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Developing connection design rules in China

Yongjiu Shi†

Department of Civil Engineering, Tsinghua University, Beijing 100084, China

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Abstract. The new version of Code for Design of Steel Structures (GB50017-2003) and other design standards in China were released over the last two years. Comparing with the previous version (GBJ17-88), many clauses covering the connection design have been revised. A number of additional provisions are supplemented to specify the design requirements for beam-column moment connections, as well as gusset plates for truss joints. In this paper, a summary on the design rules on connections specified in the current Chinese code is presented, and relevant commentary and background information is provided whenever appropriate. The design criteria governing weld and bolt resistance is examined and reviewed. Moreover, several issues such as detailing requirements for stiffeners and end-plate connections are discussed.

Key words: connection; bolt; weld; beam-column connection; truss joint; end-plate connection.

1. Introduction

A new series of design codes for steel structures in China were released in the recent two years. The design code for hot-rolled and welded steel structures is covered by GB50017-2003, and the cold-formed steel structures are regulated by GB50018-2002. In addition, some provisions are provided in CECS102: 2002 to specify the design procedures for end-plate connections, and in GB50011-2001 for connection behavior under seismic conditions.

Since the publication of GBJ17-88 which is the predecessor of GB50017-2003 in 1989, tremendous development on steel structures was observed in China. A large number of skyscrapers and long-span buildings in steel were designed and constructed around the country. Moreover, many research projects have been executed to investigate the performance of steel structures under various loading conditions. The Olympic Game in 2008 creates an unprecedented opportunity for promoting steel structures in Beijing. Both the engineering practice and the academic researches in recent years have provided solid foundations and background information for updating the steel design code in China. The new editions of BS5950, Eurocodes, AISC-LRFD also serve as important reference for China to scrutinize its structural design codes. The amendments and comments upon GBJ17-88 started in 1997, leading to the draft version of GB50017 introduced in 2001. After a trial period of two years in application and calibration, GB50017-2003 was formally published and superseded GBJ17-88 at the end of 2003. This new edition of steel design code has been prepared following the issue of a number of newly revised standards on design philosophy, materials, and loadings. The new provisions and updated clauses on steel structures include second-order analysis, brittle fracture under cold weather condition, post-

†Professor, E-mail: shiyj@mail.tsinghua.edu.cn

buckling resistance beam-column moment connection tubular structure etc. Some mandatory clauses have also been recommended on essential issues which should be implemented unconditionally. In this paper, the relevant clauses on connection design in GB50017-2003 are described and explained comprehensively. The content of this paper mainly covers the basic design resistance of bolts and welds while the design rules on beam-column connections and gusset plates are further discussed.

2. Welded connections

In general, three types of welds are permitted in structural steel construction: full penetration butt welds, fillet welds and partial penetration welds. The general design requirements specified by GB50017-2003 is described as follows:

2.1. Full penetration welds

When full penetration welds subject to tension or compression, the tension and the compression weld resistance N_t^w and N_c^w are given as follows:

$$N_t^w = l_w t f_t^w \quad \text{or} \quad N_c^w = l_w t f_c^w \tag{1}$$

where l_w is the effective length of the welds, taken as the weld length minus $2t$ if run-on/off tabs are not applied; t is the thickness of the thinner part connected; and f_t^w, f_c^w are tension and compression design strength of butt weld respectively, given by Table 1.

Table 1 Design strength of welds (N/mm^2)

Welding procedure and electrode	Connected parts		Full penetration butt weld				Fillet weld
	Steel grade	Plate thickness (mm)	Compression strength f_c^w	Tension strength f_t^w		Shear strength f_v^w	Tension, compression and shear strength f_f^w
				Class 1 and 2	Class 3		
SAW, MAW with E43 Electrode	Q235	≤16	215	215	185	125	160
		>16~40	205	205	175	120	
		>40~60	200	200	170	115	
		>60~100	190	190	160	110	
SAW, MAW with E50 Electrode	Q345	≤16	310	310	265	180	200
		>16~35	295	295	250	170	
		>35~50	2165	2165	225	155	
		>50~100	250	250	210	145	
SAW, MAW with E55 Electrode	Q390	≤16	350	350	300	205	220
		>16~35	335	335	285	190	
		>35~50	315	315	270	180	
		>50~100	295	295	250	170	
	Q420	≤16	380	380	320	220	220
		>16~35	360	360	305	210	
		>35~50	340	340	290	195	
		>50~100	325	325	275	185	

Under the combined action of moment and shear, the equivalent stress in butt welds is required to satisfy the following formula:

$$\sqrt{\sigma^2 + 3\tau^2} \leq 1.1 f_t^w \tag{2}$$

The full penetration welds are classified into 3 categories according to welding quality. The appropriate type of groove should be prepared on the basis of connected plate thickness and fabricating condition. GB50017-2003 assumes that the compression strength of qualified full penetration welds is always equal to that of the parent metal as long as the consumable electrode is compatible with the parent metal, while the tension strength of butt weld is assumed equal to that of the parent metal if the weld is qualified as Class 1 or Class 2 by visual inspection as well as non-destructive testing. In case of Class 3 butt weld, the tension strength of butt welds is taken as 85% of that of its parent metal. For full penetration welds with single groove, the design strength may be reduced to 85% of that of the parent metal if neither backing-strip nor root-seal run is applied. When the thