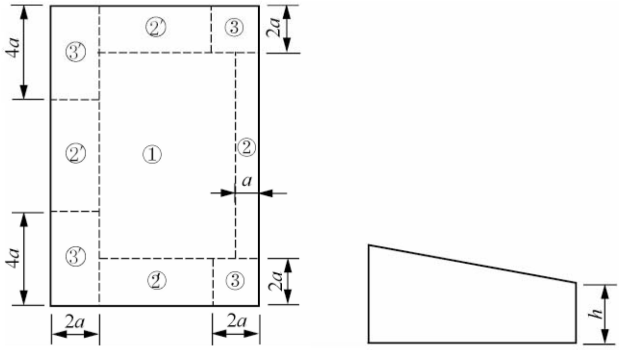


Figure 4. 2. 2-6a Wind Load Coefficient Zones of Single-slope Roof (3°≤θ≤10°)

Table 4. 2. 2-6c Wind Load Coefficient of Single-slope Roof (Wind Suction)

(10°<θ≤30°)

Wind suction coefficient of roof μ_w, for enclosing member and roofing			
Zone	Effective wind load area A (m²)	Enclosed building	Partially enclosed building
High zone Corner zone (3)	$A\leqslant1$ $1<A<10$ $A\geqslant10$	−3. 08 +0. 90log$A$−3. 08 −2. 18	−3. 45 +0. 90log$A$−3. 45 −2. 55
Edge zone (2)	$A\leqslant1$ $1<A<10$ $A\geqslant10$	−1. 78 +0. 40log$A$−1. 78 −1. 38	−2. 15 +0. 40log$A$−2. 15 −1. 75
Middle zone(1)	$A\leqslant1$ $1<A<10$ $A\geqslant10$	−1. 48 +0. 20log$A$−1. 48 −1. 28	−1. 85 +0. 20log$A$−1. 85 −1. 65

Table 4. 2. 2-6d Wind Load Coefficient of Single-slope Roof (Wind Pressure)

(10°<θ≤30°)

Wind pressure coefficient of roof μ_w, for enclosing member and roofing			
Zone	Effective wind load area A (m²)	Enclosed building	Partially enclosed building
All the zones	$A\leqslant1$ $1<A<10$ $A\geqslant10$	+0. 58 −0. 10log$A$+0. 58 +0. 48	+0. 95 −0. 10log$A$+0. 95 +0. 85

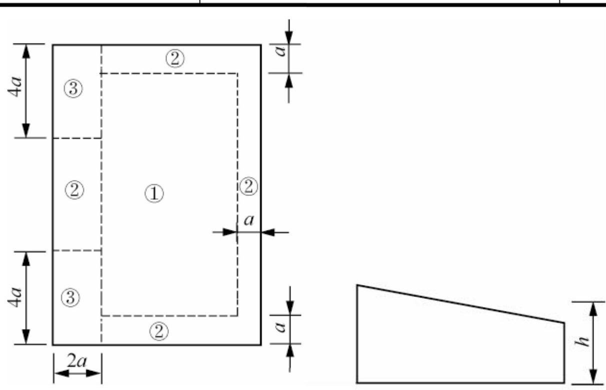


Figure 4. 2. 2-6b Wind Load Coefficient Zones of Single-slope Roof (10°≤θ≤30°)

Table 4.2.2-7a Wind Load Coefficient of Saw-tooth Roof (Wind Suction)

Wind suction coefficient ofsaw-tooth roof μ_w, for enclosing member and roofing			
Zone	Effective wind load area A (m^2)	Enclosed building	Partially enclosed building
Span 1 Corner zone (3)	$A \leqslant 1$ $1 < A \leqslant 10$ $10 < A < 50$ $A \geqslant 50$	-4.28 $+0.40\log A - 4.28$ $+2.289\log A - 6.169$ -2.28	-4.65 $+0.40\log A - 4.65$ $+2.289\log A - 6.539$ -2.65
Spans 2, 3 and 4 Corner zone (3)	$A \leqslant 10$ $10 < A < 50$ $A \geqslant 50$	-2.78 $+1.001\log A - 3.781$ -2.08	-3.15 $+1.001\log A - 4.151$ -2.45
Edge zone (2)	$A \leqslant 1$ $1 < A < 50$ $A \geqslant 50$	-3.38 $+0.942\log A - 3.38$ -1.78	-3.75 $+0.942\log A - 3.75$ -2.15
Middle zone (1)	$A \leqslant 1$ $1 < A < 50$ $A \geqslant 50$	-2.38 $+0.647\log A - 2.38$ -1.28	-2.75 $+0.647\log A - 2.75$ -1.65

Table 4.2.2-7b Wind Load Coefficient of Saw-tooth Roof (Wind Pressure)

Wind pressure coefficient of saw-tooth roof μ_w, for enclosing member and roofing			
Zone	Effective wind load area A (m^2)	Enclosed building	Partially enclosed building
Corner zone (3)	$A \leqslant 1$ $1 < A < 10$ $A \geqslant 10$	$+0.98$ $-0.10\log A + 0.98$ $+0.88$	$+1.35$ $-0.10\log A + 1.35$ $+1.25$
Edge zone (2)	$A \leqslant 1$ $1 < A < 10$ $A \geqslant 10$	$+1.28$ $-0.30\log A + 1.28$ $+0.98$	$+1.65$ $-0.30\log A + 1.65$ $+1.35$
Middle zone(1)	$A \leqslant 1$ $1 < A < 50$ $A \geqslant 50$	$+0.88$ $-0.177\log A + 0.88$ $+0.58$	$+1.25$ $-0.177\log A + 1.25$ $+0.95$

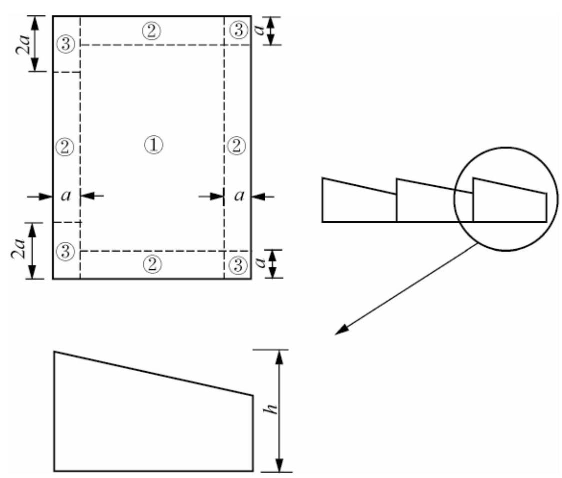


Figure 4.2.2-7 Wind Load Coefficient Zones of Saw-tooth Roof

4.2.3 Effective wind load area (A) of the member of light-weight buildings with gabled frames may be calculated according to following formula:

$$A = lc \tag{4.2.3}$$

Where l——the span of the member considered (m);

c——the wind bearing width of the member considered (m), which shall be greater than $(a+b)/2$ or $l/3$; a and b are the distance from the adjacent members on the left and right sides and the upper and lower sides; $c=l/3$ is adopted for the exterior wall without determined width and other plate-type members.

4.3 Snow Load on Roof

4.3.1 The characteristic value of snow load on the horizontal projection plane of the roof of the steel structure of light-weight buildings with gabled frames shall be calculated according to the following formula:

$$S_k = \mu_r S_0 \tag{4.3.1}$$

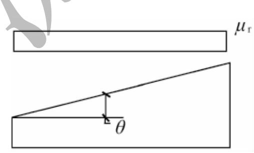
Where S_k——characteristic value of snow load (kN/m²);

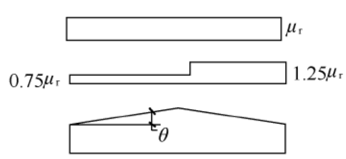
μ_r——distribution coefficient of snow load;

S_0——reference snow pressure (kN/m²), which is adopted according to snow pressure at an recurrence interval of 100 years as specified in the current national standard GB 50009 *Load Code for the Design of building Structures*.

4.3.2 The distribution coefficient of snow load on roof with single-slope, double-stope and multi-slope shall be adopted according to those specified in Table 4.3.2.

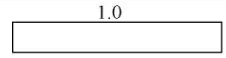
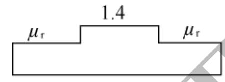
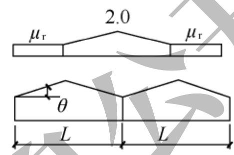
Table 4.3.2 Distribution Coefficient of Snow Load on Roof

<table>
<tr><th>Item</th><th>Type</th><th colspan="2">Roof type and distribution coefficient μ_r of snow load</th></tr>
<tr><td>1</td><td>Single-span and single-slope roof</td><td colspan="2">

<table>
<tr><td>θ</td><td>≤25°</td><td>30°</td><td>35°</td><td>40°</td><td>45°</td><td>50°</td><td>55°</td><td>≥60°</td></tr>
<tr><td>μ_r</td><td>1.00</td><td>0.85</td><td>0.70</td><td>0.55</td><td>0.40</td><td>0.25</td><td>0.10</td><td>0</td></tr>
</table></td></tr>
<tr><td>2</td><td>Single-span and double-slope roof</td><td>Uniform distribution
Non-uniform distribution</td><td>

μ_r is adopted according to those specified in Item 1</td></tr>
</table>

Table 4.3.2 (continued)

Item	Type	Roof type and distribution coefficient μ_r of snow load	
3	Double-span and double-slope roof	Uniform distribution	
		Non-uniform distribution 1	
		Non-uniform distribution 2	
			μ_r is adopted according to those specified in Item 1

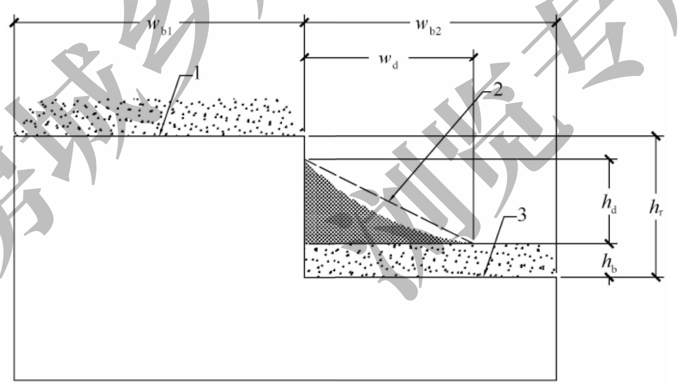
Notes: 1 For double-span and double-slope roof, where the roof slope is not greater than 1/20, non-uniform distribution conditions specified in Item 3 in this table may not be considered for the interior roof, i.e., distribution coefficients 1.4 and 2.0 of snow in this table are considered according to 1.0.

2 Distribution coefficient of snow load on multi-span roof may be adopted according to those specified in Item 3.

4.3.3 Where height of high and low roof and adjacent building roof meets $(h_r - h_b)/h_b > 0.2$, snow accumulation and drift shall be considered according to the following requirements:

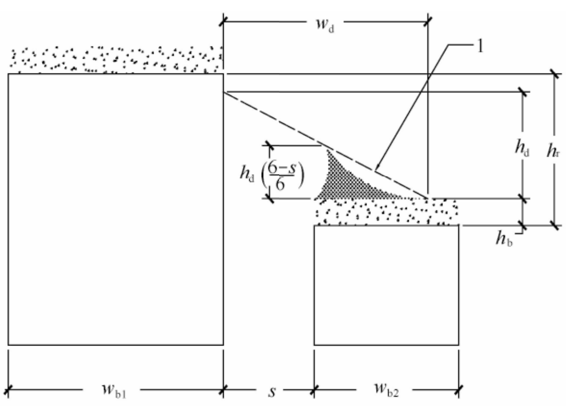
1 Distribution of snow accumulation on low-span roof shall be considered for the high and low roof (Figure 4.3.3-1);

2 Where the spacing s between adjacent buildings is less than 6m, distribution of snow accumulation on low roof shall be considered (Figure 4.3.3-2);



1—high roof; 2—snow-covered area; 3—low roof

Figure 4.3.3-1 Distribution of Snow Accumulation on Low Roof of the High and Low Roof

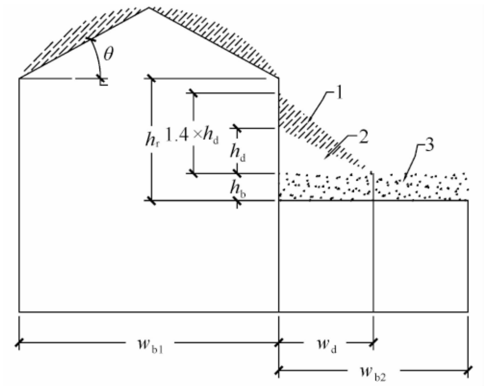


1—snow-covered area

Figure 4.3.3-2 Distribution of Snow Accumulation on Low Roof of the Adjacent Building

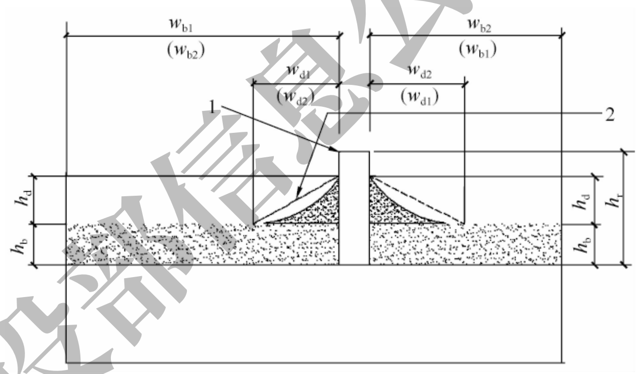
3 Where the slope θ of high roof is greater than 10° and measures to prevent the snow from down-sliding are not taken, snow drift on high roof shall be taken into consideration and the snow height shall be increased by 40%; however, $h_r - h_b$ is taken for the maximum; where the spacing between adjacent buildings is greater than h_r or 6m, snow drift on high roof is not considered (Figure 4.3.3-3);

4 Where the horizontal length of roof protrusion is greater than 4.5m, distribution of snow accumulation on the roof shall be considered (Figure 4.3.3-4);



1—drift snow; 2—snow-covered area;
3—snow load on roof

Figure 4.3.3-3 Distribution of Snow Drift on High Roof and Snow Accumulation on Low Roof



1—roof protrusion; 2—snow-covered area

Figure 4.3.3-4 Distribution of Snow Accumulation with Protrusion on the Roof

5 Piling height h_d of snow shall be calculated according to the following two formulas, whichever is larger:

$$h_d = 0.416\sqrt[3]{w_{b1}}\sqrt[4]{S_0 + 0.479} - 0.457 \leqslant h_r - h_b \tag{4.3.3-1}$$

$$h_d = 0.208\sqrt[3]{w_{b2}}\sqrt[4]{S_0 + 0.479} - 0.457 \leqslant h_r - h_b \tag{4.3.3-2}$$

Where h_d——the piling height of snow (m);

h_r——the height difference of high and low roof (m);

h_b——the height of snow load determined according to the reference snow pressure on the roof (m), $h_b = \dfrac{100S_0}{\rho}$, where ρ is the average density of snow (kg/m^3);

w_{b1}, w_{b2}——the length (width) of the roof, for which 7.5m is taken as the minimum.

6 Piling length w_d of snow shall be determined according to the following requirements:

Where $h_d \leqslant h_r - h_b$,

$$w_d = 4h_d \tag{4.3.3-3}$$

Where $h_d > h_r - h_b$,

$$w_d = 4h_d^2/(h_r - h_b) \leqslant 8(h_r - h_b) \tag{4.3.3-4}$$

7 The load value S_{max} at the highest point of piling snow load shall be calculated according to the following formula:

$$S_{max} = h_d \times \rho \qquad (4.3.3\text{-}5)$$

4.3.4 Average density ρ of snow in various regions shall meet the following requirements:

1 180kg/m^3 is taken for the Northeast China and northern Xinjiang;

2 160kg/m^3 is taken for North China and Northwest China, among which 150kg/m^3 is taken for Qinghai region;

3 180kg/m^3 is generally taken for the southern areas of Huaihe River and Qinling Mountains, among which 230kg/m^3 is taken for Jiangxi and Zhejiang areas.

4.3.5 In the design, distribution of snow shall be adopted according to the following requirements:

1 Roofing and purlin shall be adopted according to the most unfavorable conditions of non-uniform snow distribution;

2 The inclined beam of the frame shall be adopted according to the following most unfavorable conditions: uniform distribution of full-span snow; non-uniform distribution and uniform distribution of semi-span snow;

3 The frame column may be adopted according to the uniform distribution of full-span snow.

4.4 Seismic Action

4.4.1 Seismic fortification category and seismic fortification criterion of steel structure of light-weight buildings with gabled frames shall be adopted according to the requirements of the current national standard GB 50223 *Standard for Classification of Seismic Protection of Building Constructions*.

4.4.2 For steel structure of light-weight buildings with gabled frames, the seismic action shall be considered according to the following principle:

1 Horizontal seismic action is usually calculated according to the direction of two main axes of the building;

2 For the structure with obviously asymmetrical mass and stiffness distribution, two-way horizontal seismic action shall be calculated and the torsion influence shall be counted in;

3 Where the expected seismic intensity is 8 or 9, a vertical seismic action of 10% and 20% of representative value of this structural gravity load should be taken respectively; where the design basic seismic acceleration is 0.30g, 15% of the representative value of this structural gravity load should be taken;

4 Influence of wall on the seismic action should also be considered in the calculation of the seismic action.

4.5 Effect of Load Combinations and Seismic Action

4.5.1 Load combination shall meet the following principles:

1 Where the uniform live load on the roof is not considered simultaneously with the snow load, the larger value shall be taken;

2 The dust load shall be considered simultaneously with the larger one of the snow load and the uniform live load on the roof;

3 Constructional or repair concentrated load is not considered simultaneously with other loads except the deadweight of the roofing material or purlin;

4 Combination of multiple cranes shall meet the requirements of the current national standard GB 50009 *Load Code for the Design of Building Structures*;

5 Wind load is not considered simultaneously with the seismic action.

4.5.2 In a persistent and transient loading case, where the load and load effect are considered according to linear relation, design value of the effect of the basic load combination shall be determined according to the following formula:

$$S_d = \gamma_G S_{Gk} + \Psi_Q \gamma_Q S_{Qk} + \Psi_w \gamma_w S_{wk} \tag{4.5.2}$$

Where S_d——design value of load combination effect;

γ_G——partial coefficient of permanent load;

γ_Q——partial coefficient of vertical variable load;

γ_w——partial coefficient of wind load;

S_{Gk}——characteristic value of permanent load effect;

S_{Qk}——characteristic value of vertical variable load effect;

S_{wk}——characteristic value of wind load effect;

Ψ_Q, Ψ_w——combination value coefficient for variable load and wind load respectively, for which 0.7 and 0 shall be taken respectively where the permanent load effect play a control role. Otherwise, 1.0 and 0.6 or 0.7 and 1.0 shall be taken respectively where the variable load effect play a control role.

4.5.3 Under a persistent or a transient loading case, partial coefficients in basic load combination shall be adopted according to the following requirements:

1 Partial coefficient γ_G for permanent load should take 1.2, when load combination is controlled by the variable load effect, otherewise, 1.35 when combination is controlled by the permanent load, and 1.0 to be taken whenever if load combination will be favorable to the resistance of the structure.

2 1.4 should be taken for the partial coefficient γ_Q of vertical variable load.

3 1.4 should be taken for the partial coefficient γ_w of wind load.

4.5.4 In the seismic design where the effect between different actions are linearly related , design value of the basic combination effect of the load and the seismic action shall be determined according to the following formula:

$$S_E = \gamma_G S_{GE} + \gamma_{Eh} S_{Ehk} + \gamma_{Ev} S_{Evk} \tag{4.5.4}$$

Where S_E——design value of effect by the combination of load and seismic effect;

S_{GE}——effect of representative value of gravity load;

S_{Ehk}——effect of characteristic value of horizontal earthquake action;

S_{Evk}——effect of characteristic value of vertical seismic action;

γ_G——partial coefficient of gravity load;

γ_{Eh}——the partial coefficient of the horizontal seismic action;

γ_{Ev}——partial coefficient of vertical seismic action.

4.5.5 In seismic design, the partial coefficients of load and seismic action in basic combination shall be adopted according to those specified in Table 4.5.5. Where the effect of the gravity load is favorable to the structural bearing capacity, γ_G in Table 4.5.5 should not be greater than 1.0.

Table 4. 5. 5 Partial Coefficients of the Load and the Effect in the Seismic Design Situation

Load and action participated in combination	γ_{G}	γ_{Eh}	γ_{Ev}	Instructions
Gravity load and horizontal seismic action	1. 2	1. 3	—	—
Gravity load and vertical seismic action	1. 2	—	1. 3	Considered in Intensity 8 and Intensity 9 seismic designs
Gravity load and horizontal and vertical seismic action	1. 2	1 3	0. 5	Considered in Intensity 8 and Intensity 9 seismic designs

5 Types and Arrangement of Structure

5.1 Types of Structure

5.1.1 In the steel structure system of light-weight buildings with gabled frames, profiled steel roofing and cold-formed thin-wall steel purlin should be adopted for the roof. Solid-web frame members with tapered section may be adopted for the main rigid frame. Profiled steel sheet cladding and cold-formed thin-wall steel girt should be adopted for the exterior wall. The stability at the lower flange of the rafter of the main rigid frame and the out-of-plane stability at the inner flange of the rigid frame column shall be guaranteed by the diagonal brace. Tensioned rod, steel rope or section steel etc. may be adopted for the cross brace between main rigid frames.

5.1.2 Gabled frames are classified into single-span (Figure 5.1.2a), double-span (Figure 5.1.2b), multi-span (Figure 5.1.2c) frames, frames with Cornice (Figure 5.1.2d) and adjoining building (Figure 5.1.2e) and other types. Hinged connection may be adopted between the middle column and rafter of multi-span rigid frame. Gable roof or single-slope roof (Figure 5.1.2f) should be adopted for the multi-span frame, multi-span frame type composed of multiple gable roof may also be adopted.

The mezzanine, if any, may be arranged along the longitudinal direction (Figure 5.1.2g) or at transverse end span (Figure 5.1.2h). Rigid connection or hinged connection may be adopted between the mezzanine and the column.

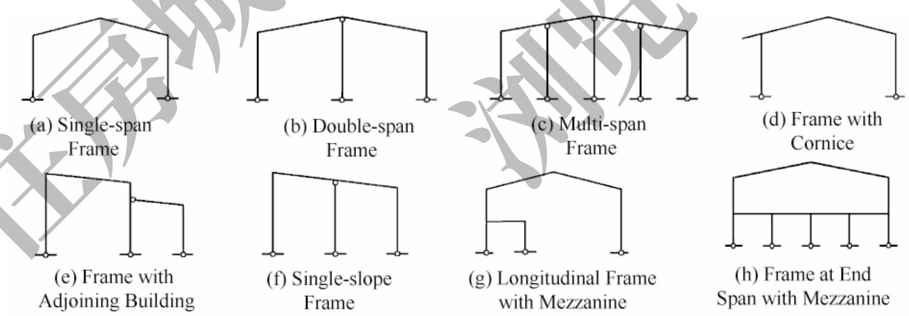


Figure 5.1.2 Examples of Gabled Frame Types

5.1.3 Solid-web welded I-shaped section or rolled H-shaped section with tapered section or constant section may be adopted for the beam and column of the gabled frame according to different spans, heights and loads. Where bridge crane is arranged, constant-section member should be adopted for the column. The variable-section member should be made into the tapered type changing the web height; where necessary, web thickness may also be changed. The structural member should not change the flange section in the manufacturing unit; where necessary, only the flange thickness may be changed; Different flange sections may be adopted for the adjacent manufacturing units. Adjacent sections of two units should have equal height.

5.1.4 Column base of gabled frame should be designed according to the hinged support. Where it is used for industrial factory building and there is above 5t bridge crane, the column may be designed in rigid connection.

5.1.5 Gabled frames may consist of multiple beam and column unit members. The column should be separate unit member; the rafter may be classified into several units according to transportation conditions. Welding shall be adopted for the unit member itself and high-strength bolted connection to be adopted through end plate between the unit members.

5.2 Arrangement of Structure

5.2.1 The dimension of steel structure of light-weight building with gabled frames shall meet the following requirements:

1 The distance between transverse frame column axes shall be taken as the span of the gabled frame.

2 The height from the outdoor ground to the intersection point of the column axis and the rafter axis shall be taken as the height of the gabled frame. Height shall be determined according to the ceiling height required by the service purpose; for the factory building with crane, it shall be determined according to the requirements for the elevation of rail top and the clearance of the crane.

3 Vertical axis through the center of the lower end (smaller end) of the column may be taken as the axis of the column. The axis parallel to the upper surface of the rafter through the center of the smallest end of the rafter section may be taken as the axis of the rafter.

4 Height from the outdoor ground to the upper edge of the purlin on the outer side of the building shall be taken as the eave height of the light-weight building with gabled frames. The height from the outdoor ground to the upper edge of the purlin at the top of the roof shall be taken as the maximum height of the light-weight building with gabled frames. Distance between the external surfaces of girts of the building sidewalls shall be taken as the width of the light-weight building with gabled frames. The distance between the external surfaces of girts of gables on both ends shall be taken as the length of the light-weight building with gabled frames.

5.2.2 The span of the single-span gabled frame should be 12m～48m. Where there is a basis, larger span may be adopted. Where side columns have unequal width, their outer sides shall be aligned. The spacing between the gabled frames, i.e., the longitudinal distance of the column grid axis, should be 6m～9m; cornice length may be determined according to service requirements and should be 0.5m～1.2m; slope of its flange should be the same as that of the rafter.

5.2.3 1/8～1/20 should be taken as the roof slope of the light-weight building with gabled frames; the larger value should be taken in the regions with much rainwater.

5.2.4 Temperature section length of the steel structure of light-weight building with gabled frames shall meet the following requirements:

1 Longitudinal temperature section should not be greater than 300m;

2 Transverse temperature section should not be greater than 150m; where the transverse temperature section is greater than 150m, temperature effect shall be taken into consideration;

3 Where there is a reliable basis, the length of the temperature section may be properly increased.

5.2.5 Arrangement of expansion joint, where necessary, shall meet the following requirements:

1 Slotted hole should be adopted at the bolted connection of lapped purlin, where the roofing shall be allowed for expansion and contraction structurally or to be arranged with double columns;

2 Slotted hole should be adopted at the connection of the crane runway beam and the column.

5.2.6 Joist or bracket should be arranged where the middle column or side column are pulled partially from the multi-span frame.

5.2.7 Influence of such factors as skylight, ventilating roof ridge, lighting belt, roofing material and supply specification of the purlin shall be taken into consideration for the arrangement of roof purlin. Thickness of the profiled steel plate for the roof and the spacing between the purlins shall be determined through calculation.

5.2.8 The gable may be arranged with the frame consisting of the rafter, end wall column, girt and its brace or adopt the gabled frame.

5.2.9 Longitudinal direction of the building shall be provided with definite and reliable force transmission system. Where certain column has relatively weak longitudinal stiffness and strength, the horizontal force shall be transmitted to the adjacent column through the transverse horizontal brace of the building.

5.3 Girt Arrangement

5.3.1 For the arrangement of the side wall girt of the steel structure of light-weight building with gabled frames, the requirements for such members and enclosing materials as doors and windows, cornice, sun shelter and canopy shall be taken into consideration.

5.3.2 For the side wall of the steel structure of light-weight building with gabled frames, where the profiled steel plate is adopted as the enclosing surface, the girt should be arranged on the outer side of the rigid frame column and its spacing shall be determined according to the type and specification of the cladding and shall not be greater than that specified by the calculation.

5.3.3 For the exterior wall of the light-weight buildings with gabled frames, where the seismic fortification intensity is 8 or below, light-weight metal cladding or non-embedded masonry should be adopted; where the seismic fortification intensity is 9, light-weight metal cladding or light-weight cladding in flexible connection with the column shall be adopted.

6 Calculation and Analysis of Structure

6.1 Calculation of Gabled Frame

6.1.1 Gabled frame shall be calculated according to elastic analysis method.

6.1.2 For the gabled frame, stressed skin effect should not be considered and the internal force may be analyzed according to the plane structure.

6.1.3 Where bracing between adjacent columns are not arranged, the column base shall be designed in rigid connection and the column shall be designed to resist two-way lateral force.

6.1.4 When second order elastic analysis is adopted for the frame, notional horizontal load shall be applied. 0.5% of the design value of the vertical load shall be taken as notional horizontal load and applied at the point where the vertical load acts. The notional load shall have the same direction as the wind load or seismic action.

6.2 Analysis of Seismic Action

6.2.1 In calculation of the seismic action of the gabled frame, values of its damping ratio shall meet the following requirements:

1 0.05 may be taken for the enclosed building;

2 0.035 may be taken for the open building;

3 The remaining buildings shall be calculated by interpolation according to the opening ratio of the exterior wall area.

6.2.2 For single-span and multi-span buildings with equal height, horizontal seismic action of transverse rigid frame may be calculated by base shear method; for the buildings with unequal height, it may be calculated according to the mode decomposition response spectrum method.

6.2.3 For the factory building with crane, in calculation of the seismic action, deadweight of the crane shall be taken into consideration by distributing it on average at the two brackets.

6.2.4 Where the masonry wall is adopted as the enclosing wall, the mass of the masonry wall shall be distributed along the height to at least two mass concentration points as the additional mass included to the steel column in calculation of the horizontal seismic action of the rigid frame in transverse direction.

6.2.5 In calculation of seismic action on the longitudinal columns by the base shear method, it shall be guaranteed that all the seismic forces distributed according to the height and mass can be transmitted to the longitudinal brace or frame.

6.2.6 Where the longitudinal length of the building is not greater than 1.5 times of the horizontal width and high and low spans are arranged in both longitudinal and transverse directions, the longitudinal brace system should be calculated according to an integral spatial frame model.

6.2.7 Checking on strong column and weak beam may be omitted for the gabled frame. Where the beam column is connected by an end plate or in a curved transition of the lower flange of the beam and column at the beam-column joint, checking of strong joint and weak member may be omitted. Under other conditions, calculation of strong joint and weak member shall be conducted and the calculation method shall be in accordance with those specified in the current national standard GB 50011 *Code for Seismic Design of Buildings*.

6.2.8 Where the light-weight building with gabled frames has a mezzanine, longitudinal seismic design of the mezzanine may be conducted separately and amplification coefficient 1.2 shall be multiplied for the longitudinal seismic action of inner column.

6.3 Analysis of Thermal Action

6.3.1 Where total width or length of the building exceeds the maximum length of the temperature section specified in 5.2.4 of this Code, measures of releasing temperature stress shall be taken or the thermal action shall be calculated.

6.3.2 In calculation of the thermal action, the basic air temperature shall be adopted according to those specified in the current national standard GB 50009 *Load Code for the Design of Building Structures*. 1.4 should be adopted for the partial coefficient of the thermal action.

6.3.3 When bolted connections are adopted wherever in the longitudinal direction, the thermal action effect may be reduced by a factor of 0.35.

7 Design of Structural Members

7.1 Calculation of Frame Members

7.1.1 Utilization of post-buckling strength of plate shall meet the following requirements:

1 Where the post-buckling strength is utilized for the flexural and compressive web plate of I-shaped section member, the section characteristic shall be calculated in accordance with effective width. The effective width of compressive area shall be calculated according to the following formula:

$$h_e = \rho h_c \tag{7.1.1-1}$$

Where h_e——effective width of compressive area in web (mm);

h_c——width of compressive area in web (mm);

ρ——coefficient of effective width, in case of $\rho > 1.0$, it adopts 1.0.

2 The coefficient of effective width ρ shall be calculated according to the following formulas:

$$\rho = \frac{1}{(0.243 + \lambda_p^{1.25})^{0.9}} \tag{7.1.1-2}$$

$$\lambda_p = \frac{h_w / t_w}{28.1\sqrt{k_\sigma}\ \sqrt{235/f_y}} \tag{7.1.1-3}$$

$$k_\sigma = \frac{16}{\sqrt{(1+\beta)^2 + 0.112(1+\beta)^2} + (1+\beta)} \tag{7.1.1-4}$$

$$\beta = \sigma_2 / \sigma_1 \tag{7.1.1-5}$$

Where λ_p——parameter related to flexual and compressive plate, where $\sigma_1 < f$, f_y in formula (7.1.1-3) may be superseded by $\gamma_R \sigma_1$ for calculation of λ_p, γ_R refers to the resistance partial coefficient and adopts 1.1 for Q235 and Q345 steel;

h_w——height of web, average height of plate is taken while tapered web used (mm);

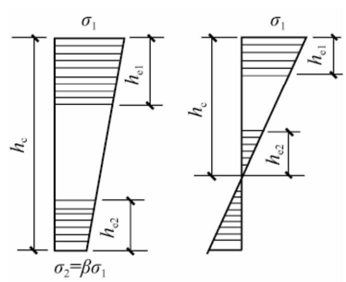
t_w——web member thickness (mm);

k_σ——buckling coefficient of member under normal stress;

β——ratio of normal stress at edge of section (Figure 7.1.1), $-1 \leqslant \beta \leqslant 1$;

σ_1, σ_2——the maximum and minimum stress of plate edge, $|\sigma_2| \leqslant |\sigma_1|$.

3 The effective width h of web shall be distributed in accordance with the following rules (Figure 7.1.1):



(a) $\beta \geqslant 0$ (b) $\beta < 0$

Figure 7.1.1 Distribution of effective width of web

Where the whole section is under compression, i.e. $\beta \geqslant 0$,

$$h_{e1} = 2h_e / (5 - \beta) \tag{7.1.1-6}$$

$$h_{e2} = h_e - h_{e1} \tag{7.1.1-7}$$

Where the partial section is under tension, i.e. $\beta < 0$,

$$h_{e1} = 0.4 h_e \tag{7.1.1-8}$$

$$h_{e2} = 0.6 h_e \tag{7.1.1-9}$$

4 For web plate of I-shaped section in shear, transverse stiffeners must be placed when post-buckling strength is considered. The ratio of length between stiffeners to the height of section at large end shall not be greater than 3.

5 For tapered member with transverse stiffeners on web, the post buckling shear strength between adjacent stiffeners shall be calculated according to the following formulas:

$$V_d = \chi_{tap}\varphi_{ps}h_{w1}t_w f_v \leqslant h_{w0}t_w f_v \tag{7.1.1-10}$$

$$\varphi_{ps} = \frac{1}{(0.51+\lambda_s^{3.2})^{1/2.6}} \leqslant 1.0 \tag{7.1.1-11}$$

$$\chi_{tap} = 1-0.35\alpha^{0.2}\gamma_p^{\ 2/3} \tag{7.1.1-12}$$

$$\gamma_p = \frac{h_{w1}}{h_{w0}} - 1 \tag{7.1.1-13}$$

$$\alpha = \frac{a}{h_{w1}} \tag{7.1.1-14}$$

Where f_v——design value of shear strength of steels (N/mm^2);

h_{w1}, h_{w0}——the heights of web on the large and small ends of tapered web (mm);

t_w——thickness of web (mm);

λ_s——parameter related to shear plate and adopted according to the requirements of Item 6 in 7.1.1;

χ_{tap}——reduction coefficient of tapering ratio of post-buckling shear strength of web;

γ_p——tapering ratio of web grid;

α——length-to-height ratio of the grid;

a——spacing between stiffeners (mm).

6 The parameter λ_s shall be calculated according to the following formulas:

$$\lambda_s = \frac{h_{w1}/t_w}{37\sqrt{k_\tau}\ \sqrt{235/f_y}} \tag{7.1.1-15}$$

For $a/h_{w1}<1$,

$$k_\tau = 4+5.34/(a/h_{w1})^2 \tag{7.1.1-16}$$

For $a/h_{w1}\geqslant 1$,

$$k_e = \eta_s[5.34+4/(a/h_{w1})^2] \tag{7.1.1-17}$$

$$\eta_s = 1-w_1\sqrt{\gamma_p} \tag{7.1.1-18}$$

$$\omega_1 = 0.41-0.897\alpha+0.363\alpha^2-0.041\alpha^3 \tag{7.1.1-19}$$

Where k_τ——the buckling coefficient of shear plate, without transverse stiffeners, it adopts $5.34\eta_s$.

7.1.2 The strength calculation of frame member and the arrangement of stiffeners shall meet the following requirements:

1 Under the combined action of shear force V and bending moment M, the flexural member with I-shaped section shall meet the requirements of the following formulas:

For $V\leqslant 0.5V_d$,

$$M\leqslant M_e \tag{7.1.2-1}$$

For $0.5V_d<V\leqslant V_d$,

$$M\leqslant M_f+(M_e-M_f)\left[1-\left(\frac{V}{0.5V_d}-1\right)^2\right] \tag{7.1.2-2}$$

For the bi-axial symmetrical section,

$$M_f = A_f(h_w+t_f)f \tag{7.1.2-3}$$

Where M_f——bending moment borne by two flanges (N·mm);

M_e——bending moment borne by effective section of the member (N·mm), $M_e = W_e f$;

W_e——section modulus of the maximum compression fiber in the effective section of member (mm^3);

A_f——sectional area of member flange (mm^2);

h_w——web height of calculated section (mm);

t_f——flange thickness of calculated section (mm);

V_d——design value of shear capacity for web (N); it shall be calculated according to formula (7.1.1-10) of this Code.

2 Under the combined action of shear force V, bending moment M and axial pressure N, resistance of the compression-bending member with I-shaped section shall meet the requirements of the following formulas:

For $V \leqslant 0.5V_d$,

$$\frac{N}{A_e} + \frac{M}{W_e} \leqslant f \tag{7.1.2-4}$$

For $0.5V_d \leqslant V < V_d$,

$$M \leqslant M_f^N + (M_e^N - M_f^N)\left[1 - \left(\frac{V}{0.5V_d} - 1\right)^2\right] \tag{7.1.2-5}$$

$$M_e^N = M_e - NW_e/A_e \tag{7.1.2-6}$$

For the bi-axial symmetrical section,

$$M_f^N = A_f(h_w + t)(f - N/A_e) \tag{7.1.2-7}$$

Where A_e——effective sectional area (mm^2);

M_f^N——bending moment borne by flanges when bearing the pressure N concurrently (N·mm).

3 The transverse stiffeners shall be set at the joint with interior column, position with relatively large concentrated load and flange turning place for beam flange in accordance with the following requirements:

1) When using the post-buckling strength, the beam web's interior stiffeners shall bear the pressure not only generated by concentrated load and flange turning but also generated by tension field, the latter one shall be calculated according to the following formulas:

$$N_s = V - 0.9\varphi_s h_w t_w f_v \tag{7.1.2-8}$$

$$\varphi_s = \frac{1}{\sqrt[3]{0.738 + \lambda_s^6}} \tag{7.1.2-9}$$

Where N_s——pressure generated by the tension field (N);

V——design value of shear capacity of beam (N);

φ_s——stability coefficient of shear buckling of web, $\varphi_s \leqslant 1.0$;

λ_s——normalized web slenderness for shear buckling of web, which is calculated according to formula (7.1.1-15) of this Code;

h_w——web height (mm);

t_w——web thickness (mm).

2) As for checking calculation of stiffener stability, its section shall include the web area within $15\,t_w\sqrt{235/f_y}$ of web on each side with effective length h_w.

4 The combined strength of axial force, bending moment and shear force shall be checked and calculated for section on small end .

7.1.3 The stability of tapered column in the plane of frame shall be calculated according to the following formulas:

$$\frac{N_1}{\eta_t \varphi_x A_{e1}} + \frac{\beta_{mx} M_1}{(1 - N_1/N_{cr}) W_{e1}} \leqslant f \tag{7.1.3-1}$$

$$N_{cr} = \pi^2 E A_{e1} / \lambda_1^2 \tag{7.1.3-2}$$

For $\bar{\lambda}_1 \geqslant 1.2$,

$$\eta_t = 1 \tag{7.1.3-3}$$

For $\bar{\lambda}_1 < 1.2$,

$$\eta_t = \frac{A_0}{A_1} + \left(1 - \frac{A_0}{A_1}\right) \times \frac{\bar{\lambda}_1^2}{1.44} \tag{7.1.3-4}$$

$$\lambda_1 = \frac{\mu H}{i_{x1}} \tag{7.1.3-5}$$

$$\bar{\lambda}_1 = \frac{\lambda_1}{\pi} \sqrt{\frac{E}{f_y}} \tag{7.1.3-6}$$

Where N_1——design value of axial pressure on large end (N);

M_1——design value of bending moment on large end (N·mm);

A_{e1}——effective sectional area on large end (mm^2);

W_{e1}——section modulus of maximum compression fiber of effective section on large end (mm^3);

φ_x——axial compressional stability coefficient of member, for tapered column it may be found in current national standard GB 50017 *Standard for Design of Steel Structures* with the help of calculated length coefficient stipulated in Appendix A of this code. Calculation of member's slenderness ratio is based on radius of curvature of its section on large end;

β_{mx}——equivalent bending moment coefficient, with sidesway frame column, it adopts 1.0;

N_{cr}——Euler critical force (N);

λ_1——slenderness ratio with effective length coefficient based on large-end section of the member;

$\bar{\lambda}_1$——normalized slenderness ratio;

i_{x1}——radius of curvature about major axis of section based on large end (mm);

μ——calculated length coefficient of the column, stipulated in Appendix A of this Code;

H——height of column (mm);

A_0, A_1——gross sectional areas of the member on small end and large end respectively (mm^2);

E——modulus of elasticity of steel column (N/mm^2);

f_y——yield strength value of steel column (N/mm^2).

Note: Where the maximum bending moment of column is not emerged on large end, M_1 should take the maximum bending moment where it appears and the effective section modulus of the section E_{e1} will take related one with that accordingly.

7.1.4 Stability of frame beam with tapered section shall meet the following requirements:

1 Stability of tapered beam stressed with linearly varied bending moment shall be calculated according to the following formulas:

$$\frac{M_1}{\gamma_x \varphi_b W_{x1}} \leqslant f \tag{7.1.4-1}$$

$$\varphi_b = \frac{1}{(1-\lambda_{b0}^{2n}+\lambda_b^{2n})^{1/n}} \tag{7.1.4-2}$$

$$\lambda_{b0} = \frac{0.55-0.25k_\sigma}{(1+\gamma)^{0.2}} \tag{7.1.4-3}$$

$$n = \frac{1.51}{\lambda_b^{0.1}} \sqrt[3]{\frac{b_1}{h_1}} \tag{7.1.4-4}$$

$$k_\sigma = k_M \frac{W_{x1}}{W_{x0}} \tag{7.1.4-5}$$

$$\lambda_b = \sqrt{\frac{\gamma_x W_{x1} f_y}{M_{cr}}} \tag{7.1.4-6}$$

$$k_M = \frac{M_0}{M_1} \tag{7.1.4-7}$$

$$\gamma = (h_1 - h_0)/h_0 \tag{7.1.4-8}$$

Where φ_b——overall stability coefficient of the tapered beam, $\varphi_b \leqslant 1.0$;

k_σ——ratio of compressive stress of the section on large end to that on small end;

k_M——ratio of smaller bending moment to larger one in the member;

λ_b——normalized slenderness ratio of the beam;

γ_x——plastic development coefficient of the section, determined according to current national standard GB 50017 *Standard for Design of Steel Structures*;

M_{cr}——critical bending moment for elastic buckling of tapered beam (N·mm), calculated according to Item 2 in 7.1.4;

b_1, h_1——width of compressive flange of the section where bending moment reaches maximum, and the distance between middle surfaces of upper and lower flanges (mm);

W_{x1}——sectional modulus of the compressive side of the section where bending moment reaches maximum(mm^3);

γ——tapering ratio of the tapered beam;

h_0——distance between middle surfaces of upper and lower flanges of the section on small end (mm);

M_0——bending moment on small end (N·mm);

M_1——bending moment on large end (N·mm).

2 Critical bending moment of elastic buckling shall be calculated according to the following formulas:

$$M_{cr} = C_1 \frac{\pi^2 E I_y}{L^2}\left[\beta_{x\eta} + \sqrt{\beta_{x\eta}^2 + \frac{I_{w\eta}}{I_y}\left(1+\frac{G J_\eta L^2}{\pi^2 E I_{w\eta}}\right)}\right] \tag{7.1.4-9}$$

$$C_1 = 0.4k_M^2 \eta_i^{0.346} - 1.32k_M \eta_i^{0.132} + 1.86\eta_i^{0.023} \tag{7.1.4-10}$$

$$\beta_{x\eta} = 0.45(1+\gamma\eta)h_0 \frac{I_{yT}-I_{yB}}{I_y} \tag{7.1.4-11}$$

$$\eta = 0.55 + 0.04(1-k_\sigma)\sqrt[3]{\eta_i} \tag{7.1.4-12}$$

$$I_{w\eta} = I_{w0}(1+\gamma\eta)^2 \tag{7.1.4-13}$$

$$I_{w0}=I_{yT}h_{sT0}^2+I_{yB}h_{sB0}^2 \tag{7.1.4-14}$$

$$J_{\eta}=J_0+\frac{1}{3}\gamma\eta(h_0-t_f)t_w^3 \tag{7.1.4-15}$$

$$\eta_i=\frac{I_{yB}}{I_{yT}} \tag{7.1.4-16}$$

Where C_1——the equivalent bending moment coefficient, $C_1 \leqslant 2.75$;

η_i——ratio of moment of inertia;

I_{yT}, I_{yB}——moment of inertia about minor axis of compressive and tensile flanges of the section respectively where the bending moment reaches maximum (mm^4);

$\beta_{x\eta}$——section asymmetry coefficient;

I_y——moment of inertia about the minor axis of the tapered beam (mm^4);

$I_{w\eta}$——equivalent moment of inertia of warping of the tapered beam (mm^4);

I_{w0}——moment of inertia of warping of section on small end (mm^4);

J_{η}——equivalent Saint-Venant torsion constant of the tapered beam;

J_0——free torsion constant of the section on small end;

h_{sT0}, h_{sB0}——distance from the central plane of upper/lower flange of section on small end to shear center respectively (mm);

t_f——thickness of flange (mm);

t_w——thickness of web (mm);

L——out-of-plane calculated length of the beam section (mm).

7.1.5 Out-of-plane stability of tapered column shall be calculated by parts according to the following formulas. when that not satisfied, lateral brace or diagonal brace shall be set and the out-of-plane stability shall be checked for each part section.

$$\frac{N_1}{\eta_{ty}\varphi_y A_{e1} f}+\left(\frac{M_1}{\varphi_b \gamma_x W_{e1} f}\right)^{1.3-0.3k_\sigma} \leqslant 1 \tag{7.1.5-1}$$

For $\bar{\lambda}_{1y} \geqslant 1.3$,

$$\eta_{ty}=1 \tag{7.1.5-2}$$

For $\bar{\lambda}_{1y} < 1.3$,

$$\eta_{ty}=\frac{A_0}{A_1}+\left(1-\frac{A_0}{A_1}\right)\times\frac{\bar{\lambda}_{1y}^2}{1.69} \tag{7.1.5-3}$$

$$\bar{\lambda}_{1y}=\frac{\lambda_{1y}}{\pi}\sqrt{\frac{f_y}{E}} \tag{7.1.5-4}$$

$$\lambda_{1y}=\frac{L}{i_{y1}} \tag{7.1.5-5}$$

Where $\bar{\lambda}_{1y}$——normalized slenderness ratio about the minor axis;

λ_{1y}——slenderness ratio about minor axis;

i_{y1}——radius of curvature about minor axis of the section on large end (mm);

φ_y——stability coefficient of axial compressive member with out-of-plane bending moment, acting at large end. Requirements of current national standard GB 50017 *Standard for Design of Steel Structures* should be followed with effective length of column adopted between bracing points in longitudinal direction;

N_1——axial pressure on large end of the calculated member (N);

M_1——bending moment on large end section of calculated member (N • mm);

φ_b——stability coefficient adopted according to 7.1.4 of this Code.

7.1.6 Design of rafter and its diagonal brace shall meet the following requirements:

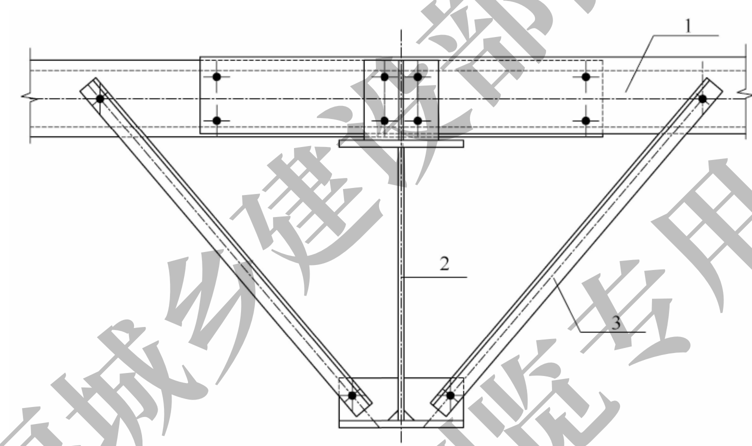
1 Rafter of solid-web frame may be designed as resisting compression and bending in plane, and must be designed as resisting compression and bending out-of-plane.

2 Out-of-plane effective length of rafter shall take the distance between lateral bracing points. When distance between lateral bracing points is unequal for two flanges, the distance for rafter shall be taken for maximum compression flange.

3 Where the lower flange of rafter is under compression, the purlin supported at the upper flange of rafter cannot served as lateral bracing point alone.

4 In case that the diagonal brace set between rafter and purlin meet the following conditions, the out-of-plane length during calculating for rafter under compression of the lower flange may be considered as having a function of supporting diagonal brace:

1) Diagonal braces are arranged on both sides of roof (Figure 7.1.6);



1—purlin; 2—steel beam; 3—diagonal brace

Figure 7.1.6 Diagonal Braces for Roof Inclined Beam

2) The position of upper supporting point of diagonal brace is not less than the centroid line of purlin;

3) Diagonal brace should conform to corresponding design requirements.

5 For one-sided layout of diagonal brace, the horizontal action of pressure exerting by the diagonal brace to the lower flange of rafter as a practical support for purlins shall be considered. Design of rafter on strength and stability should take into consideration of their influences.

6 Where the transverse stiffeners were not arranged at the section supporting concentrated load on upper flange of rafter, except converted stress under the combined action of normal stress and shear stress at top edge of web as well as local compressive stress shall be checked up according to the requirements of current national standard GB 50017 *Standard for Design of Steel Structures*, but also the requirements of the following formulas shall be meet:

$$F \leqslant 15\alpha_m t_w^2 f \sqrt{\frac{t_f}{t_w}} \sqrt{\frac{235}{f_y}} \tag{7.1.6-1}$$

$$\alpha_m = 1.5 - M/(W_e f) \tag{7.1.6-2}$$

Where F——concentrated load supported by upper flange (N);

t_f, t_w——thickness of flange and web of the rafter respectively (mm);

α_m——parameter, while $\alpha_m \leqslant 1.0$, it takes 1.0 in negative moment area of the rafter;

M——bending moment at the section where there is a concentrated load (N · mm);

W_e——section modulus of the maximum compressive fiber of the effective section (mm^3).

7 The stability coefficient of rafter supporting beam for diagonal brace shall be determined in accordance with the requirements of 7.1.4 in this Code, in which k_σ refers to the ratio of stress in large end to that in small one, and taking the stress ratio of the beam sections and tapering ratio γ both within a range of triple diagonal brace spacing, the critical bending moment of elastic buckling shall be calculated according to the following formulas:

$$M_{cr} = \frac{GJ + 2e\sqrt{k_b(EI_y e_1^2 + EI_w)}}{2(e_1 - \beta_x)} \tag{7.1.6-3}$$

$$k_b = \frac{1}{l_{kk}}\left[\frac{(1-2\beta)l_p}{2EA_p} + (a+h)\frac{(3-4\beta)}{6EI_p}\beta l_p^2 \tan\alpha + \frac{l_k^2}{\beta l_p E A_k \cos\alpha}\right]^{-1} \tag{7.1.6-4}$$

$$\beta_x = 0.45h\frac{I_1 - I_2}{I_y} \tag{7.1.6-5}$$

Where J, I_y, I_w——free torsion constant, moment of inertia about the minor axis and the moment of inertia of warping of the section on large end respectively (mm^4);

G——modulus of shear for steel of rafter (N/mm^2);

E——modulus of elasticity for steel of rafter (N/mm^2);

a——distance from the section centroid of the purlin to the upper flange centre of the beam (mm);

h——distance between the middle planes of upper and lower flange of the section on small end (mm);

α——angle included between diagonal brace and purlin axis(°);

β——ratio of distance from connecting point of diagonal brace with purlin to distance from main beam to purlin span;

l_p——span of purlin (mm);

I_p——moment of inertia about major axis of the purlin section (mm^4);

A_p——sectional area of the purlin (mm^2);

A_k——sectional area of the diagonal bracing (mm^2);

l_k——length of diagonal bracing (mm);

l_{kk}——spacing between diagonal braces (mm);

e——vertical distance from bracing point of diagonal brace to the centroid line of the purlin (mm);

e_1——distance from shear center of the beam section to the centroid line of the purlin (mm);

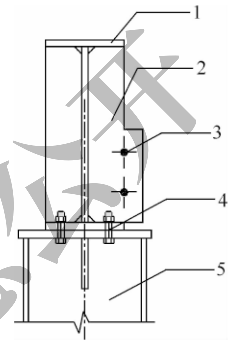
I_1——moment of inertia about axis of the flange supported by the diagonal brace (mm^4);

I_2——moment of inertia about axis of the flange connected with the purlin (mm^4).

7.2 Calculation of End Wall Members

7.2.1 The lower end of end wall column base may be supported either with hinged or rigid connection. Where the roofing material adapts to the relatively large deformation, the top of end wall column may adopt fixed connection (Figure 7.2.1) serving as the vertical hinged support for interior columns.

7.2.2 Except end wall column stated in 7.2.3 of this Code, diagonal brace should not be arranged between rafter and purlin for end wall member.



1—roof beam on the end of plant; 2—stiffener; 3—connecting hole of roof brace; 4—connection of end wall column with roof beam; 5—end wall column

Figure 7.2.1 Connection of End Wall Column with Member

7.2.3 Near by end wall column, rigid tie rod shall be set between rafter of adjacent frames. For building with roof ridge height less than 10m or the basic wind pressure not less than 0.55kN/m^2, where ridge height of the building is less than 8m, the rigid tie rod may be superseded by a double-purlin section. In such case, the connection of diagonal brace with rafter and purlin shall use high-strength bolts while connection with cold-formed purlin shall add double-sided fill plate to strengthen local bearing strength. The connection point shall not be lower than the purlin center line of steel section. Connection of rigid member with double purlins shall be checked with 3% of design value of its axial resistance along roof slope direction on strength and stability under combined force effect .

7.2.4 As a member supporting compressive load and bending, end wall columns are checked and calculated for strength and stability. Diagonal braces may be arranged between end wall column and girt. The effective length of out-of-plane flexural and torsional stability shall be no less than twice of the spacing between diagonal braces.

8 Design of Bracing

8.1 General principles

8.1.1 Complete bracing system is required for each temperature section, structural unit or section, structural unit in construction stage, and constitutes an independent steady space system with frame structure. In the construction and installation stage, arrangement of temporary constructional brace shall also meet the related requirements of Chapter 14 in this Code.

8.1.2 Brace between adjacent columns and transverse brace in roofing shall be arranged preferably in the same bay.

8.2 Design of Longitudinal Bracing along Columns

8.2.1 Column brace shall be arranged along the sidewall column. Besides, it should also be arranged along interior column in case when the building is greater than 60m in width,. Column brace must be arranged as well in building where there is crane spanned on columns at both sides.

8.2.2 Bracing set in a line of column should not consist of different types with various rigidity. They will resist horizontal load applied on columns with the load distributed in accordance with the rigidity of each brace.

8.2.3 The types adopted by column brace may be as : gabled frame, steel rod or cable in cross, rolled section steel in cross, square or round hollow section in V-type, etc.. With crane in building the rolled section steel cross brace has to be selected for brace below crane bracket.

8.2.4 Where the height of building is greater than 2 times of the column spacing, column brace should be arranged by layer. Corresponding bracing points shall be arranged at mass concentration point, connection point in crane bracket or position of low roof (if any) along the column height.

8.2.5 The column brace shall be arranged according to conditions such as longitudinal column spacing of building, force conditions and temperature section. For buildings without crane, space of column brace should take 30m～45m with end column brace arranged at the first or second bay on the end of building . For buildings with crane, the lower brace under crane bracket should be arranged in the middle position of temperature section or at one third points in case of relatively long temperature section. Besides, the brace spacing shall not be greater than 50m. For upper part upon bracket arrangement of brace is mainly identical to that with buildings where wihout crane.

8.2.6 Brace between adjacent columns shall be designed as a vertical cantilever truss supported on the column base. Rod or steel rope cross brace shall be designed in tension, rolled section may also be designed in tension. Rigid tie rod to be designed in compression.

8.3 Design of Transverse and Longitudinal Bracing in Roof

8.3.1 Transverse bracing in roof ends shall be arranged at first or second bay in ends of temperature section of building. when arranged at second bay, rigid tie rods must be arranged at the corresponding position on top of end wall column in the first bay of building.

8.3.2 Steel rod or steel rope cross brace may be used generally for roof bracing When a suspension crane is installed on the rafter nearby, rolled section cross brace shall be used for roof transverse brace. Besides, the joint connection related shall be strengthened accordingly.

8.3.3 Transverse bracing in roof shall be designed as a truss supported on top of a brace truss between adjacent columns. Steel rod or steel rope cross brace shall be designed in tension, rolled section may be designed in tension, rigid tie rod shall be designed in compression.

8.3.4 For the frame spanned for bridge crane equipped with driver cab and having lifting capacity greater than 15t, the longitudinal brace shall be arranged along the roof edge. Where column subtracted, the longitudinal bracing alone with an under-supported truss should be arranged.

8.4 Arrangement of Diagonal Brace

8.4.1 In case that the beam and column flange of solid-web gabled frame are under compression, the diagonal brace shall be arranged on the side of compression flange to connect with the purlin or girt.

8.4.2 The diagonal brace shall be designed as an axial compression member. The design value of axial force N may be calculated according to following formula. When diagonal brace arranged in pairs (two-side layout), the calculated axial force of each diagonal brace may take 1/2 of the calculated value.

$$N = Af/(60\cos\theta) \tag{8.4.2}$$

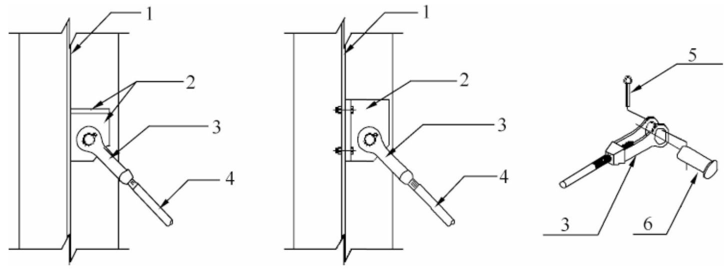
Where A ——section area of braced flange (mm^2);

f ——design value for the tensile strength of braced flange (N/mm^2);

θ ——the included angle of diagonal brace with purlin axis(°).

8.5 Connection Joint of Rod Brace with Frame

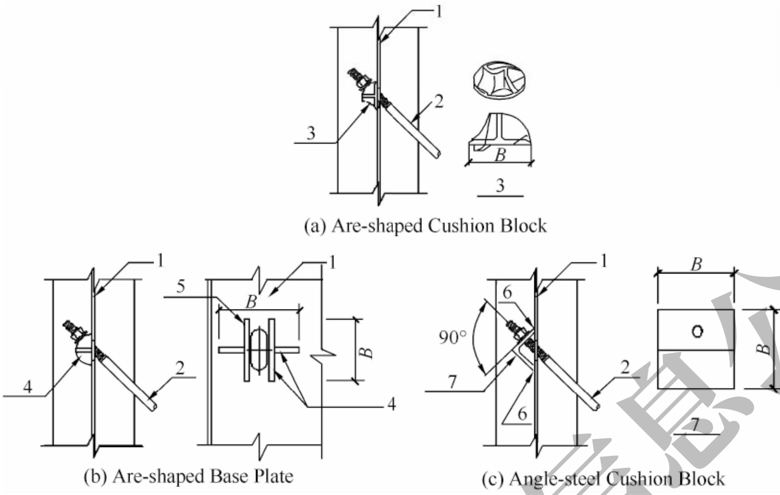
8.5.1 Connection joint of rod brace with frame may be carried out with gusset plate and pins (Figure 8.5.1).



1—web; 2—connecting plate; 3—U-shaped connecting clamp;
4—rod; 5—split pin; 6—plug pin

Figure 8.5.1 Connection of Rod Brace with Connecting Plate

8.5.2 When the rod brace is connected to the beam-column web directly, the cushion block or gusset plate shall be set up with a dimension B not less than 4 times of the rod diameter (Figure 8.5.2).



1—web; 2—rod; 3—arc-shaped cushion block; 4—arc-shaped gusset plate with thickness greater than or equal to 10mm; 5—single welding; 6—welding; 7—angle-steel cushion block with thickness greater than or equal to 12mm

Figure 8.5.2 Connection of rod brace with web

9 Design of Purlin and Girt

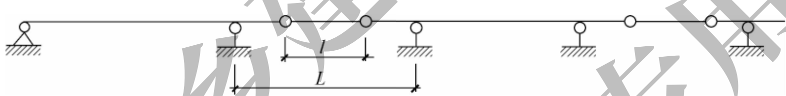
9.1 Design of solid-web purlin

9.1.1 Purlin should adopt preferably solid-web member and may also adopt truss-type member when simple-supported purlin with span greater than 9m.

9.1.2 Solid-web purlin should adopt cold-formed straight-edged or Z-shaped inclined-edged (with inclined angle equal to 60°) thin-wall section or high-frequency welded H-shaped steel.

9.1.3 Solid-web purlin may be designed as single-span simple-supported member and continuous member, the latter may be composed of lapped section. Continuous purlins were calculated on deflection and internal force considering rigidity variation caused by loosness of embedded lap.

Solid-web purlin may also adopt multi-span statically-determinate beam pattern (Figure 9.1.3), the length of interior-span of purlin l should be 0.8L, joints on the ends of purlin shall be provided with rigid connecting parts to clamp the member web for twisting resistance, and overall stability of mid-span purlin is calculated according to the purlin section between joints or inflection points in simply-supported beam pattern.



L—purlin span; l—inner-span purlin length

Figure 9.1.3 Multi-span Statically-determinate Beam Pattern

9.1.4 Width-thickness ratio of hemming of solid-web purlin should not be greater than 13 and the ratio of hemming width to flange width should be not less than 0.25 and should not be greater than 0.326.

9.1.5 Calculation of solid-web purlin shall meet the following requirements:

1 Where the roof is able to prevent purlin from lateral displacement and twisting, strength rather than global stability calculation may be conducted only. The strength may be calculated according to the following formulas:

$$\frac{M_{x'}}{W_{enx'}} \leqslant f \qquad (9.1.5\text{-}1)$$

$$\frac{3V_{y'max}}{2h_0 t} \leqslant f_v \qquad (9.1.5\text{-}2)$$

Where $M_{x'}$——the designed value of bending moment in the plane of web (N·mm);

$W_{enx'}$——the effective net section modulus (for cold-formed thin-wall steel) or net section modulus (for hot rolled section steel) (mm³) calculated according to in the plane of web (Figure 9.1.5, around x'-x' axis), the effective net cross-section of cold-formed thin-wall steel shall be calculated according to the specified method in current national standard GB 50018 *Technical Code of Cold-formed Thin-wall Steel Structures*, herein, flange buckling coefficient may adopt 3.0, web buckling coefficient may take 23.9 and hemming buckling coefficient may take 0.425; while for the section with double-purlin overlap, it may adopt the sum of effective net section modulus of two purlins multiplied by reduction coefficient 0.9;

$V_{y'max}$——the design value of shear force in the plane of web (N);

h_0——the height of straight section of the purlin web after deduction of the cold bending radius (mm);

t——the purlin thickness (mm), for double-purlin overlap, adopts the sum of two purlins thickness multiplied by reduction coefficient 0.9;

f——the design value of tensile, compressive and bending strength of steels (N/mm^2);

f_v——the design value for the shear strength of steel (N/mm^2).

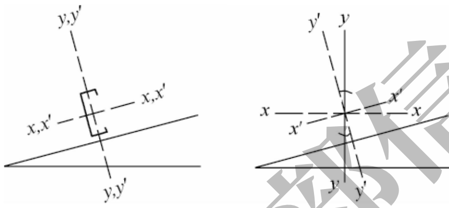


Figure 9.1.5 Calculated Inertia Axis of Purlin

2 Where the roof fails to prevent purlin from lateral displacement and twisting, stability of purlin shall be calculated according to the following formula:

$$\frac{M_x}{\varphi_{by} W_{enx}} + \frac{M_y}{W_{eny}} \leqslant f \qquad (9.1.5\text{-}3)$$

Where M_x, M_y——the design value of bending moment for principal axes x and y of section (N·mm);

W_{enx}, W_{eny}——the effective net section modulus (for cold-formed thin-wall steel) or net section modulus (for hot rolled section steel) of the principal axes x and y of the section (mm^3);

φ_{by}——the overall stability coefficient of the beam, cold-formed thin-wall steel members are calculated according to current national standard GB 50018 *Technical Code of Cold-formed Thin-wall Steel Structures* while hot rolled section steel members are calculated according to current national standard GB 50017 *Standard for Design of Steel Structures*.

3 Under the wind suction, the lower flange stability under compression shall be conducted according to those specified in current national standard GB 50018 *Technical Code of Cold-formed Thin-wall Steel Structures*, where the lower flange under compression is restrained by lining plate and is able to prevent the purlin section from twisting, the global stability may not be calculated.

9.1.6 Where the height-thickness ratio of purlin web is greater than 200, purlin bearing plate shall be set to connect the purlin web for force transmission; otherwise, purlin bearing plate may be not set, with force transmission conducted via flange brace, while the partial buckling bearing capacity of purlin shall be calculated according to the following formulas. In case of failure of meeting the following specifications, local strengthening measures shall be taken for web.

1 For purlin with flange hemming

$$P_n = 4t^2 f(1 - 0.14\sqrt{R/t})(1 + 0.35\sqrt{b/t})(1 - 0.02\sqrt{h_0/t}) \qquad (9.1.6\text{-}1)$$

2 For purlin without flange hemming

$$P_n = 4t^2 f(1-0.4\sqrt{R/t})(1+0.6\sqrt{b/t})(1-0.03\sqrt{h_0/t}) \quad (9.1.6\text{-}2)$$

Where P_n——the partial buckling bearing capacity of purlin;

t——the wall thickness of purlin (mm);

f——the design strength of purlin steels (N/mm²);

R——the inner surface radius of cold forming of purlin (mm), may take 1.5t;

b——the bearing length of purlin force transmission (mm), shall not be less than 20mm;

h_0——the height of straight section of the purlin web after deduction of the cold bending radius (mm).

3 For continuous purlin, the combined action of bending moment and partial bearing at the support of purlin shall be calculated according to the following formula:

$$\left(\frac{V_y}{P_n}\right)^2 + \left(\frac{M_x}{M_n}\right)^2 \leqslant 1.0 \quad (9.1.6\text{-}3)$$

Where V_y——the support reaction of the purlin (N);

P_n——the partial buckling bearing capacity (N) of purlin obtained via formula (9.1.6-1) or (9.1.6-2), in case of double purlins, adopts the sum of both;

M_x——the bending moment at the support of purlin (N·mm);

M_n——the flexural bearing capacity of purlin (N·mm); in case of double purlins, adopts the sum of both multiplied by reduction coefficient 0.9.

9.1.7 Where the purlin double as the transverse horizontal bracing compression rod and longitudinal tie rod along roof, the slenderness ratio of purlin shall not be greater than 200.

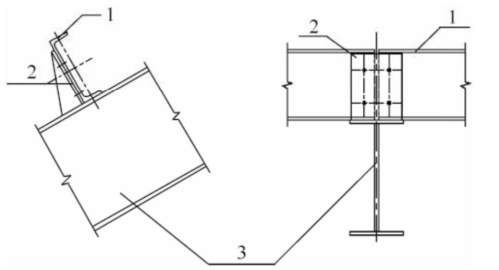
9.1.8 The purlin doubled as compression rod and longitudinal tie rod shall be calculated in accordance with compression-bending member, for the stress generated by superposed axial force in formulas (9.1.5-1) and (9.1.5-3) of this Code, its compression rod stability coefficient shall be calculated in accordance with the direction out the plane of member and calculated length shall take the spacing between tensioned rod or brace rod.

9.1.9 Common concentrated load suspended on the roof should directly act on the purlin web with bolts or self-tapping Screws, may also set cold-formed thin-wall steel between purlins as shoulder pole to support the suspended load and the shoulder pole of cold-formed thin-wall steel and purlin should be connected with bolts or tapping screws.

9.1.10 The connection between purlin and frame or that between purlin and brace shall meet the following requirements:

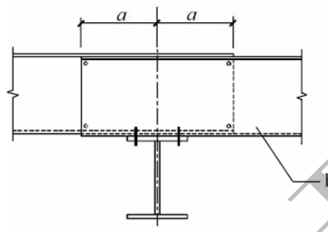
1 Roof purlin and frame rafter should be connected with ordinary bolts and each end of purlin shall be arranged with two bolts(Figure 9.1.10-1). Purlins should be connected by purlin bearing plate which shall be added with stiffening plate in case of relatively high purlin. For Z-shaped continuous purlin connected in embedded lap, if with reliable reference, may not be set with purlin bearing plate and it is connected to the frame with bolts by Z-shaped flange.

2 Overlapping length $2a$ of continuous purlin should be not less than 10% of purlin span (Figure 9.1.10-2), the purlin at nested overlapping section shall adopt bolted connection, which strength is checked and calculated according to the bending moment at the support of continuous purlin.



1—purlin; 2—purlin bearing plate; 3—roof inclined beam

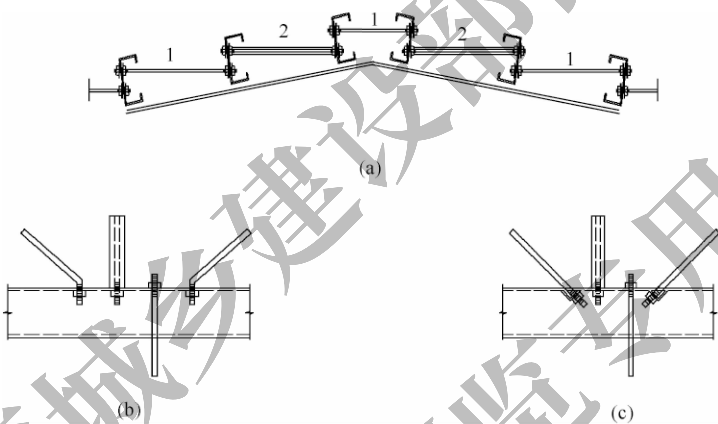
Figure 9.1.10-1 Inclined Beam Connection of Purlin with Frame



1—purlin

Figure 9.1.10-2 Overlap for Continuous Purlin

3 Tensioned rod and brace rod between purlins should directly connect to purlin web and adopt ordinary bolt connection (Figure 9.1.10-3a), and the diagonal tensioned rod ends should be buckled or arranged with cushion block(Figure s 9.1.10-3b and 9.1.10-3c).



1—tensioned rod; 2—brace rod

Figure 9.1.10-3 Connection of Tensioned Rod and Brace Rod with Purlin

4 Purlins between both sides of roof ridge may be connected with channel steel, angle steel and round steel (Figure 9.1.10-4).

(a) Connection of Roof-ridge Purlins with Channel Steel

(b) Connection of Roof-ridge Purlins with Round Steel

Figure 9.1.10-4 Connection of Roof-ridge Purlin

9.2 Truss Purlin

9.2.1 Truss purlin may adopt plane truss and plane truss purlin shall be set with tensioned rod system.

9.2.2 Plane truss purlin shall be calculated according to the following requirements:

1 All the joints shall be calculated in accordance with hinged connection while the axial force of upper/lower chord rod shall be calculated according to the following formula:

$$N_s = M_x / h \tag{9.2.2-1}$$

For upper chord rod, partial bending moment among joints shall be calculated according to the following formula:

$$M_{1x}=q_x a^2/10 \tag{9.2.2-2}$$

The axial pressure of web rod shall be calculated according to the following formula:

$$N_w=V_{max}/\sin\theta \tag{9.2.2-3}$$

Where N_s——the axial force of upper/lower chord rod of purlin (N);

N_w——the axial pressure of web rod (N);

M_x, M_{1x}——the primary moment and secondary bending moment among joints perpendicular to roof plate (N·mm);

h——the distance from upper and lower chord rods of purlin (mm);

q_x——the loading vertical to the roof (N/mm);

a——the length among joints of upper cord (mm);

V_{max}——the maximum shear force of the purlin (N);

θ——the included angle of web rod with chord rod (°).

2 Under the action of gravity load, where the roof plate is able to prevent purlin from lateral displacement, the strength checking of upper and lower chord rods shall meet the following requirements:

1) Strength of upper chord shall be checked and calculated according to the following formula:

$$\frac{N_s}{A_{n1}}+\frac{M_{1x}}{W_{n1x}}\leqslant 0.9f \tag{9.2.2-4}$$

Where A_{n1}——the net sectional area of the rod (mm^2);

W_{n1x}——the net section modulus of the rod (mm^3);

f——the design strength of steels (N/mm^2).

2) Strength of lower chord shall be checked and calculated according to the following formula:

$$\frac{N_s}{A_{n1}}\leqslant 0.9f \tag{9.2.2-5}$$

3) Web rod shall be checked and calculated according to the following formulas:

Strength

$$\frac{N_w}{A_{n1}}\leqslant 0.9f \tag{9.2.2-6}$$

Stability

$$\frac{N_w}{\varphi_{min}A_{n1}}\leqslant 0.9f \tag{9.2.2-7}$$

Where φ_{min}——the axial compression stability coefficient of web rod, referring to the smaller value of (φ_x, φ_y), the calculated length adopts the distance between joints.

3 Under the action of gravity load, where the roof fails to prevent purlin from lateral displacement, out-of-plane stability of upper chord shall be calculated according to the following formula:

$$\frac{N_w}{\varphi_y A_{n1}}+\frac{\beta_{tx}M_{1x}}{\varphi_b W_{n1xc}}\leqslant 0.9f \tag{9.2.2-8}$$

Where φ_y——the axial compression stability coefficient of upper chord, the calculated length adopts the distance between lateral bracing points;

φ_b——the even flexural overall stability coefficient of upper chord, the calculated length adopts the distance between lateral bracing points. For $I_y \geqslant I_x$, φ_b may take 1.0;

β_{tx}——the equivalent bending moment coefficient, may take 0.85;

W_{n1xc}——the net section modulus of compression fiber of upper chord under the action of M_{1x} (mm^3).

4 Under the wind suction, the out-of-plane stability of lower chord shall be calculated according to the following formula:

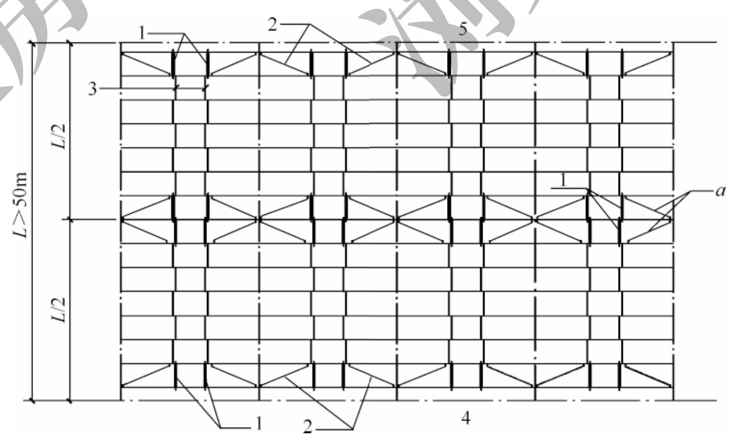
$$\frac{N_s}{\varphi_y A_{n1}} \leqslant 0.9f \tag{9.2.2-9}$$

Where φ_y——the out-of-plane under compression stability coefficient of lower chord, the calculated length adopts the distance between lateral bracing points.

9.3 Design of Tensioned Rod

9.3.1 Solid-web purlin span should not be greater than 12m; where it is greater than 4m, the tensioned rod or brace rod should be set at mid-span position among purlins; greater than 6m, each tensioned rod or brace rod should be set at the three dividing points of purlin span respectively; greater than 9m, each tensioned rod or brace rod should be set at the four dividing points of purlin span respectively. Truss structure system composed of diagonal tensioned rod and rigid brace rod shall be arranged at cornice and roof ridge respectively (Figure 9.3.1), while the diagonal tensioned rod and rigid brace rod may be omitted there under the guarantee of mutual tie balance between tensioned rod at the roof ridge.

In case that the single slope length is greater than 50m, a truss structure system (Figure 9.3.1) composed of bidirectional diagonal tensioned rod and rigid brace rod should be added in the middle.



1—rigid brace rod; 2—diagonal tensioned rod; 3—tensioned rod; 4—cornice; 5—roof ridge; L—single slope length; a—bidirectional diagonal tensioned rod and rigid brace bar system composed of diagonal tensioned rod and rigid brace rod

Figure 9.3.1 Bidirectional Diagonal Tensioned Rod and Brace Rod System

9.3.2 The slenderness ratio of brace rod shall not be greater than 220; where round steel is adopted as tensioned rod, its diameter should be not less than 10mm. The tensioned rod made of round steel may be set within the range of 1/3 of web height from purlin flange.

9.3.3 The brace form of purlin brace may adopt rigid or flexible brace system, one-layer purlin brace or upper/lower two-layer purlin brace shall be arranged in accordance with the global stability.

9.3.4 The overturning moment generated by roof to purlin may be mutually balanced via the direction variation of purlin flange, in case of failure of balancing the overturning moment, it shall be transmitted to roof beam via purlin braces jointly composed of tensioned rod and diagonal tensioned rod. The size and orientation of overturning force shall be calculated as well as the bearing capacity of purlin bracing shall be checked and calculated in accordance with the roof load and slope. Where the overturning force P_L acts on the tensioned rod close to the upper flange of purlin, with the roof ridge direction as positive, shall be calculated according to the following formulas:

1 Where all the C-shaped purlin flanges face towards the same direction of roof ridge:

$$P = 0.05W \tag{9.3.4-1}$$

2 Simple-supported Z-shaped purlin

For one purlin brace,

$$P_L = \left(\frac{0.224b^{1.32}}{n_p^{0.65}d^{0.83}t^{0.50}} - \sin\theta\right)W \tag{9.3.4-2}$$

For two purlin braces,

$$P_L = 0.5\left(\frac{0.474b^{1.22}}{n_p^{0.57}d^{0.89}t^{0.33}} - \sin\theta\right)W \tag{9.3.4-3}$$

For over two purlin braces,

$$P_L = 0.35\left(\frac{0.474b^{1.22}}{n_p^{0.57}d^{0.89}t^{0.33}} - \sin\theta\right)W \tag{9.3.4-4}$$

3 Continuous Z-shaped purlin

For one purlin brace,

$$P_L = C_{ms}\left(\frac{0.116b^{1.32}L^{0.18}}{n_p^{0.70}dt^{0.50}} - \sin\theta\right)W \tag{9.3.4-5}$$

For two purlin braces,

$$P_L = C_{th}\left(\frac{0.181b^{1.15}L^{0.25}}{n_p^{0.54}d^{1.11}t^{0.29}} - \sin\theta\right)W \tag{9.3.4-6}$$

For over two purlin braces,

$$P_L = 0.7C_{th}\left(\frac{0.181b^{1.15}L^{0.25}}{n_p^{0.54}d^{1.11}t^{0.29}} - \sin\theta\right)W \tag{9.3.4-7}$$

Where P ——the total design value of internal force of tensioned rod within one column spacing (N), subject to average bearing for multiple tensioned rods;

P_L——the design value of internal force of one tensioned rod (N);

b ——the width of purlin flange (mm);

d ——the depth of purlin section (mm);

t——the wall thickness of purlin (mm);

L ——the span of purlin (mm);

θ ——the roofing slope angle (°) ;

n_p——the number of purlins in the load-carrying area of purlin brace; for $n_p < 4$, n_p adopts 4; for $4 \leq n_p \leq 20$, n_p adopts actual value; for $n_p > 20$, n_p adopts 20;

C_{ms}——the coefficient; for purlin brace at end span, C_{ms} adopts 1.05; otherwise, C_{ms} adopts 0.90;

C_{th}——the coefficient; for purlin brace at end span, C_{th} adopts 0.57; otherwise, C_{th} adopts 0.48;

W——the design value (N) of total vertical load of the roof in load-carrying area borne by purlin brace within 1 column spacing, with downward as positive.

9.4 Design of Girt

9.4.1 The girt of light-weight wall structure should adopt hemming channel-shaped or Z-shaped cold-formed thin-wall steel or high-frequency welding H-shaped steel, and such members as girt and door frame doubled as window frame should adopt hemming groove-shaped cold-formed thin-wall steel or combined rectangular section member.

9.4.2 Girt may be designed as simple-supported or continuous member with both ends supported by frame column and girt mainly bearing horizontal wind load, and the web should be placed at level. With the self-supporting at the bottom end of wallboard and reliable connection between girt and wallboard, bending moment and shear force caused by self-weight of wall surface may not be considered. Where the girt required to bear the wallboard weight, compound bending be taken into consideration.

9.4.3 For the girt span within 4m～6m, a tensioned rod should be set in mid-span; while for the girt span greater than 6m, each tensioned rod should be set at three dividing points in span respectively. The diagonal tensioned rod should be arranged at the uppermost layer girt to transmit the tension to support column or wall trestle column, while with the reliable approach for directly transmit the vertical load of wallboard to ground surface or joist, the tensioned rod for the transmission of vertical load may be omitted.

9.4.4 For the girt hanging wallboard in single side, the strength and stability shall be calculated according to the following formulas:

1 With bearing the wind pressure towards panel, girt strength may be check-calculated according to the following formulas:

$$\frac{M_{x'}}{W_{enx'}}+\frac{M_{y'}}{W_{eny'}}\leqslant f \tag{9.4.4-1}$$

$$\frac{3V_{y',max}}{2h_0t}\leqslant f_v \tag{9.4.4-2}$$

$$\frac{3V_{x',max}}{4b_0t}\leqslant f_v \tag{9.4.4-3}$$

Where $M_{x'}$, $M_{y'}$——the bending moments generated by horizontal load and vertical load respectively (N·mm); the subscript x′ and y′ respectively represent the vertical axis and horizontal axis of girt, with self-supporting at the bottom end of wallboard, $M_{y'}=0$;

$V_{x',max}$, $V_{y',max}$——the shear force generated by vertical load and horizontal load respectively (N); with self-supporting at the bottom end of wallboard, $V_{x',max}=0$;

$W_{enx'}$, $W_{eny'}$——the effective net section modulus (for cold-formed thin-wall steel) or net section modulus(for hot rolled section steel) around the vertical axis x' and horizontal axis y' respectively (mm^3);

b_0, h_0——the calculated height in vertical and horizontal direction of girt respectively (mm), adopts the distance between starting points of two circular arcs at bending section of plate;

t——the wall thickness of girt (mm).

2 Only for the girt equipped with profiled steel plate outside, the stability under the wind suction may be calculated according to the requirements of current national standard GB 50018 *Technical Code of Cold-formed Thin-wall Steel Structures*.

9.4.5 For the girt hanging wallboard on double sides, the wind pressure towards panel and the strength under the wind suction shall be calculated in accordance with 9.4.4 in this Code; with self-supporting at bottom end of wallboard on one side, $M_{y'}$ and $V_{x',max}$ may take 0.

10 Design of Connection and Joint

10.1 Welding

10.1.1 Where the minimum thickness of plate connected is greater than 4mm, the strengths for butt weld, fillet weld and partial-penetration butt weld shall be calculated according to the requirements of current national standard GB 50017 *Standard for Design of Steel Structures*. Otherwise the strength increase coefficient β_f of front fillet weld adopts 1.0. The requirements of welding quality grade shall be conformed according to those specified in current national standard GB 50205 *Code for Acceptance of Construction Quality of Steel Structures*.

10.1.2 Where the thickness of T-connection web is no greater than 8mm and conforms the following specification, the single-side fillet weld (Figure 10.1.2) may be welded with automatic or semiautomatic submerged arc.

1 Single-side fillet weld applies to the weld bearing shear only;

2 Single-side fillet weld may only be used for the non-open-air structural member bearing static load and indirectly bearing dynamic load and have to be away from high corrosion medium;

3 Weld leg size, weld throat and minimum root penetration shall be in accordance with those specified in Table 10.1.2;

4 Prequalified welding parameters and technique shall not be altered;

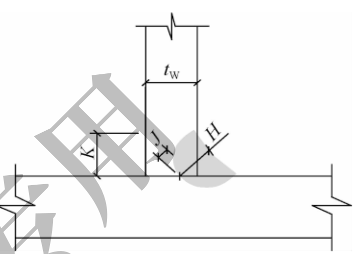


Figure 10.1.2 Single-side Fillet Weld

5 The connection of column with baseplate, bracket and beam end plate as well as crane beam and hanger supporting partial suspended load etc. shall not adopt single-side fillet weld unless otherwise specified;

6 Members of steel structure of light-weight buildings with gabled frames designed under control by seismic action shall not be connected with single-side fillet weld.

Table 10.1.2 Parameters of Single-side Fillet Weld mm

Web thickness t_w	The minimum weld leg size k	Effective thickness H	Minimum root penetration J (Diameter of welding wire 1.2～2.0)
3	3.0	2.1	1.0
4	4.0	2.8	1.2
5	5.0	3.5	1.4
6	5.5	3.9	1.6
7	6.0	4.2	1.8
8	6.5	4.6	2.0

10.1.3 For the connection of flange of frame member with end plate or column base plate, in case of flange thickness greater than 12mm, it should adopt the full-penetration butt weld and shall meet the relevant requirements of the current national standard GB/T 985.1 *Recommended Joint*

Preparation for Gas Welding, *Manual Metal Arc Welding*, *Gas-shield Arc Welding and Beam Welding* and GB/T 985.2 *Recommended Joint Preparation for Submerged Arc Welding*; Under other conditions, it should adopt fillet weld with equal-strength connection or butt-fillet joint weld and shall meet the requirements of the current national standard GB 50661 *Code for Welding of Steel Structures*.

10.1.4 Welding of upper/lower flange of bracket with column flange shall adopt groove full penetration butt weld with weld grade II, while the welding of bracket web with column flange plate shall adopt two-side fillet weld and the weld leg size shall not be less than 0.7 time of bracket web thickness.

10.1.5 Within 600mm range from upper/lower flange of the bracket on the column, the connection weld of web with flange shall adopt two-side fillet weld.

10.1.6 For the adoption of trumpet-shaped welded joint, it shall meet the following requirements:

1 The trumpet-shaped welded joint may be classified into single-side trumpet-shaped welded joint(Figure 10.1.6-1) and double-side trumpet-shaped welded joint (Figure 10.1.6-2). Therein, the weld leg size of the former shall not be less than the thickness of connected plate.

2 Where the minimum thickness of connecting plate is no greater than 4mm, strength of trumpet-shaped weld connection shall be calculated in accordance with butt weld and the shear strength of welded joint may be calculated according to following formula:

$$\tau = \frac{N}{t l_w} \leqslant \beta f_t \tag{10.1.6-1}$$

Where N ——the axial tension or axial pressure design value (N);

t ——the minimum thickness of connected plate (mm);

l_w——the effective length of welded joint (mm); equals to the weld length deducting 2 times of weld leg size;

β ——the strength reduction coefficient; where the acting force passing through welded joint centroid is vertical to the weld axis direction (Figure 10.1.6-1a), take $\beta = 0.8$; where the acting force passing through welded joint centroid is parallel to the weld axis direction (Figure 10.1.6-1b), take $\beta = 0.7$;

f_t——the design value of tensile strength of steels of the connected plate (N/mm^2).

3 Where the minimum thickness of connecting plate is greater than 4mm, strength of trumpet-shaped weld connection shall be calculated in accordance with fillet weld.

1) The shear strength of the single-side trumpet-shaped welded joint shall be calculated according to the following formula:

$$\tau = \frac{N}{h_f l_w} \leqslant \beta f_f^w \tag{10.1.6-2}$$

2) The shear strength of double-side trumpet-shaped welded joint may be calculated according to following formula:

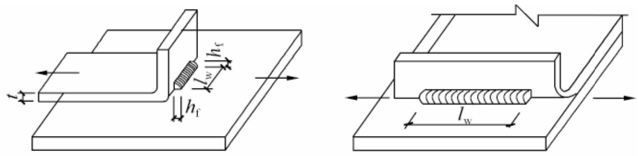
$$\tau = \frac{N}{2 h_f l_w} \leqslant \beta f_f^w \tag{10.1.6-3}$$

Where h_f——the weld leg size (mm);

β ——the strength reduction coefficient; where the acting force passing through welded joint centroid is vertical to the weld axis direction(Figure 10.1.6-1a), $\beta = 0.75$;

where the acting force passing through welded joint centroid is parallel to the weld axis direction(Figure 10. 1. 6-1 b), $\beta= 0.7$;

f_f^w——design value of strength of the fillet weld (N/mm^2).

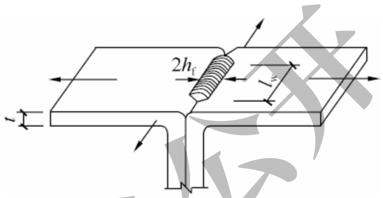


(a) Acting Force Vertical to the Weld Axis Direetion

(b) Acting Force Parallel to the wdld Axis Direction

t—minimum thickness of connected plate; h_f—weld leg size; l_w—effective length of welded joint

Figure 10. 1. 6-1 Single-side Trumpet-shaped Welded Joint



t—minimum thickness of connected plate; h_f—weld leg size; l_w—effective length of welded joint

Figure 10. 1. 6-2 Double-side Trumpet-shaped Welded Joint

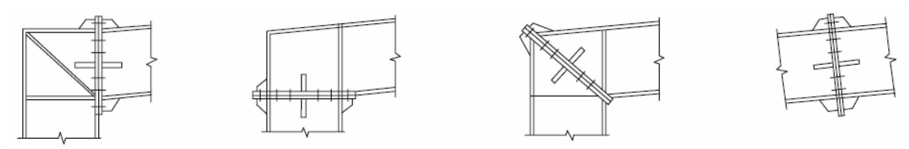
4 For combined members, trumpet-shaped welded joint among combined parts may adopt intermittent weld. The length of intermittent fillet weld segment shall not be less than $8t$ or 40mm and its clear distance among intermittent welds shall not be greater than $15t$ (for compressive member) or $30t$ (for tensile member), t refers to the minimum thickness of weldment.

10. 2 Connection Design

10. 2. 1 Connection design shall be simple and direct in force transmission, reasonable in detail, and provided with necessary ductility; it shall be convenient to welding, able to avoid stress concentration and excessive restraint stress, convenient for processing and installation, easy to take position and adjust.

10. 2. 2 Frame members may be connected with high-strength bolts and end plate. The diameter of high-strength bolt shall be determined according to stress and may adopt M16～M24 bolts. High-strength bolt bearing-type connection may be used for the structure subject to static load and indirectly subject to dynamic load, High-strength bolt friction-type connection shall be adopted for the important structures or structure subject to dynamic load, and bearing-type connection may be adopted for the power-wasting connection joint.

10. 2. 3 Connection joint of gabled frame girder with upright column, end plate may be in following three forms: upright laying (Figure 10. 2. 3a), horizontal laying(Figure 10. 2. 3b) and diagonal laying (Figure 10. 2. 3c). The tensive side of connection joint of rafter with frame column should adopt extended end plate. When end plate of the rafter vertically connected with the column flange,the later must be designed in thickness no less than of rafter's end plate (Figure 10. 2. 3d) and extended connection shall be adopted, keep the center of internal and external bolt groups of flange coincide with the flange center or nearly so. The ratio of long side to short side of triangular



(a) Upright Laying of End Plate

(b) Horizontal Laying of End Plate

(c) Diagonal Laying of End Pate

(d) Rafter Splicing

Figure 10. 2. 3 Frame Connection Joint

short stiffening plate at the connection joint should be greater than 1.5 : 1.0, when the requirements not met, plate thickness may be added.

10.2.4 Bolts on end plate shall be arranged in pairs. The distance from bolt center to flange plate surface shall meet the construction requirements for bolt tightening, which should not be less than 45mm. Distance from bolt center to surface of plate near by shall not be less than two-times of the diameter of bolt hole and distance between center of adjacent bolts shall not be less than three-times of that. In case when the distance between two pairs of bolts on end plate is greater than 400mm, a pair of additional bolts may be placed at the middle of end plate.

10.2.5 Where the connection of end plate only bear axial force and bending moment coactions with shear perhaps less than its sliding resistance, the friction surface treatment may not be conducted for the end plate surface.

10.2.6 The connection of end plate shall be designed in accordance with the larger one between maximum internal force and a force not less than half of the resistance of smaller section being connected.

10.2.7 Design of end plate connection joint shall include design of connecting bolts, determination of end plate thickness, checking and calculation of panel zone shear stress, checking and calculation of the strength of member web at end plate where bolts set and the rigidity of end plate connection, and shall meet the following requirements:

1 For connecting bolts, the strength in tension, shear force or under their combined action shall be check-calculated according to those specified in current national standard GB 50017 *Standard for Design of Steel Structures*.

2 The end plate thickness t shall be determined according to the supporting conditions. Thickness of end plate in various supporting conditions shall be calculated respectively according to the following formulas:

1) Extended part

$$t \geqslant \sqrt{\frac{6e_f N_t}{bf}} \tag{10.2.7-1}$$

2) Non-stiffened parts in grid

$$t \geqslant \sqrt{\frac{3e_w N_t}{(0.5a + e_w)f}} \tag{10.2.7-2}$$

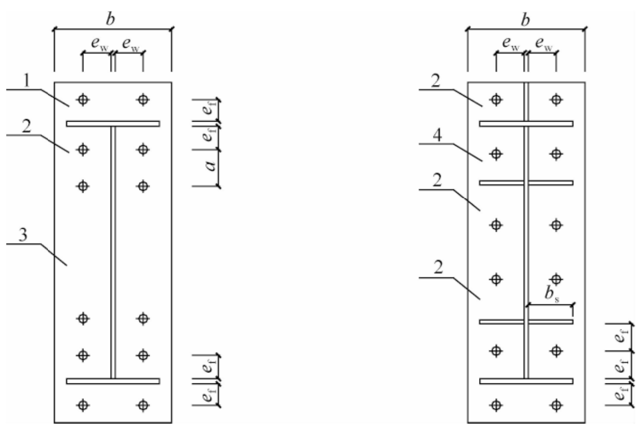
3) Two-adjacent-sides-supported grid

For outward extension of end plate,

$$t \geqslant \sqrt{\frac{6e_f e_w N_t}{[e_w b + 2e_f(e_f + e_w)]f}} \tag{10.2.7-3}$$

For end plate with flange plate parallel to and level at edge,

$$t \geqslant \sqrt{\frac{12e_f e_w N_t}{[e_w b + 4e_f(e_f + e_w)]f}} \tag{10.2.7-4}$$



1—outrigger; 2—both sides; 3—non-rib; 4—three sides

Figure 10.2.7-1 End plate Supporting Condition

4) Three-side-supported grid

$$t \geqslant \sqrt{\frac{6e_{\mathrm{f}}e_{\mathrm{w}}N_{\mathrm{t}}}{[e_{\mathrm{w}}(b+2b_{\mathrm{s}})+4e_{\mathrm{f}}^{2}]f}} \tag{10.2.7-5}$$

Where N_t——the design value of the tensile capacity of one high-strength bolt (N/mm²);

e_w, e_f——the distance from the bolt center to the web and the flange plate surface respectively (mm);

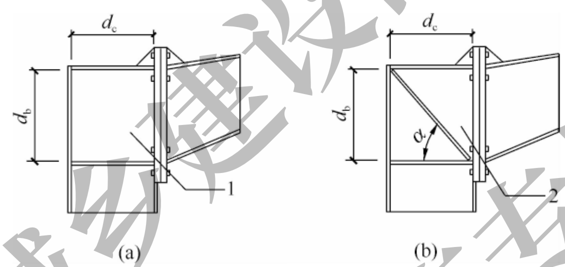
b, b_s——the width of end plate and stiffening rib plate respectively (mm);

a——the bolt spacing (mm);

f——the design value for the tensile strength of end plate (N/mm²).

5) Thickness of end plate adopted is based on the maximum value determined with various support conditions but shall not be less than 16mm and 0.8 times of the diameter of high-strength bolt.

3 For the panel zone where gabled frame rafter with column intersects (Figure 10.2.7-2a), shear stress should be checked in accordance with the following formula. When failed to meet with formula (10.2.7-6), the web plate shall be thickened and diagonal stiffening ribs be arranged (Figure 10.2.7-2b).



1—panel zone; 2—panel zone reinforced by diagonal stiffening rib

Figure 10.2.7-2 Panel zone

$$\tau = \frac{M}{d_{\mathrm{b}}d_{\mathrm{c}}t_{\mathrm{c}}} f_{\mathrm{v}} \tag{10.2.7-6}$$

Where d_c, t_c——the width and thickness of panel zone respectively (mm);

d_b——the depth of rafter or height of panel zone (mm);

M——the bending moment resisted by connection (N·mm); for interior column of multi-span frame, it shall adopt the algebraic sum of rafter end bending moment on both sides or bending moment on column end;

f_v——the design value of shear strength of panel zone steel(N/mm²).

4 The strength of member web at end plate where bolts set shall be calculated according to the following formulas:

For $N_{t2} \leqslant 0.4P$,

$$\frac{0.4P}{e_{\mathrm{w}}t_{\mathrm{w}}} \leqslant f \tag{10.2.7-7}$$

For $N_{t2} > 0.4P$,

$$\frac{N_{\mathrm{t2}}}{e_{\mathrm{w}}t_{\mathrm{w}}} \leqslant f \tag{10.2.7-8}$$

Where N_{t2}——the design value of axial tension force of one bolt in the second row inside the flange (N/mm²);

P——the design value of pretensioned force of one high-strength bolt (N);

e_w——the distance from bolt center to web surface (mm);

t_w——the web thickness (mm);

f——the design value for the tensile strength of web steel (N/mm^2).

5 The stiffness of end plate connection shall be checked according to the following requirements:

1) The stiffness of beam-to-column connection joint shall meet the requirements of following formula:

$$R \geqslant 25EI_b/l_b \tag{10.2.7-9}$$

Where R——rotational stiffness of beam-to-column connection of frame (N·mm);

I_b——moment of inertia of average cross section of frame beam in span (mm^4);

l_b——span of frame beam (mm); for interior post being leaning stanchion taking two times of distance from leaning stanchion to frame column;

E——modulus of elasticity of steel (N/mm^2).

2) The rotational stiffness of beam-to-column connection shall be calculated according to the following formulas:

$$R = \frac{R_1 R_2}{R_1 + R_2} \tag{10.2.7-10}$$

$$R_1 = Gh_1 d_c t_p + Ed_b A_{st} \cos^2\alpha \cdot \sin\alpha \tag{10.2.7-11}$$

$$R_2 = \frac{6EI_e h_1^2}{1.1e_f^3} \tag{10.2.7-12}$$

Where R_1——stiffness corresponding to shear deformation of panel zone (N·mm);

R_2——stiffness of joint as a while, including that of bending of end plate, tension of bolts and bending of column flange (N·mm);

h_1——distance between center of flanges at end section of beam (mm);

t_p——web thickness of column panel zone (mm);

I_e——moment of inertia of end plate (mm^4);

e_f——the distance from the bolt center of extended part in end plate to outer edge of stiffening rib (mm);

A_{st}——the total sectional area of two diagonal stiffening ribs (mm^2);

α——the inclination angle of diagonal stiffening ribs (°);

G——modules of shear of steel (N/mm^2).

10.2.8 The joint for leaning stanchion to roof beam connection shall be arranged as hinged connection joint with an end plate placed horizontally (Figure 10.2.8).

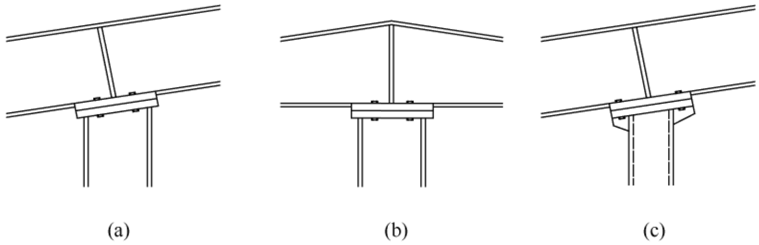


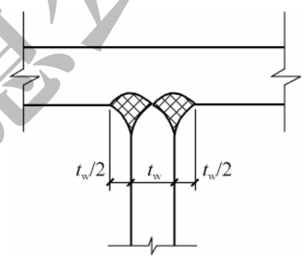
Figure 10.2.8 Connection Joint of Roof Beam with Leaning Stanchion

10.2.9 Because of bearing dynamic load, the crane runway beam structure and connection joint shall meet the following requirements:

1 The splice joint of flange and web plate for welded crane runway beam should be performed in full penetration groove butt weld with arc ending plate and polished smoothly after the arc ending plate being cut off. Single-side fillet weld is strictly forbidden for the butt weld of flange plate with web.

2 For welded crane runway beam or crane runway truss, complete penetration T joint should be performed with composite butt and fillet weld (Figure 10.2.9-1).

3 The transverse stiffener for welded crane runway beam should not be welded with tensile flange, but may be welded with compressive flange. The transverse stiffener is preferably disconnected at 50mm~100mm apart from the tensile lower flange (Figure 10.2.9-2) and the weld attached with web should not scratch start and end at lower end of the stiffneer. Where the tensile flange of crane runway beam has to connect with a brace, welding should not be adopted.



t_w—web thickness

Figure 10.2.9-1 Complete Penetration T Connection Weld

4 The crane runway beam and brake beam may be connected with high-strength bolt friction-type connection or welding. Connection of crane runway beam with the upper part of frame column should be arranged with a slotted hole (Figure 10.2.9-3a), connection of crane runway beam with the base plate of bracket should adopt welding connection (Figure 10.2.9-3b) while connection between crane runway beams should adopt high-strength bolts.

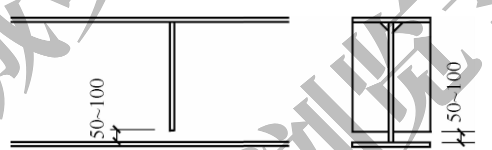
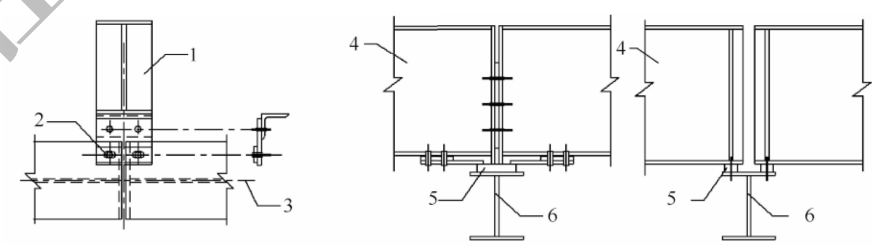


Figure 10.2.9-2 Setting of Transverse Stiffener



(a) Connection of Crane runway Beam with Upper Column

(b) Connection of Crane runway Beam with Bracket

1—the upper column; 2—slotted hole; 3—center line of crane runway beam;

4—crane runway beam; 5—base plate; 6—bracket

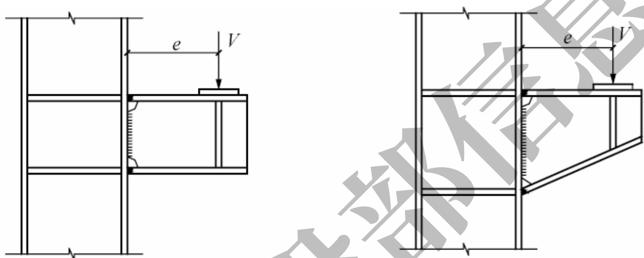
Figure 10.2.9-3 Connection Joint of Crane runway Beam

10.2.10 Bracket supporting crane runway beam may be designed of constant section or variable section. In case when variable section bracket is adopted, the section height at outer end of cantilevered bracket shall not be less than 1/2 of root height (Figure 10.2.10). Transverse stiffeners shall be set at proper position of column corresponding to upper/lower flanges of the bracket, Base plate shall be set at the support of crane runway beam on upper flange of bracket and connected with periphery welds. Transverse stiffeners shall be set at bracket web section corresponding to crane runway beam support. Where the bracket is connected with column, the strength of member sections and adjoining welds must be calculated on acting shear V and bending moment M according to those specified in current national standard GB 50017 *Standard for Design of Steel Structures*. The bending moment M shall be calculated according to the following formula.

$$M = Ve \qquad (10.2.10)$$

Where V ——the shear Force transferred from crane runway beam (N);

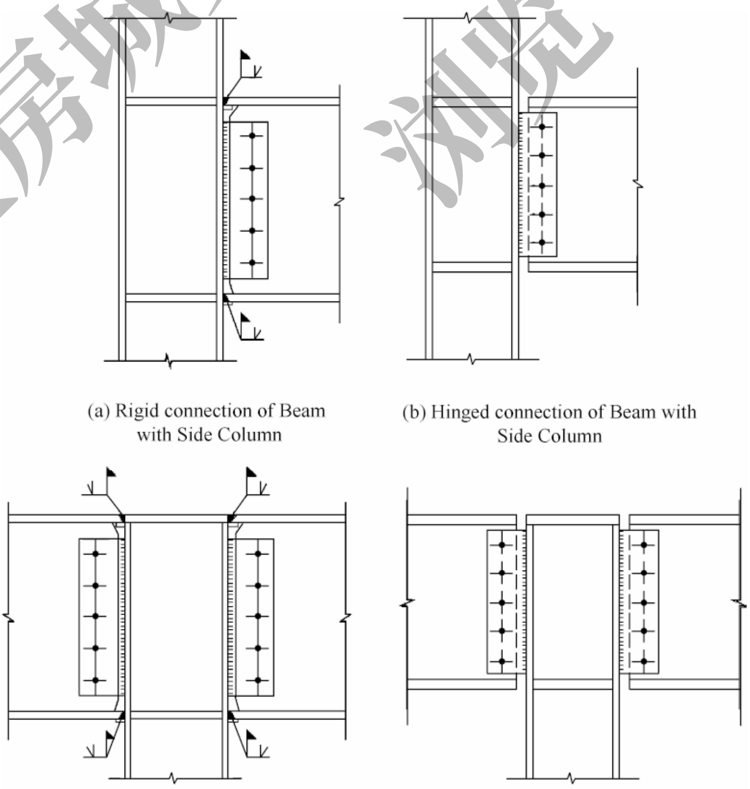
e ——the distance from center line of crane runway beam to column surface (mm).



(a) Bracket with Constant Section　　(b) Bracket with Variable Section

Figure 10.2.10　Bracket Connections

10.2.11 For the structure arranged with mezzanine, beam and column of mezzanine may be connected with rigid joint, but hinged connection is allowed (Figure 10.2.11). When rigid connections are adopted, beam flange to column flange shall be connected with full-penetration

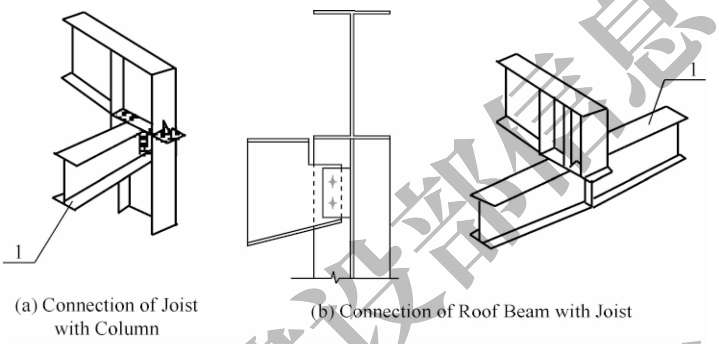


(a) Rigid connection of Beam with Side Column　　(b) Hinged connection of Beam with Side Column

(c) Rigid connection of Beam with Central Column　　(d) Hinged connection of Beam with Central Column

Figure 10.2.11　Connection Joint of Mezzanine Beam with Column groove welds and connection of web with column in bolts. Horizontal stiffener shall be set at the position of the column corresponding to upper/lower flanges of the beam.

10.2.12 Where column is deducted, bearing truss or bearing beam should be arranged with hinge support on column (Figure 10.2.12a). If bearing truss or bearing beam were quite large, rigid connection may adopted, but the resulting bending moment on column shall be considered. Roof beam supported on bearing truss or bearing beam commonly set in hinged connection (Figure 10.2.12 b). When rigid connection is selected, bearing beam section must be designed capable to resist twisting. Moreover, the connection of bearing truss or bearing beam shall consider the horizontal thrust generated by roof beam.



1—joist

Figure 10.2.12 Connection Joint of Joist

10.2.13 The upright column of parapet wall may be directly welded on roof beam (Figure 10.2.13). It should be analyzed for internal stress as a cantilever member and connected at roof beam with well designed welds.

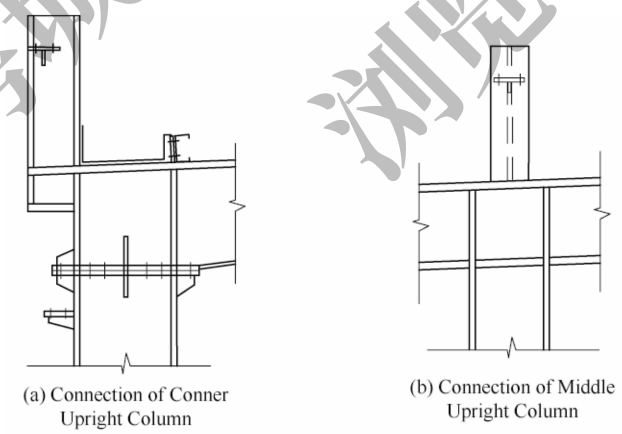


Figure 10.2.13 Connection Joint of Parapet Wall

10.2.14 Clerestory or skylight may directly welded at roof beam or channel bearing member (Figure 10.2.14), When clerestory are arranged in identical space with main frames of building, channel bearing beam may be cancelled. The support and connection of clerestory should be calculated.

10.2.15 Column base joint shall meet the following requirements:

1 Base of gabled frame column should adopt flat plate type hinged column base(Figure 10.2.15-1); and may also adopt rigid column base (Figure 10.2.15-2).

2 Calculation of upward reaction for anchor bolts of column base should consider the reaction

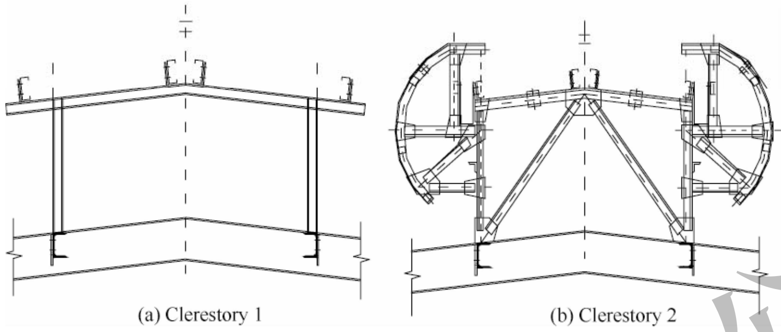
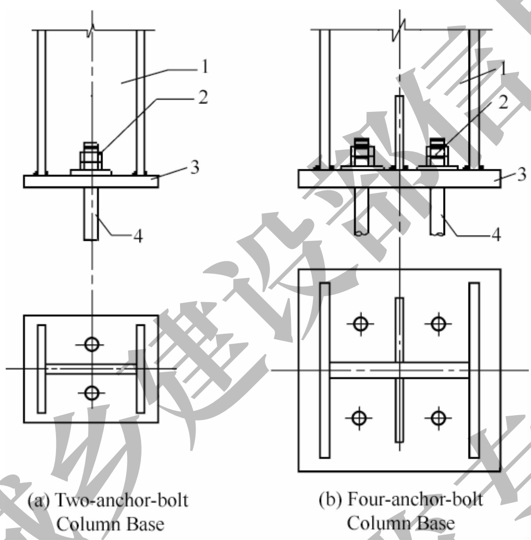
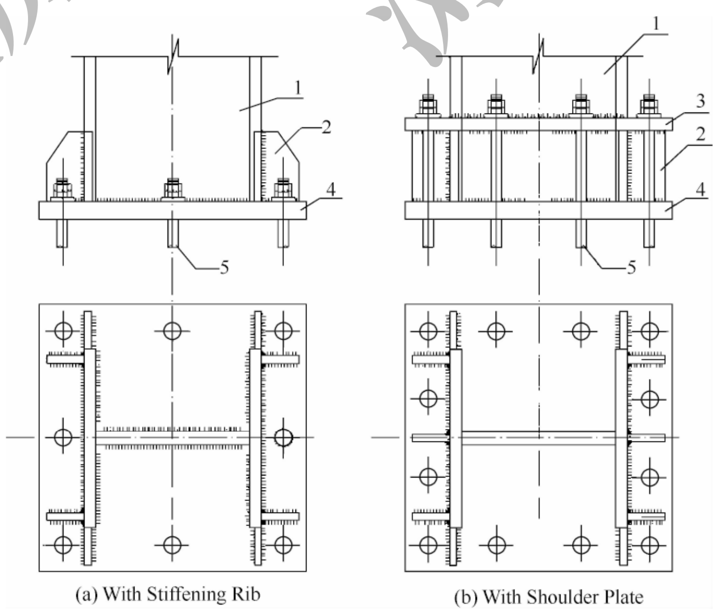


Figure 10.2.14　Detail Drawing of Clerestory



1—column; 2—double nuts and base plate; 3—baseplate; 4—anchor bolt

Figure 10.2.15-1　Hinged Column Base



1—column; 2—stiffening plate; 3—anchor bolt support bracket; 4—baseplate; 5—anchor bolt

Figure 10.2.15-2　Rigid Connected Column Base effect of brace between adjacent columns under the action of wind load. The maximum vertical component force generated by column brace shall be counted in, excluding the effect of live load, snow load, dust load and additional load. The partial coefficient of dead load shall take 1.0. For calculation of tensile capacity of column base anchor, the effective sectional area at the screw thread shall be adopted.

3 Anchor bolts set for base plate with shoulder plates is unsuitable for shear resistance. Resistance on shear should rely on friction force among baseplate and concrete foundation. The friction coefficient may take 0.4. In case considering friction force, the effect of upward force generated by roof wind suction shall be considered. When the shear force is resisted by anchor bolt without shoulder plates in base plate, the nut and gusset plate should be tightly fixed and with welds. The shear resistance of column base may take accordingly 0.6 times of shear capacity of anchor. In case when existent shear of column may be greater than shear capacity, shear key shall be set.

4 Anchors at column base shall be made of Q235 or Q345 steel. Hook or anchor bolt shall be set at anchor bolt end and shall meet the requirements of the current national standard GB 50010 *Code for Design of Concrete Structures*. The minimum anchorage length l_a (projected length) of anchor bolt shall be in accordance with those specified in Table 10.2.15 and shall not be less than 24mm. Diameter of anchor bolt d should be not less than 24mm and double nuts shall be adopted.

Table 10.2.15 Minimum Anchorage Length of Anchor Bolt

Anchor bolt steels	Concrete strength grade					
	C25	C30	C35	C40	C45	≥C50
Q235	$20d$	$18d$	$16d$	$15d$	$14d$	$14d$
Q345	$25d$	$23d$	$21d$	$19d$	$18d$	$17d$

11 Design of Enclosure System

11.1 Design of Roofing and Cladding

11.1.1 Such metal plates as galvanized or coated steel sheets, stainless steel sheet, Al-Mg-Mn alloy plate, Ti-Zn plate and copper plate or other plates made of lightweight materials may be selected for the roofing and cladding .

11.1.2 The calculation and construction for color-coated profiled steel sheet of roof and wall surface used for general building shall be in accordance with those specified in current national standard GB 50018 *Technical Code of Cold-formed Thin-wall Steel Structures*.

11.1.3 Types of the connection between the roofing and purlin may be classified into: standing seam, buckled and screwed.

11.1.4 Material property of the roofing and cladding shall meet the following requirements:

1 The mechanical property of base plate for roofing and claddingfor which color-coated profiled steel sheet shall meet the requirements of the current national standard GB/T 12755 *Profiled Steel Sheet for Building*, whose yield strength shall not be less than 350N/mm^2 and that for the base plate in buckled connection shall not be less than 500N/mm^2.

2 The zinc coating amount of hot-dip galvanizing substrate shall not be less than 275g/m^2 and coating shall be adopted; while that for aluminized and galvanized substrate shall be not less than 150g/m^2 and shall meet the requirements of the current national standard GB/T 12754 *Prepainted Steel Sheet* and GB/T 14978 *Continuously Hot-dip Aluminum-zinc Alloy Coated Steel sheet and Strip*.

11.1.5 The thickness of substrate for external plate of roof and wall surface shall be not less than 0.45mm and that for internal plate no less than 0.35mm.

11.1.6 Roofing shall not serve as the lateral brace of purlin in case of standing seam or buckled connection, while it may serve as lateral brace in case of screw connection.

11.1.7 Point or strip lighting panel may be arranged on metal plate roof where natural lighting is required for interior building; where strip light panel is adopted, measures of releasing temperature deformation shall be taken.

11.1.8 Where metal plate roofing is connected with the matched roof light panel, effective sealant must be adopted in length and width directions and the connection method should be consistent with that between metal plates.

11.1.9 As for material of accessories above the metal roof, aluminum alloy or stainless steel should be preferentially selected, and reliable water-proof measures shall also be taken for connection between roofing.

11.1.10 The lapped position of roofing along plate length direction should be on roof purlin with lapped length of no less than 150mm and water-proof treatment shall be conducted for lapped joints; lapped length of cladding shall be not less than 120mm.

11.1.11 Roof drainage slope shall be not less than the limit specified in Table 11.1.11:

Table 11.1.11 Limit of Roof Drainage Slope

Connection type	Roof slope for drainage
Plate in standing seam connection	1/30
Plate in buckling and screw connection	1/20

11.1.12 Under the action of wind load, uplift bearing capacity between roofing/cladding and purlin shall have reliable basis.

11.2 Isolation and Heat Preservation

11.2.1 Under the premise of energy saving and environmental protection, isolation and heat preservation materials with smaller heat conductivity coefficient shall be selected for the isolation and heat preservation of the roof and wall surface of light-weight building with gabled frames, which shall be designed in combination with water-proof, damp-proof and fire-proof requirements. Lightweight fibrous isolation material and organic foam materials shall be mainly selected for heat preservation of steel buildings while lightweight block or aerated concrete plate may be adopted for wall surface.

11.2.2 The isolation and heat preservation structure of roof and wall surface shall be determined according to heat engineering calculation. Material for isolation and heat preservation shall be mutually matched.

11.2.3 One of the following methods may be adopted for isolation and heat preservation of roof:

1 The glass fiber felt or mineral wool felt coiled material with aluminum-foil damp-proof layer shall be arranged under profiled steel sheet; where damp-proof layeris not reinforced by fiber, such materials with tensile resistance as steel wire mesh or glass fiber fabric at the bottom to bear the self-weight of heat preservation material;

2 Composite sandwich plate with metal covers;

3 Isolation material is filled in the middle of double-layer profiled steel sheet;

4 Rigid foaming isolation material is laid on profiled steel sheet and heat-fusion flexible water-proof coiled material is laid externally.

11.2.4 One of the following methods may be adopted for isolation and heat preservation of exterior wall:

1 The same isolation and heat preservation method as that for roof;

2 Profiled steel sheet is adopted for outer side, precast plate and gypsum plasterboard or other fiberboard for inner side, and isolation materials for middle;

3 Aerated concrete block or aerated concrete plate is adopted, with the outer side painted with water-proof coating;

4 Such lightweight masonry as perforated brick is adopted.

11.3 Design of Roof Drainage

11.3.1 Rectangle or trapezium sections may be adopted for the gutter. The exterior gutter may be made of colorful metallic coating steel sheet with thickness of no less than 0.45mm. The interior gutter should be made of stainless steel with thickness of no less than 1.0mm. Reliable anti-corrosion treatment shall be conducted where other materials are adopted; steel sheet thickness of the gutter of ordinary steel sheet shall be not less than 3.0mm.

11.3.2 Gutter shall meet the following construction requirements:

1 Gutter at expansion joint or settlement joint of buildings shall be arranged with deformation seam correspondingly.

2 Roofing shall be extended to gutter. Where interior gutter is adopted, sealing measures shall be taken for its connection between roofing.

3 Overflow port shall be arranged for interior gutter with top 50mm～100mm lower than upper eave. Downpipe quantity shall be properly increased where overflow port fails to be arranged.

4 Where internal drainage is selected for roofing drainage, mesh enclosure shall be arranged outside collect pan to prevent downpipe from garbage blocking.

11.3.3 As for downpipe, round or square section may be adopted, and metallic coating steel sheet, stainless steel and PVC may be adopted as its material. Collect pan and gutter shall be tightly connected, while downpipe and wall surface structure or other member shall be reliably connected.

12 Protection of Steel Structure from Corrosion

12.1 General Requirements

12.1.1 Steel structure of light-weight building with gabled frames shall be subjected to fire resistance and anti-rust design. As for antirust design, reasonable anti-rust coating design scheme shall be determined according to importance of structural members, classification of atmospheric environment aggressiveness as well as design service life of protective layer.

12.1.2 The designed service life of protective layer shall be not less than 5 year; that for steel member difficult to maintain in application shall be not less than 10 years.

12.1.3 The regular inspection and maintenance requirement for steel structure shall be indicated in steel structure design document.

12.2 Design of Fire Resistance

12.2.1 Design of fire resistance and fire endurance limit of steel member shall meet the requirements of the current national standard GB 50016 *Code for Fire Protection Design of Buildings* to reasonably determine the type and grade for fire resistance of buildings.

12.2.2 Prior to construction of fire-resistance coating, steel member shall be subjected to rust removal according to the requirements of 12.3 in this Code, and anti-rust primer painting. Fire-resistance coating shall be compatible with primer and combined well.

12.2.3 Such factors as form, property and thickness of fire-resistance coating shall be determined according to fire endurance limit of steel member.

12.2.4 The bonding strength and compression strength of fire-resistance coating shall meet the design requirements while corresponding inspection method shall meet the requirements of the current national standard GB/T 9978 *Fire-resistance Tests—Elements of Building Construction*.

12.2.5 Where fire-resistance structure of wrapped plate is adopted steel member shall be subjected to rust removal and primer/finish coating according to the requirements of 12.3 in this Code; and fire resistance of such structure shall meet the requirements of the current national standard GB 50016 *Code for Fire Protection Design of Buildings* or determined through test.

12.2.6 Where fire-resistance structure of wrapped concrete is adopted, steel member shall be subjected to rust removal, without anti-rust paint; its wrapped concrete thickness and structure requirements shall meet the requirements of the current national standard GB 50016 *Code for Fire Protection Design of Buildings*.

12.2.7 As for steel member directly bearing vibration, structure reinforcement measures shall be taken where thick coating or wrapped structure is adopted.

12.3 Coating

12.3.1 In the design, the substrate type, surface de-rusting grade, coating structure, coating thickness, coating method, service condition and expected anti-rust life shall be taken into comprehensive consideration to propose reasonable de-rusting method and coating requirements.

12.3.2 The initial rusting grade, de-rusting method and grade requirement on steel surface shall meet the requirements of the current national standard GB/T 8923.1 *Preparation of Steel Substrates before Application of Paints and Related Products—Visual Assessment of Surface Cleanliness—Part 1: Rust Grades and Preparation Grades of Uncoated Steel Substrates and of Steel Substrates after Overall Removal of Previous Coatings*.

12.3.3 As for load-bearing member in weak- and middle-corrosion environment, its surface shall be subjected to jet or projectile de-rusting before plant fabrication and coating with de-rusting grade no lower than Sa2; where manual and mechanical de-rusting is adopted on site, the de-rusting grade shall not be lower than St2. The type of anti-rust paint and steel surface de-rusting grade shall be matched and shall be in accordance with those specified in Table 12.3.3.

Table 12.3.3 Minimum De-rusting Grade of Steel Substrates

Painting variety	De-rusting grade
Such primer or anti-rust paint as oily phenolic and alcohol acid	St2
Such primer or anti-rust paint as high chlorinated polyethylene, chlorinated rubber, chlorosulfonated polyethylene, epoxy resin and polyurethane	Sa2
Such primer as inorganic zinc-rich, silicone and perchloroethylene	Sa $2\frac{1}{2}$

12.3.4 The rust removal and coating engineering of steel structure shall be conducted after the fabrication quality of members is accepted. The interval after surface treatment and before primer coating shall not exceed 4h, during which the surface shall be free from welding slag, dust, oil stain, water or burrs.

12.3.5 Variety of coating material shall be reasonably selected according to classification of environment corrosivity and designed service life of steel structure coating system.

12.3.6 Where environmental corrosion is classified into weak- and middle-corrosion, the total thickness of dry paint film of steel structure indoors and outdoors should not be less than 125μm and 150μm and 20μm~40μm coating thickness should be increased for position outdoors and that with special requirements; as for primer thickness, steel structure indoors should not be less than 50μm while outdoors should not be less than 75μm.

12.3.7 Coating shall be conducted in clean environment with appropriate temperature and humidity. The solidification temperature of coating shall meet the requirements of coating product instruction, if the product instruction has no requirement, the temperature should be between 5℃~38℃. Coating shall not be conducted where the relative humidity in the construction environment is greater than 85%. Film solidification duration is related to environment temperature, relative humidity and variety of coating material; after each coating, surface shall be free from rain and contamination within at least 4h.

12.3.8 The inspection method for quality and thickness of coating layer shall be in accordance with those specified in current national standard GB 1720 *Method of Test for Adhesion of Paint Films* or GB/T 9286 *Paints and Varnishes—Cross Cut Test for Films*, and 1% member quantity shall be randomly taken for inspection and shall be not less than 3 pieces, 3 points for each piece.

12.3.9 The marking, identification and number of the member after completion of coating shall be legible and integral.

12.3.10 Coating engineering acceptance shall be included in intermediate inspection and completion acceptance.

12.4 Requirements of Steel Structure on Anti-rust

12.4.1 Sectional form of solid-web or closed member easy for coating and maintenance should be adopted and closed section shall be sealed; where the bar with binding section is adopted, the gap width between profiled steel shall meet the requirements of coating construction and maintenance.

12.4.2 Surface hot-dip galvanizing or aluminizing and galvanizing should be adopted for anti-rust of such cold-formed thin-wall members as roof purlin, wall beam, diagonal brace and tensioned rod, as well as profiled steel sheet.

12.4.3 The anti-corrosion requirements for connecting parts and members with hot-dip galvanizing protective measures shall not be lower than those of the main structure, and the same anti-corrosion measures as that of main structure should be adopted after installation; sealing measures shall be taken for gap in environment not lower than weak corrosion.

12.4.4 Where galvanizing is taken for anti-rust, the double-side zinc coating mass on surface of indoor steel member shall be not less than $275g/m^2$; that for outdoor member shall be not less than $400g/m^2$.

12.4.5 Isolation measures to prevent contact corrosion shall be taken for contact position between different metal materials.

13 Fabrication

13.1 General Requirements

13.1.1 The sampling re-inspection of steel, inspection and acceptance of welding material and fabrication of steel structure shall be in accordance with those specified in current national standard GB 50205 *Code for Acceptance of Construction Quality of Steel Structures* and GB 50755 *Code for Construction of Steel Structures*.

13.1.2 Steel, auxiliary materials as well as connecting and coating material adopted for steel structures shall be provided with quality certificate document, and shall be in accordance with those specified in design documents and relevant current national standards.

13.1.3 Prior to fabrication of steel member, processing document shall be prepared to establish reasonable process flow and develop quality assurance system according to the requirements of design document and construction detail drawing, and technical conditions of fabricating organization.

13.2 Processing of Steel Member

13.2.1 Material setting out, marking off, cutting as well as indication shall be in accordance with the design and technology requirements.

13.2.2 Such welding materials as welding electrode and welding wire shall be cleanly and intactly stacked in dry welding materials storeroom classified according to material, type and specification.

13.2.3 During welding of member with H-shaped section, the flange, web plate and end plate must be calibrated to be straight and flat. The members with overlarge welding deformation shall be corrected by cool-working or local heating method.

13.2.4 Over-weld hole should be machined by lock mouth or may also be cut via scribing with planeness, cut depth and local notch depth in accordance with those specified in the current national standard GB 50205 *Code for Acceptance of Construction Quality of Steel Structures*.

13.2.5 As for the same hole group in larger quantity on thicker steel sheet, hole should be made by drilling while by punching for thinner steel sheet and cold-formed thin-wall steel member. Spacing between centers of two holes on cold-formed thin-wall steel member shall not be less than 80mm.

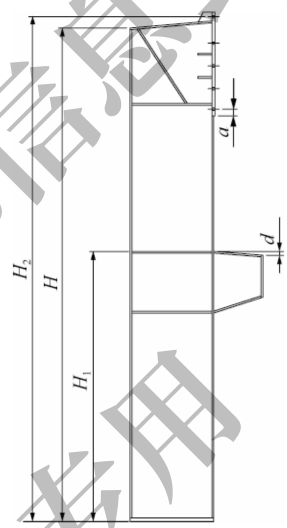
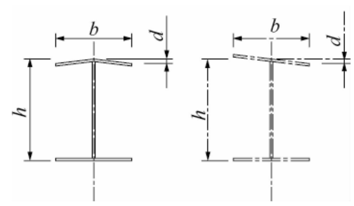
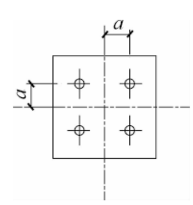
13.2.6 The cutting and shear surface of cold-formed thin-wall steel shall be free from crack, sawtooth and non-designed broken corner greater than 5mm. The permissible deviation of cold-formed thin-wall steel cutting shall be ±2mm.

13.3 Configuration and Geometry of Members

13.3.1 Configuration of steel member shall be free from visible bending deformation and flange plate and end edge shall be straight and flat. Flange and web surface shall be free from visible concave-convex surface, damage and scratch, weld beading, oil stain, sand and burrs.

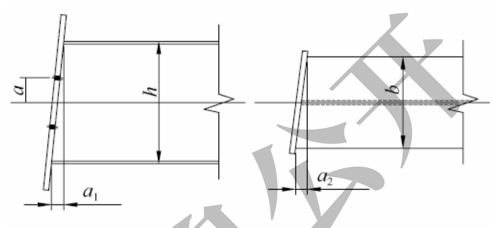
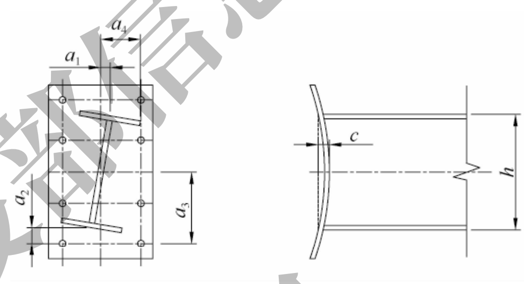
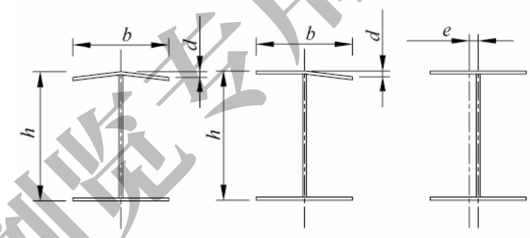
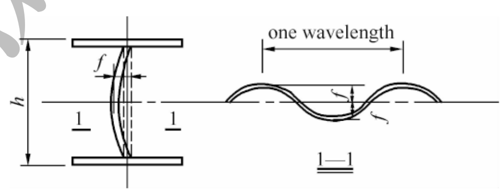
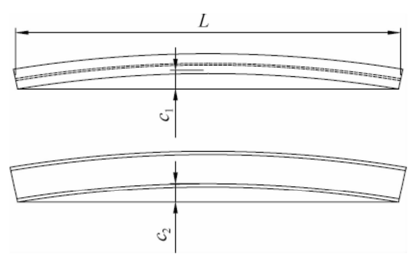
13.3.2 The deviation for overall dimension of single-layer steel column shall not be greater than that specified in Table 13.3.2.

Table 13.3.2 Permissible Deviation for Overall Dimension of Single-Layer Steel Column

No.	Item	Permissible deviation (mm)	Legend
1	Spacing from the column bottom surface to the uppermost installation hole connected with the inclined beam, at the column end (H_2)	$\pm H_2/1500$ ± 5.0	
2	Spacing from column bottom surface to the supporting surface of bracket (H_1)	$\pm H_1/2000$ ± 4.0	
3	Spacing from surface of load-carrying support plate to the first installation hole (a)	± 1.0	
4	Warpage of bracket surface (d)	2.0	
5	Torsion of column body: bracket/other positions	3.0 5.0	
6	Width and height of column section	$+3.0$ -2.0	
7	Bending rise of column body (f)	$H/1000$ 9.0	
8	Verticality of flange plate to web (d): Connection places Other positions	1.5 $b/100$ 3.0	
9	Planeness of column base plate	3.0	
10	Spacing between center of column base bolt hole and column axis(a)	2.0	

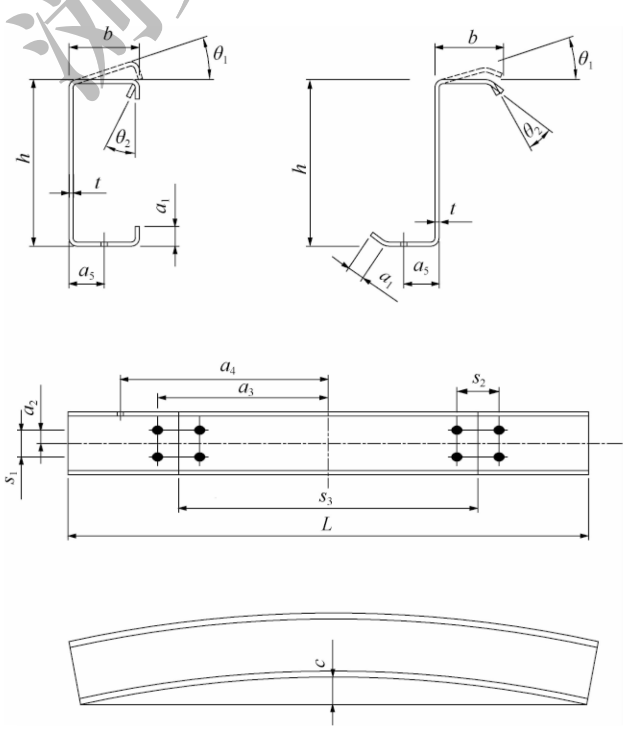
13.3.3 The deviation for overall dimension of welding solid-web beam shall not be greater than that specified in Table 13.3.3.

Table 13.3.3 Permissible deviation for Overall Dimension of Welding Solid-web Beam

No.	Item	Permissible deviation (mm)	Legend
1	Spacing between beam center line and the first bolt hole close to the line (d)	±1.0	
2	Inclination of end plate and flange plate(a_1, a_2) $h\leqslant300$; $b\leqslant200$ $h>300$; $b>200$	 ±1.0 ±1.5	
3	Spacing between upper and lower flange midpoint of beam to beam center line (a_1, a_2)	±3.0	
4	Distance between center of external angle hole on end plate to beam center (a_3, a_4)	±1.5	
5	Concave flexibility of end plate (c)	h/300	
6	Inclination of flange plate(d) Connection place: Other positions:	 2.0 3.0	
7	Length and width of column section	+3.0, −2.0	
8	Spacing between web and flange center line(e)	2.0	
9	Partial unflatness of web (f) and plate thickness: (mm) 6～10 10～12 ≥14	h/100 5.0 4.0 3.0	
10	Side bending and arch bending (c_1, c_2) $L\leqslant9$m $L>9$m	 6.0 9.0	
11	Length of beam (L)	±L/2000 ±10.0	
12	Distortion	h/250 10.0	

13.3.4 The deviation for overall dimension of purlin and girt shall not be greater than that specified in Table 13.3.4.

Table 13.3.4 Permissible Deviation for Overall Dimension of Purlin and Girt

No.	Item	Symbol	Permissible deviation (mm)
1	Section height	h	±3
2	Flange width	b	+5 −2
3	Length of inclined or right-angle crimping	a_1	+6 −3
4	Unevenness of flange	θ_1	±3°
5	Angle of inclined crimping	θ_2	±5°
6	Spacing from web hole center to the member center line	a_2	±1.0
7	Spacing from web hole center to the member center	a_3	±1.5
8	Spacing from flange hole center to the member center	a_4	±3
9	Spacing from flange hole center to the web outer-edge	a_5	±3
10	Spacing between web transverse hole within the same group	s_1	±1.5
11	Spacing between web longitudinal hole within the same group	s_2	±1.5
12	Center-to-center distance of bolt group on both ends	s_3	±3
13	Length of member	L	≤9m，±3；>9m，±4
14	Flexibility	c	≤L/500
15	Minimum thickness	t	In accordance with the current national standard of adopted steel strip
16	Schematic diagram	 	

13.3.5 The deviation for profiled metal plate shall not be greater than that specified in Table 13.3.5.

Table 13.3.5 Permissible Deviation for Profiled Metal Plate

Item			Permissible deviation(mm)
Wave spacing			±2.0
Wave height	Profiled plate	$h \leq 70$	±1.5
		$h > 70$	±2
Coverage width	Corrugated profiled plate	$h \leq 70$	−3, +9
		$h > 70$	−2, +6
	Crimping whipstitch profiled plate	$h \leq 70$	−2, +6
		$h > 70$	−3, +9
Plate length			−3, +6
Transverse shearing deviation of plate			5
Transverse cutting deviation of plate end			10
Included angle between bending surface	Included angle of edge bending surfaces		±2°
	Included angle of other bending surface		±3°
Side bending of sideline and plate rib			$\leq L/500$
Unflatness of plate level zone and free edge (within length range of 0.1m to plate edge center line)			2
Minimum thickness			In accordance with the current national standard of adopted materials

Note: L refers to length of plate; h refers to section height of plate (mm).

13.3.6 The geometric dimension deviation of metal flashing and siding parts shall not be greater than that specified in Table 13.3.6.

Table 13.3.6 Permissible Deviation for Machining of Metal Flashing and Siding Parts

Inspection item	Permissible deviation(mm)
Length	±6
Transverse shearing deviation	5
Sectional dimension	±3
Angle	±3°
Minimum thickness	In accordance with the current national standard of adopted materials

13.4 Welds for Members

13.4.1 Various connecting welds for steel member shall be welded by selecting corresponding welding procedure according to weld quality grade and the requirements of product processing drawing. During the product processing, spacing between joint plate welds on the same cross-section should not be less than 200mm.

13.4.2 Welding environment shall meet the relevant requirements of the current national standard GB 50661 *Code for Welding of Steel Structures*.

13.4.3 Non-destructive testing for welds shall be conducted according to the requirements of current national standard GB/T 11345 *Non-destructive Testing of Welds-Ultrasonic Testing-Techniques, Testing Levels, and Assessment* and JG/T 203 *Method for Ultrasonic Testing and*

Classification for Steel Structures. The welds quality level and testing proportion shall be in accordance with those specified in Table 13. 4. 3.

Table 13. 4. 3 Weld Quality Level

Weld quality level		Leve Ⅰ	Level Ⅱ	Level Ⅲ
Internal defect Ultrasonic testing	Evaluation level	Ⅱ	Ⅲ	—
	Inspection level	Level B	Level B	—
	Testing proportion	100%	20%	—

Note: counting method for testing proportion: as for the similar welds, plant-made welds are calculated according to each weld; site-installed welds are calculated according to accumulative length of weld at each joint; where flaw testing length is no less than 200mm, there shall be at least one weld.

13. 4. 4 As for welds rejected by testing inspection, not only rejected positions shall be subjected to repair, but also be doubled for re-inspection. If they fail to pass the re-inspection, 100% testing inspection shall be conducted for these welds.

14 Transportation, Erection and Acceptance

14.1 General Requirements

14.1.1 Transportation and erection of steel structure shall be conducted according to construction organization design while corresponding procedure must guarantee structural stability and against permanent deformation.

14.1.2 Before erection of steel member, detailed inspection shall be conducted on overall dimension, position and diameter of bolt holes, position of connecting parts, welds, friction surface treatment as well as anti-rust coating; in case of any deformation and defect, the member shall be corrected and repaired on the ground and may be erected only after it is deemed as acceptable.

14.1.3 During erection of steel structure, construction of such procedures as onsite holing, welding, assembling as well as coating shall be in accordance with those specified in the current national standard GB 50205 *Code for Acceptance of Construction Quality of Steel Structures*.

14.1.4 As for damaged coating layer of steel member during transportation, storage and lifting, primer shall be subjected to complementary coating before finish.

14.1.5 Before the member lifting, foreign bodies such as oil stain, ice/snow, silt and dust on the surface shall be removed.

14.2 Erection and Correction

14.2.1 Prior to erection of steel structure, locating axis, basement axis and elevation as well as position of foundation bolt shall be inspected, basement shall be re-measured and handover acceptance with basement construction side shall be conducted.

14.2.2 Foundation bolt of rigid frame column base shall be subjected to positioning with reliable methods; plane dimension of buildings shall not only include the length of right angle side, but also the diagonal length. Prior to erection of steel structure, spatial position of foundation bolt shall be corrected to guarantee the plane dimension and elevation of basement top surface meets the design requirements.

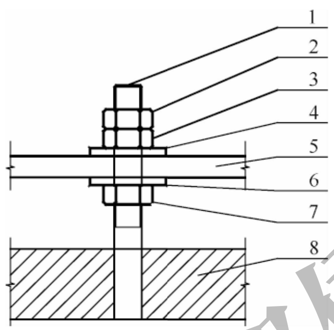
14.2.3 Where the basement top surface is directly used as the bearing surface of column and the embedded steel sheet on the basement top surface or support is used as the bearing surface of column, the permissible deviation of bearing surface and foundation bolt(anchor bolt) shall not be greater than those specified in Table 14.2.3.

Table 14.2.3 Permissible Deviation of Bearing Surface and Foundation Bolt

Item		Permissible deviation(mm)
Bearing surface	Elevation	±3.0
	Planeness	$L/1000$
Foundation bolt	Bolt center deviation	5.0
	Exposed length of the bolt	+20.0 0
	Length of thread	+20.0 0
Center deviation of reserved hole		10.0

Note: L refers to the maximum plane dimension of column base plate.

14.2.4 As for reserved space for column basement during secondary pouring, it should not be greater than 50mm during hinged connection for column base and not greater than 100mm during rigid connection for column base. During erection of column base, high precision control for column mark may be conducted by adding adjusting nut to anchor bolt under bottom plate (Figure 14.2.4).



1—foundation bolt; 2—retaining nut; 3—clamp nut; 4—nut subplate; 5—bottom plate of steel column; 6—subplate of bottom nut ; 7—adjusting nut; 8—reinforced concrete basement

Figure 14.2.4 Erection of Column Base

14.2.5 During erection of steel structure of light-weight building with gabled frames, measures shall be taken to guarantee structural stability according to requirements of design and construction conditions.

14.2.6 Erection of main member shall meet the following requirements:

1 Erection should start from rigid frames on both ends close to gable with column brace. After completion of the erection of rigid frames, purlin, brace and diagonal brace between them shall be all erected, and their verticality shall also be inspected. With these two rigid frames as starting point, they shall be erected in toward the other end of building.

2 Column should be firstly erected during the erection of rigid frame: assembled inclined beam on ground shall be subjected to lifting and positioning, and connected with column.

3 After space rigid unit is formed and corrected during erection of steel structure, gaps in the column bottom plate and top surface of basement shall be timely subjected to secondary pouring with fine stone concrete.

4 As for member with large span and little lateral rigidity, gravity center shall be determined before erection, reasonable lifting point position and lifting device shall be selected and stability check before lifting shall be conducted for significant member and elongated member, temporary reinforcement shall be conducted according to checked result and necessary measures such as haul/pull, brace as well as temporary connection should be taken in the process of member erection.

5 During erection, high-altitude operation shall be reduced. If crane capability is allowed, expanded erection unit shall be assembled on ground; necessary fixation should be conducted for heavily-loaded positions e.g. such auxiliary means as iron shoulder pole and pulley block may be increased; blind and risked lifting shall be avoided.

6 Erection check shall be conducted for lifting point of large membersso that the internal force generated by each part is less than the bearing capacity of the member to avoid permanent deformation.

14.2.7 Correction for erection of steel structure shall meet the following requirements:

1 As for measurement and correction for erection of steel structure, measuring process and correction scheme shall be prepared in advance according to engineering characteristics.

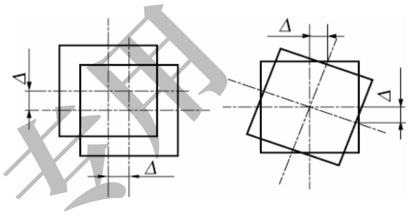
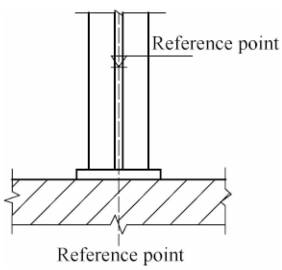
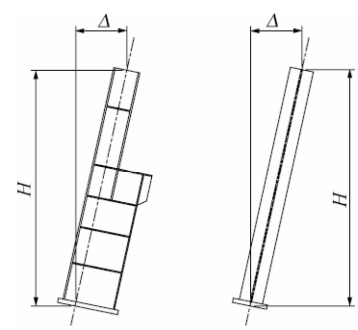
2 Such primary members as rigid frame column, beam and brace shall be subjected to correction immediately after erection. And then permanent fixation shall be conducted immediately.

14.2.8 Where there is reliable basis, already-erected steel structure may be utilized for lifting of other member and equipment. Appropriate assurance measures shall be taken prior to operation.

14.2.9 As for joint required to be tightly pushed in design, 70% surface shall be closely against each other on contact surface, which is inspected with 0.3mm thick feeler gauge, sum of inserted area shall not be greater than 30% total area of tightly-pushed joint, and edge maximum clearance shall not be greater than 0.8mm.

14.2.10 The deviation for erection of rigid frame column shall not be greater than that specified in Table 14.2.10.

Table 14.2.10 Permissible Deviation for Erection of Rigid Frame Column

No.	Item			Permissible deviation (mm)	Legend
1	Displacement of column base center line to positioning axis (Δ)			5.0	
2	Reference point elevation of column	Column with crane beam		+3.0 −5.0	
3		Column without crane beam		+5.0 −8.0	
4	Deflection rise			H/1000 10.0	
5	Column axis verticality (Δ)	Single-layer column	H≤12m	10.0	
6			H>12m	H/1000 20.0	
7		Multi-layer column	Bottom-layer column	10.0	
8			Full height of column	20.0	
9	Column top elevation (Δ)			≤±10.0	

14.2.11 The deviation for erection of rafter of rigid frame shall not be greater than that specified in Table 14.2.11.

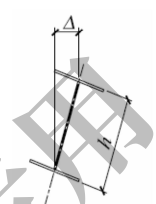
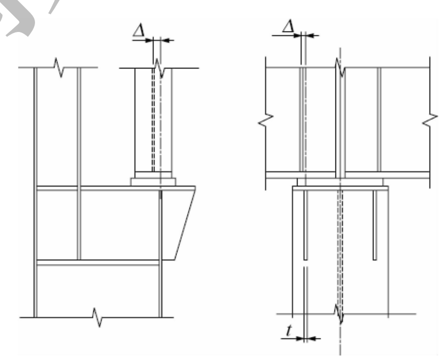
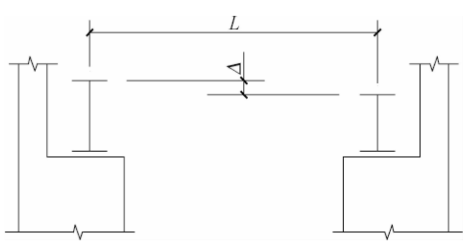
Table 14.2.11 Permissible Deviation for Erection of rafter of Rigid Frame

Item		Permissible deviation(mm)
Mid-span verticality of beam		$H/500$
Warpage of beam	Lateral direction	$L/1000$
	Vertical direction	+10.0, −5.0
Joint position of two adjacent beams	Center misalignment	3.0
	Height difference between top surfaces	2.0
Height difference between top surface of two adjacent beams	Bearing position	1.0
	Other positions	$L/500$

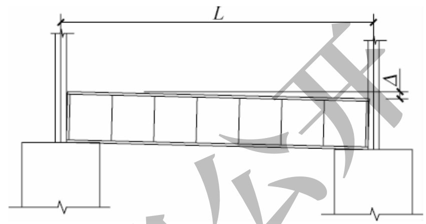
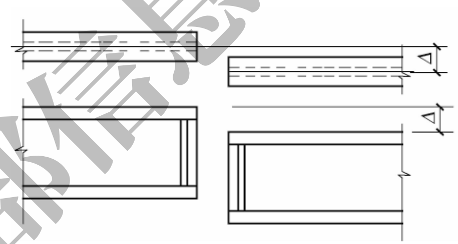
Note: H refers to section height in beam span; L refers to the maximum span of adjacent beams.

14.2.12 The deviation for erection of crane beam shall not be greater than that specified in Table 14.2.12.

Table 14.2.12 Permissible Deviation for Erection of Crane Beam

No.	Item	Permissible deviation (mm)	Legend
1	Mid-span verticality of beam(Δ)	$h/500$	
2	Lateral bending rise	$L/1500$ 10.0	
3	Vertical up-arch rise	10.0	
4	Center distance of supports on both ends(Δ); Displacement to bracket center if installed on steel column	5.0	
5	Center-to-center displacement between stiffening plate of crane beam support and the pressure-bearing stiffening plate of column(Δ)	$t/2$	
6	Top surface elevation difference of the crane beam in the same cross section within the same span(Δ): At the support Other positions	 10.0 15.0	
7	Central span of crane beam in any cross section within the same span(L)	±10.0	

continned table 14.2.10

No.	Item	Permissible deviation (mm)	Legend
8	Top surface elevation difference of crane beam between two adjacent columns in the same column(Δ)	L/1500 10.0	
9	Joint part misalignment of two adjacent crane beams(Δ): Center misalignment Height difference between top surface	 2.0 1.0	

14.2.13 After erection and adjustment of main steel structure, such tensile supporting members as column brace and roof brace shall be tensioned.

14.3 High-strength Bolts

14.3.1 High-strength bolt connector entering into site shall be subjected to re-inspection and re-inspection data shall be in accordance with relevant requirements of the current national standard GB 50205 *Code for Acceptance of Construction Quality of Steel Structures*. Re-inspection data for torque coefficient of high-strength large hexagon head bolt connector shall not only meet the relevant requirements, but also be regarded as parameter for screwing.

14.3.2 As for friction-type connection of high-strength bolt, antiskid coefficient of friction surface shall be tested according to those specified in current national standard GB 50205 *Code for Acceptance of Construction Quality of Steel Structures* and design document.

14.3.3 The quantity of temporary bolts used during erection is to such degree that shall be able to bear member deadweight and exogenic action during connection correction, at least 2 for each. The high-strength bolts for connection shall not be doubled as temporary bolt.

14.3.4 During erection of high-strength bolts, access hole must not be knocked in forcible way; appropriate reamer and special tool may be adopted for reaming hole; maximum diameter of hole after correction shall be less than 1.2 times of the bolt diameter; gas cutting shall not be adopted for reaming.

14.3.5 Contact surface of steel sheet in high-strength bolt connection shall be flat and smooth and treatment may be omitted where the clearance is less than 1.0mm; 1.0mm～3.0mm, raised one side shall be worn to inclined plane with slope of 1:10 with direction for wearing vertical to that of force; greater than 3.0mm, subplate shall be added with treatment method for both sides the same as that for friction surface of connecting plates.

14.3.6 Screwing of high-strength bolt connector shall be divided into primary screwing, secondary screwing and final screwing from joint center position of bolt group to outward edge in sequence, which shall be completed within 24h.

14.3.7 Acceptance for construction torque of high-strength large hexagon head bolt: a straight line may be marked on the side of screw rod and nut, then unscrew nut approximately 60° and screw up with torque wrench to make end line lapped, and it is deemed acceptable if measured torque is within the range of 10% of that prior to construction.

14.3.8 Quantity of bolt connectors for random inspection on torque of each joint shall be 10% and shall not be less than 1. If connectors fail to pass sampling inspection, another 10% shall be re-sampled for inspection; if they are still rejected, less-tightened connector and connector with missing tightening shall be subjected to supplementary tightening while super-tightened connector shall be replaced with new bolt. Torque inspection shall be completed after 1h of construction and within 24h.

14.4 Welding and Other Fasteners

14.4.1 The fixed-position welding of erection shall meet the following requirements:

1 Site welds shall be operated by the welder with weld certificate, and those without certificate must not conduct welding;

2 The model of adopted welding materials shall be matched with the material of weldment;

3 The weld thickness shall not exceed 2/3 designed weld height or 8mm;

4 The weld length should not be less than 25mm.

14.4.2 Ordinary bolt connection shall meet the following requirements:

1 Each end of each bolt shall not be provided with more than two washers or large nut is used to replace the washer;

2 Tail-exposed thread shall not be less than two thread pitches after bolt is screwed;

3 Gas cutting ream is prohibited for any bolt hole.

14.4.3 Where member is connected in combined means of welding and high-strength bolt, bolting shall be conducted before welding during construction.

14.4.4 Self-drilling tapping screws, rivet and drive pin shall firmly and closely stick to the connecting steel sheet in an order arrangement. Specification and dimension shall match with connecting steel sheet, and the spacing and edge distance shall meet the design requirements.

14.4.5 Drive pin, rivet and foundation bolt shall be subjected to quality acceptance according to relevant technical documents of manufacturer and the design requirements.

14.5 Erection of Purlins and Girts

14.5.1 According to division of erection unit, erection for such secondary members as purlins and girts shall be conducted immediately after the primary member finishes erection..

14.5.2 Bolts of purlins, girts and eaves purlins between rigid frames shall be screwed after calibration excluding two initially-erected rigid frames.

14.5.3 During erection of purlins and girts, stay bar or tensioned rod shall be arranged and tensioned but shall not bend the purlin and girt.

14.5.4 During lifting of cold-formed thin-wall steel member such as purlins and girts, adequate measures shall be taken to avoid permanent deformation, and contact position between cable loop and member shall be well cushioned.

14.5.5 Erected and positioned purlin and girt members shall not be used for lifting other heavy objects.

14.6 Erection of Enclosure System

14.6.1 During erection of wallboard and roofing, girt and purlin shall be kept straightly and flatly.

14.6.2 Heat preservation material shall be laid flatly and smoothly with both ends fixed to the structure main body; where single-side vapor barrier, if adopted, shall be placed at the inner side of building. Longitudinal and transverse lapping section of vapor barrier shall be bonded or screwed. Felt material at end shall be reflexed and closed with vapor barrier. If material of vapor barrier fails to bear deadweight of heat preservation material, brace mesh shall be laid under the barrier.

14.6.3 Where fixed roofing is connected with purlin and wallboard is connected with girt, center distance between screws should not be greater than 300mm. If end of building is connected with end socket of roofing, spacing between screws should be reduced. Spacing between screws at side lapping of roofing may be properly enlarged while that of wallboard may be further increased based on former.

14.6.4 Continuous sealing rubber strip shall be arranged at lapped joints along longitudinal and transverse direction of roofing. As for lapped edge at cornice, the plug with the same section shape as that of roofing shall also be arranged apart from rubber strip.

14.6.5 The flashing plate or wrap-edge-plate with favorable sealing performance and appearance shall be arranged around angle section, roof ridge, cornice, roofing opening or protrusion.

14.6.6 During erection of profiled steel roof, effective measures shall be taken to distribute construction load to relatively large area to prevent roofing from local buckling due to construction concentrated load.

14.6.7 During construction on roof, safety measures such as safety rope shall be adopted, and safety net shall also be adopted where necessary.

14.6.8 During laying of profiled steel sheet, attention shall be paid to perennial wind direction which shall be opposite to that in the lapping of plate rib.

14.6.9 The flatness of plate on both sides shall be inspected for every erection of 5～6 profiled steel sheets, and deviation, if any, shall be timely regulated.

14.6.10 The deviation for erection of profiled steel sheet shall not be greater than permissible deviation specified in Table 14.6.10.

Table 14.6.10 Permissible Deviation for Erection of Profiled Steel sheet

Item	Permissible deviation (mm)
Misalignment between adjacent columns of profiled steel sheet on the beam	10.0
End misalignment of two adjacent profiled steel sheets at the cornice	5.0
Verticality of the wavy line of profiled steel sheet to ridge	$L/1000$
Verticality of cladding's wavy line	$H/1000$
Verticality of wrap-angle-plate on wall surface	$H/1000$
Lower-end misalignment of two adjacent profiled steel sheets on wall surface	5.0

Note: H refers to the building height and L refers to the length of profiled steel sheet.

14.7 Acceptance

14.7.1 According to the requirements of current national standard GB 50300 *Unified Standard for Constructional Quality Acceptance of Building Engineering*, completion acceptance for steel structure shall be conducted based on subsection project while that for large steel structure engineering may be conducted based on several divided sub-subsection projects.

14.7.2 Acceptable quality level for subsection project shall meet the following requirements:

1 The quality of each subsection project shall meet the acceptable quality level;

2 Quality control information and document shall be complete;

3 Each item of inspection shall meet the requirements of current national standard GB 50205 *Code for Acceptance of Construction Quality of Steel Structures*.

14.7.3 The following documents and records shall be provided during completion acceptance for subsection project:

1 The completion drawing and relevant design documents of steel structure engineering;

2 Records of quality management inspection on construction site;

3 Inspection records of relevant safety and function inspection and witness inspection items;

4 Inspection records of relevant appearance quality inspection items;

5 Quality acceptance record of sub-item project contained in subsection project;

6 Quality acceptance record of each inspection lot contained in sub-item project;

7 Inspection records and certification documents of inspection items required by compulsory provision;

8 Inspection and acceptance records of inspection items for concealed engineering;

9 Quality compliance certification documents, Chinese mark, performance test report of raw material and finished product;

10 Treatment records and acceptance records of rejected item;

11 Implementation scheme and acceptance record of significant quality and technical problems;

12 Other relevant documents and records.

14.7.4 Quality acceptance record of steel structure engineering shall meet the following requirements:

1 Records of quality management inspection on construction site shall be in accordance with those specified in current national standard GB 50300 *Unified Standard for Constructional Quality Acceptance of Building Engineering*;

2 Acceptance record of sub-item project shall be in accordance with those specified in current national standard GB 50300 *Unified Standard for Constructional Quality Acceptance of Building Engineering*;

3 Acceptance record of acceptance lot in sub-item project shall be in accordance with those specified in current national standard GB 50205 *Code for Acceptance of Construction Quality of Steel Structures*;

4 The acceptance record of sub-section and sub-subsection project shall be in accordance with those specified in current national standard GB 50300 *Unified Standard for Constructional Quality Acceptance of Building Engineering*.

Appendix A　Calculated Length of Frame Column

A.0.1　The critical load of lateral elastic buckling of and the calculated length coefficient of tapered gabled frame column may be calculated according to the following formulas:

$$N_{cr} = \frac{\pi^2 E I_1}{(\mu H)^2} \tag{A.0.1-1}$$

$$\mu = 2\left(\frac{I_1}{I_0}\right)^{0.145}\sqrt{1+\frac{0.38}{K}} \tag{A.0.1-2}$$

$$K = \frac{K_z}{6 i_{c1}}\left(\frac{I_1}{I_0}\right)^{0.29} \tag{A.0.1-3}$$

Where　μ ——the calculated length coefficient where tapered column is converted to constant section column subject to the section on large end;

I_0——the inertia moment of the section on small end of post (mm^4);

I_1——the inertia moment of the section on large end of post (mm^4);

H ——the height of tapered column (mm);

K_z——the rotation restraint of beam to column (N·mm);

i_{c1}—— the linear rigidity of column (N·mm), $i_{c1} = EI_1/H$.

A.0.2　The determination for rotation restraint of frame beam to frame column shall meet the following requirements:

1　In case of rigid connection of beam's both ends and column, assume that deformation form of beam make inflection point at mid-span, take out semi-span beam, far-end hinged brace, calculate near-end rotation angle (θ) via imposing the bending moment (M) at near end and rotation restraint shall be calculated according to the following formula:

$$K_z = \frac{M}{\theta} \tag{A.0.2}$$

2　Either the far-end of frame beam is simple-supported or is leaning stanchion, s stated in A.0.3 of this Code shall be the length of full-span beam;

3　Where near-end of frame beam is simply-supported to column, rotation restraint shall be 0.

A.0.3　The rotation restraint of tapered beam to frame column shall, according to condition of the variable section of frame beam, be respectively calculated according to the following formulas:

1　One-segment variable section for frame beam (Figure A.0.3-1):

$$K_z = 3 i_1\left(\frac{I_0}{I_1}\right)^{0.2} \tag{A.0.3-1}$$

$$i_1 = \frac{EI_1}{s} \tag{A.0.3-2}$$

Where　I_0——the inertia moment of the section on small end at mid-span of tapered beam (mm^4);

I_1——the inertia moment of the section on large end at cornice of tapered beam (mm^4);

s——the inclined length of tapered beam(mm).

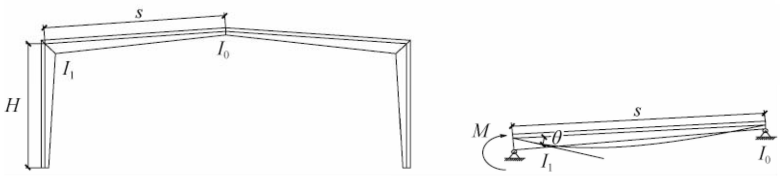


Figure A. 0. 3-1 One-segment Variable Section for Frame Beam and its Rotational Rigidity

2 Two-segment variable sections for frame beam (Figure A. 0. 3-2):

$$\frac{1}{K_z}=\frac{1}{K_{11,1}}+\frac{2s_2}{s}\frac{1}{K_{12,1}}+\left(\frac{s_2}{s}\right)^2\frac{1}{K_{22,1}}+\left(\frac{s_2}{s}\right)^2\frac{1}{K_{22,2}} \quad \text{(A. 0. 3-3)}$$

$$K_{11,1}=3i_{11}R_1^{0.2} \quad \text{(A. 0. 3-4)}$$

$$K_{12,1}=6i_{11}R_1^{0.44} \quad \text{(A. 0. 3-5)}$$

$$K_{22,1}=3i_{11}R_1^{0.712} \quad \text{(A. 0. 3-6)}$$

$$K_{22,2}=3i_{21}R_2^{0.712} \quad \text{(A. 0. 3-7)}$$

$$R_1=\frac{I_{10}}{I_{11}} \quad \text{(A. 0. 3-8)}$$

$$R_2=\frac{I_{20}}{I_{21}} \quad \text{(A. 0. 3-9)}$$

$$i_{11}=\frac{EI_{11}}{s_1} \quad \text{(A. 0. 3-10)}$$

$$i_{21}=\frac{EI_{21}}{s_2} \quad \text{(A. 0. 3-11)}$$

$$s=s_1+s_2 \quad \text{(A. 0. 3-12)}$$

Where R_1——the ratio of far-end to near-end section inertia moment in the first segment of tapered beam connected with post;

R_2——the ratio of far-end to near-end section inertia moment in the second segment of tapered beam;

s_1——the inclined length of the first segment of tapered beam connected with post(mm);

s_2——the inclined length of the second segment of tapered beam(mm);

s——the inclined length of tapered beam(mm);

i_{11}——the linear rigidity calculated with inertia moment of the section on large end (N·mm);

i_{21}——the linear rigidity calculated with the second segment of far-end section inertia moment(N·mm);

I_{10}, I_{11}, I_{20}, I_{21}——the inertia moments of tapered beam(mm^4) (Figure A. 0. 3-2).

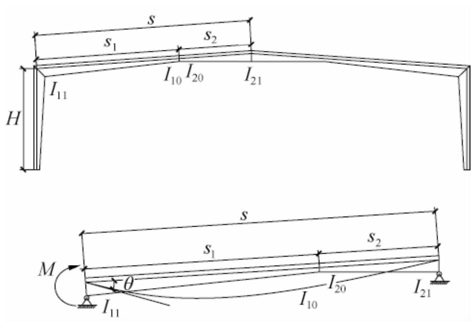


Figure A. 0. 3-2 Two-segment Variable Sections for Frame Beam and its Rotational Rigidity

3 Three-segment variable sections for frame beam (Figure A.0.3-3):

$$\frac{1}{K_z}=\frac{1}{K_{11,1}}+2\left(1-\frac{s_1}{s}\right)\frac{1}{K_{12,1}}+\left(1-\frac{s_1}{s}\right)^2\left(\frac{1}{K_{22,1}}+\frac{1}{3i_2}\right)$$
$$+\frac{2s_3(s_2+s_3)}{s^2}\frac{1}{6i_2}+\left(\frac{s_3}{s}\right)^2\left(\frac{1}{3i_2}+\frac{1}{K_{22,3}}\right) \quad \text{(A.0.3-13)}$$

$$K_{11,1}=3i_{11}R_1^{0.2} \quad \text{(A.0.3-14)}$$

$$K_{12,1}=6i_{11}R_1^{0.44} \quad \text{(A.0.3-15)}$$

$$K_{22,1}=3i_{11}R_1^{0.712} \quad \text{(A.0.3-16)}$$

$$K_{22,3}=3i_{31}R_3^{0.712} \quad \text{(A.0.3-17)}$$

$$R_1=\frac{I_{10}}{I_{11}},R_3=\frac{I_{30}}{I_{31}} \quad \text{(A.0.3-18)}$$

$$i_{11}=\frac{EI_{11}}{s_1},i_2=\frac{EI_2}{s_2},i_{31}=\frac{EI_{31}}{s_3} \quad \text{(A.0.3-19)}$$

Where I_{10}, I_{11}, I_2, I_{30}, I_{31}——the inertia moments of tapered beam (mm^4) (Figure A.0.3-3).

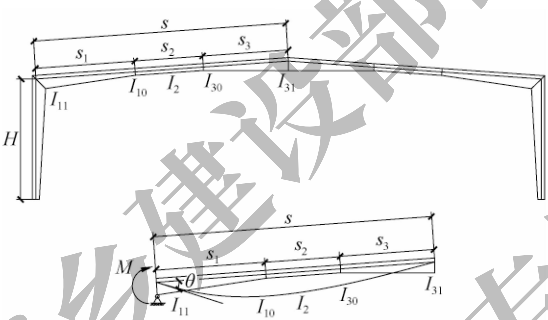


Figure A.0.3-3 Three-segment Variable Sections for Frame Beam and its Rotational Rigidity

A.0.4 Where the stepped column or two segments of column adopted, calculated length of the lower column and the upper column shall be determined according to the following formulas:

Calculated length coefficient of the lower column

$$\mu_1=\sqrt{\gamma}\cdot\mu_2 \quad \text{(A.0.4-1)}$$

Calculated length coefficient of the upper column

$$\mu_2=\sqrt{\frac{6K_1K_2+4(K_1+K_2)+1.52}{6K_1K_2+K_1+K_2}} \quad \text{(A.0.4-2)}$$

$$K_2=\frac{K_{z2}}{6i_{c2}} \quad \text{(A.0.4-3)}$$

$$K_1=\frac{K_{z1}}{6i_{c2}}+\frac{b+\sqrt{b^2-4ac}}{12a} \quad \text{(A.0.4-4)}$$

$$a=(a_1b_1\gamma-a_2b_2)i_{c2}^2 \quad \text{(A.0.4-5)}$$

$$b=(K_{z0}i_{c1}\gamma b_1-\gamma c_2a_1-i_{c1}a_3b_2+c_1a_2)i_{c1} \quad \text{(A.0.4-6)}$$

$$c=i_{c1}(c_1a_3-K_{z0}c_2\gamma) \quad \text{(A.0.4-7)}$$

$$a_1=K_{z0}+i_{c1} \quad \text{(A.0.4-8)}$$

$$a_2=K_{z0}+4i_{c1} \quad \text{(A.0.4-9)}$$

$$a_3=4K_{z0}+9.12i_{c1} \quad \text{(A.0.4-10)}$$

$$b_1=K_{z2}+4i_{c2} \quad \text{(A.0.4-11)}$$

$$b_2=K_{z2}+i_{c2} \quad \text{(A.0.4-12)}$$

$$c_1=K_{z1}K_{z2}+(K_{z1}+K_{z2})i_{c2} \quad \text{(A.0.4-13)}$$

$$c_2 = K_{z1}K_{z2} + 4(K_{z1} + K_{z2})i_{c2} + 9.12i_{c2}^2 \tag{A.0.4-14}$$

$$\gamma = \frac{N_2 H_2}{N_1 H_1}\frac{i_{c1}}{i_{c2}} \tag{A.0.4-15}$$

$$i_{c1} = \frac{EI_{11}}{H_1}\left(\frac{I_{10}}{I_{11}}\right)^{0.29} \tag{A.0.4-16}$$

$$i_{c2} = \frac{EI_2}{H_2} \tag{A.0.4-17}$$

Where K_{z0}——the rotation restraint provided by column to column base(N • mm); $K_{z0} = 0.5i_{c1}$ where column base is hinged; $K_{z0} = 50i_{c1}$ where column base is fixed;

K_{z1}——the rotation restraint provided by intermediate beam (low-span roof beam and mezzanine beam) to column (N • mm), determined according to those specified in A. 0. 3 of this Code;

K_{z2}——the rotation restraint of roof beam to upper column top (N • mm), determined according to those specified in A. 0. 3 of this Code;

i_{c1}——the linear rigidity of lower column (N • mm) where it is the variable section;

i_{c2}——the linear rigidity of the upper column (N • mm);

I_1, I_2, I_{10}, I_{11}——the inertia moment of column (mm^4) (Figure A. 0. 4);

N_1, N_2——the respective axial forces of the upper and the lower columns(N);

H_1, H_2——the respective heights of the upper and the lower columns(mm).

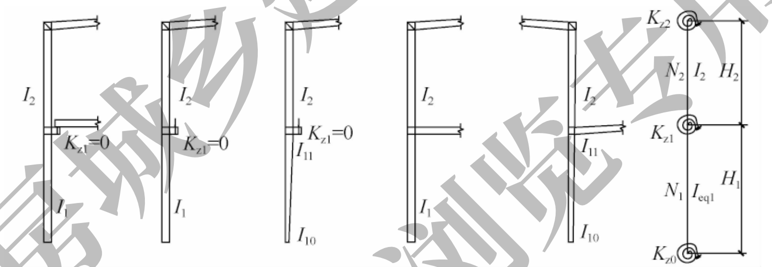


Figure A. 0. 4 Calculation Model for Tapered and Stepped Frame Column

A. 0. 5 Where the two-step column or three segments of column are adopted, calculated length of the lower, the intermediate and the upper column shall be determined according to different calculation models or shall be calculated according to the following formulas:

$$\mu_2 = \sqrt{\frac{6K_1K_2 + 4(K_1 + K_2) + 1.52}{6K_1K_2 + K_1 + K_2}} \tag{A.0.5-1}$$

$$\mu_1 = \sqrt{\gamma_1} \cdot \mu_2 \tag{A.0.5-2}$$

$$\mu_3 = \sqrt{\gamma_3} \cdot \mu_2 \tag{A.0.5-3}$$

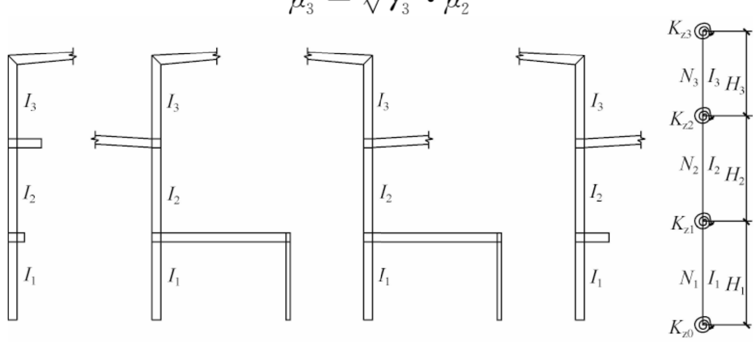


Figure A. 0. 5 Calculation Model for Three-step Frame Column

Middle-section column:

$$K_1 = K_{b1} - \frac{\eta}{6}, K_2 = K_{b2} - \frac{\xi}{6} \tag{A.0.5-4}$$

ξ and η are determined by one of these three groups of solutions given by the following formulas, besides, K_1 and K_2 meeting the requirements of Formula (A.0.5-7, A.0.5-8 and A.0.5-9) is the only efficient solution.

$$\eta_j = 2\sqrt[3]{r}\cos\left[\frac{\theta + 2(j-2)\pi}{3}\right] - \frac{b}{3a} \quad (j = 1, 2, 3) \tag{A.0.5-5}$$

$$\xi_j = \frac{6(e_3\eta + e_4)}{e_1\eta + e^2} \qquad (j = 1, 2, 3) \tag{A.0.5-6}$$

$$K_1 > -\frac{1}{6} \tag{A.0.5-7}$$

$$K_2 > -\frac{1}{6} \tag{A.0.5-8}$$

$$6K_1K_2 + K_1 + K_2 > 0 \tag{A.0.5-9}$$

Where:

$r = \sqrt{\frac{m^3}{27}}$; $\theta = \arccos\frac{-n}{\sqrt{-4m^3/27}}$; $\Delta = \frac{n^2}{4} + \frac{m^3}{27}$; $m = \frac{3ac - b^2}{3a^2}$; $n = \frac{2b^3 - 9abc + 27a^2d}{27a^3}$; $a = \gamma_1 a_2 g_4 - a_1 g_1$; $b = \gamma_1 a_2 g_5 + 6\gamma_1 K_{b0} K_{c1} g_4 - a_1 g_2 - 6K_{c1} a_3 g_1$; $c = \gamma_1 a_2 g_6 + 6\gamma_1 K_{b0} K_{c1} g_5 - a_1 g_3 - 6K_{c1} a_3 g_2$; $d = 6K_{c1}(\gamma_1 K_{b0} g_6 - a_3 g_3)$; $e_1 = a_2 b_1 \gamma_1 - a_1 b_2 \gamma_3$; $e_2 = 6K_{c1}(K_{b0}\gamma_1 b_1 - a_3 b_2 \gamma_3)$; $e_3 = K_{c3}(\gamma_3 K_{b3} a_1 - b_3 a_2 \gamma_1)$; $e_4 = 6K_{c1}K_{c3}(\gamma_3 K_{b3} a_3 - \gamma_1 K_{b0} b_3)$; $a_1 = 6K_{b0} + 4K_{c1}$; $a_2 = 6K_{b0} + K_{c1}$; $a_3 = 4K_{b0} + 1.52K_{c1}$; $b_1 = 6K_{b3} + 4K_{c3}$; $b_2 = 6K_{b3} + K_{c3}$; $b_3 = 4K_{b3} + 1.52K_{c3}$; $c_1 = 6K_{b1} + 4$; $c_2 = 6K_{b1} + 1$; $d_1 = 6K_{b2} + 4$; $d_2 = 6K_{b2} + 1$; $f_1 = 6K_{b1}K_{b2} + K_{b2} + K_{b1}$; $f_2 = 6K_{b1}K_{b2} + 4(K_{b2} + K_{b1}) + 1.52$; $g_1 = e_3 - \frac{1}{6}d_2 e_1$; $g_2 = f_1 e_1 - c_2 e_3 - \frac{1}{6}d_2 e_2 + e_4$; $g_3 = f_1 e_2 - c_2 e_4$; $g_4 = e_3 - \frac{1}{6}d_1 e_1$; $g_5 = f_2 e_1 - c_1 e_3 - \frac{1}{6}d_1 e_2 + e_4$; $g_6 = f_2 e_2 - c_1 e_4$; $K_{b0} = \frac{K_{z0}}{6i_{c2}}$; $K_{b1} = \frac{K_{z1}}{6i_{c2}}$; $K_{b2} = \frac{K_{z2}}{6i_{c2}}$; $K_{b3} = \frac{K_{z3}}{6i_{c2}}$; $K_{c1} = \frac{i_{c1}}{i_{c2}}$; $K_{c3} = \frac{i_{c3}}{i_{c2}}$; $\gamma_1 = \frac{N_2 H_2}{N_1 H_1}\frac{i_{c1}}{i_{c2}}$; $\gamma_3 = \frac{N_2 H_2}{N_3 H_3}\frac{i_{c3}}{i_{c2}}$; $i_{c1} = \frac{EI_1}{H_1}$; $i_{c2} = \frac{EI_2}{H_2}$; $i_{c3} = \frac{EI_3}{H_3}$;

Where μ_1, μ_2, μ_3 —— the respective calculated length coefficients of lower section, middle section and upper section columns;

i_{c1}, i_{c2}, i_{c3} —— the respective linear rigidities of the upper section, middle section and lower section columns (N·mm).

A.0.6 During determination of rotation restraint of beam to frame column with leaning stanchion (Figure A.0.6), determined calculated length coefficient shall multiply amplification coefficient η under assumption of far-end hinge brace on top of leaning stanchion.

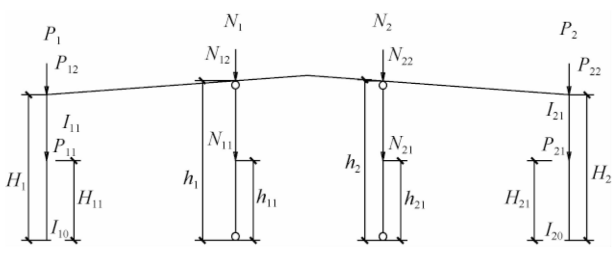


Figure A.0.6 Frame Provided with Leaning Stanchion

1 Amplification coefficient η shall be calculated according to the following formulas:

$$\eta=\sqrt{1+\frac{\sum N_j/h_j}{1.1\sum P_i/h_i}} \tag{A.0.6-1}$$

$$N_j=\frac{1}{h_j}\sum_k N_{jk}h_{jk} \tag{A.0.6-2}$$

$$P_i=\frac{1}{H_i}\sum_k P_{ik}H_{ik} \tag{A.0.6-3}$$

Where N_j —— the axial pressure of leaning stanchion converted to column top (N);

N_{jk}, h_{jk} —— the k_{th} vertical load on j_{th} leaning stanchion (N) and its acting height (mm);

P_i —— the axial pressure of frame column converted to column top (N);

P_{ik}, H_{ik} —— the k_{th} vertical load on i_{th} leaning stanchion (N) and its acting height (mm);

h_j —— the height of j_{th} leaning stanchion (mm);

H_i —— the height of i_{th} leaning stanchion (mm).

2 Where there is no vertical load in the middle of column of leaning stanchion, the calculated length coefficient of stanchion takes 1.0;

3 Where there is vertical load in the middle of column of leaning stanchion, calculated length coefficient of each column section may be determined with consideration of interaction of upper/lower column section.

A.0.7 Where second-step analysis is adopted, the calculated length of column shall meet the following requirements:

1 Calculated length coefficient of single-section column with constant section may take 1.0;

2 Where crane plant is provided, calculated length coefficient of each column section for two-step or three-step column shall be determined according to model hinged on column top free from lateral displacement. Where there is mezzanine or high-low span, calculated length coefficient of each column section may take 1.0;

3 Calculated length coefficient for single-section tapered column articulated on column base μ_{r} shall be calculated according to the following formula:

$$\mu_{\mathrm{r}}=\frac{1+0.35\gamma}{1+0.54\gamma}\sqrt{\frac{I_1}{I_0}} \tag{A.0.7-1}$$

$$\gamma=\frac{h_1}{h_0}-1 \tag{A.0.7-2}$$

Where γ —— the tapering ratio of the tapered beam;

h_0, h_1 —— the respective height of the section on small end and large end (mm);

I_0, I_1 —— the respective inertia moment of the section on small end and large end (mm^4).

A.0.8 As for single-layer multi-span building, where elevation of roof beam is free from sudden change from span to span (without high-low span), in-plane stability calculation of frame column may be conducted with corrected calculated length coefficient with consideration of mutual support between each column. Corrected calculated length coefficient shall be calculated according to the following formulas. Where the calculation value is less than 1.0, 1.0 shall be taken.

$$\mu_j'=\frac{\pi}{h_i}\sqrt{\frac{EI_{\mathrm{c}j}\left[1.2\sum(P_i/H_i)+\sum(N_k/h_k)\right]}{P_j\cdot K}} \tag{A.0.8-1}$$

$$\mu_j'=\frac{\pi}{h_i}\sqrt{\frac{EI_{\mathrm{c}j}\left[1.2\sum(P_i/H_i)+\sum(N_k/h_k)\right]}{1.2P_j\sum(P_{\mathrm{cr}j}/H_j)}} \tag{A.0.8-2}$$

Where N_k, h_k——the axial force (N) and height (mm) on leaning stanchion respectively;

K——the lateral rigidity of rigid frame calculated with horizontal acting force at cornice height;

P_{crj}——the critical load of frame column calculated via traditional method and its calculated length coefficient may be calculated according to Formula (A. 0. 1-2).

A. 0. 9 Calculated length coefficient of frame column determined according to this Appendix is applicable to roof slope no larger than 1/5, otherwise adverse effect from axial force of transverse beam shall be considered.

Explanation of Wording in This Code

1 Words used for different degrees of strictness are explained as follows in order to mark the differences in executing the requirements in this Code:

1) Words denoting a very strict or mandatory requirement:

"Must" is used for affirmation; "must not" for negation.

2) Words denoting a strict requirement under normal conditions:

"Shall" is used for affirmation, "shall not" or "should not" for negation.

3) Words denoting a permission of a slight choice or an indication of the most suitable choice when conditions permit:

"Should" is used for affirmation; "preferably not" for negation;

4) "May" is used to express the option available, sometimes with the conditional permit.

2 "Shall comply with..." or "shall meet the requirements of..." is used in the Specification to indicate that it is necessary to comply with the requirements stipulated in other relative standards.

List of Quoted Standards

1 GB 50009 *Load Code for the Design of Building Structures*
2 GB 50010 *Code for Design of Concrete Structures*
3 GB 50011 *Code for Seismic Design of Buildings*
4 GB 50016 *Code for Fire Protection Design of Buildings*
5 GB 50017 *Standard for Design of Steel Structures*
6 GB 50018 *Technical Code of Cold - formed Thin - wall Steel Structures*
7 GB 50046 *Code for Anti - corrosion Design of Industrial Constructions*
8 GB 50205 *Code for Acceptance of Construction Quality of Steel Structures*
9 GB 50223 *Standard for Classification of Seismic Protection of Building Constructions*
10 GB 50300 *Unified Standard for Constructional Quality Acceptance of Building Engineering*
11 GB 50661 *Code for Welding of Steel Structures*
12 GB 50755 *Code for Construction of Steel Structures*
13 GB/T 700 *Carbon Structural Steels*
14 GB/T 985.1 *Recommended Joint Preparation for Gas Welding Manual Metal Arc Welding Gas - Shield Arc Welding and Beam Welding*
15 GB/T 985.2 *Recommended Joint Preparation for Submerged Arc Welding*
16 GB/T 1228 *High Strength Bolts with Large Hexagon Head for Steel Structures*
17 GB/T 1229 *High Strength Large Hexagon Nuts for Steel Structures*
18 GB/T 1230 *High strength Plain Washers for Steel Structures*
19 GB/T 1231 *Specifications of High Strength Bolts with Large Hexagon Head, Large Hexagon Nuts, Plain Washers for Steel Structures*
20 GB/T 1591 *High Strength Low Alloy Structural Steels*
21 GB 1720 *Method of Test for Adhesion of Paint Films*
22 GB/T 2518 *Continuously Hot - dip Zinc - coated Steel Sheet and Strip*
23 GB/T 3098.1 *Mechanical Properties of Fasteners - Bolts, Screws and Studs*
24 GB/T 3632 *Sets of Torshear Type High Strength Bolt Hexagon Nut and Plain Washer for Steel Structures*
25 GB/T 5117 *Covered Electrodes for Manual Metal Arc Welding of Non - alloy and Fine Grain Steels*
26 GB/T 5118 *Covered Electrodes for Manual Metal Arc Welding of Creep - resisting Steels*
27 GB/T 5282 *Slotted Pan Head Tapping Screws*
28 GB/T 5283 *Slotted Countersunk Head Tapping Screws*
29 GB/T 5284 *Slotted Raised Countersunk Head Tapping Screws*
30 GB/T 5285 *Hexagon Head Tapping Screws*
31 GB/T 5293 *Carbon Steel Electrodes and Fluxs for Submerged Arc Welding*

32 GB/T 5780 *Hexagon head bolts - Product Grade C*

33 GB/T 5782 *Hexagonal Head Bolt*

34 GB/T 8110 *Welding Electrodes and Rods for Gas Shielding Arc Welding of Carbon and Low Alloy Steel*

35 GB/T 8923.1 *Preparation of Steel Substrates before Application of Paints and Related Products - Visual Assessment of Surface Cleanliness - Part 1: Rust Grades and Preparation Grades of Uncoated Steel Substrates and of Steel Substrates after Overall Removal of Previous Coatings*

36 GB/T 9286 *Paints and Varnishes - Cross Cut Test for Films*

37 GB/T 9978 *Fire - resistance Tests - Elements of Building Construction*

38 GB/T 10045 *Flux - cored Electrode used for Non - alloy and Grain Refining Steel*

39 GB/T 11345 *Non - destructive Testing of Welds - Ultrasonic Testing - Techniques, Testing Levels, and Assessment*

40 GB/T 12470 *Low - alloy Steel Electrodes and Fluxs for Submerged Arc Welding*

41 GB/T 12615.1～GB/T 12615.4 *Closed End Blind Rivets with Break Pull Mandrel and Protruding Head*

42 GB/T 12616.1 *Closed End Blind Rivets with Break Pull Mandrel and Countersunk Head*

43 GB/T 12617.1～ GB/T 12617.5 *Open End Blind Rivets with Break Pull Mandrel and Countersunk Head*

44 GB/T 12618.1～GB/T 12618.6 *Open End Blind Rivets with Break Pull Mandrel and Protruding Head*

45 GB/T 12754 *Prepainted Steel Sheet*

46 GB/T 12755 *Profiled Steel sheet for Building*

47 GB/T 14957 *Steel Wires for Melt Welding*

48 GB/T 14978 *Continuously Hot - dip Aluminum-zinc Alloy Coated Steel Sheet and Strip*

49 GB/T 15856.1 *Cross Recessed Pan Headed Self - drilling and Self - tapping Screw*

50 GB/T 15856.2 *Cross Recessed Countersunk Head Drilling Screws*

51 GB/T 15856.3 *Cross Recessed Raised Countersunk Head Drilling Screws with Tapping Screw Thread*

52 GB/T 15856.4 *Hexagon Flange Head Drilling Screws with Tapping Screw Thread*

53 GB/T 15856.5 *Hexagon Washer Head Drilling Screws with Tapping Screw Thread*

54 GB/T 17493 *Low Alloy Steel Flux Cored Electrodes for Arc Welding*

55 GB/T 18981 *Fastener*

56 JG/T 203 *Method for Ultrasonic Testing and Classification for Steel Structures*

57 YB/T 4155 *Hot - rolled Round Carbon Steel Bars and Rods for Standard Parts*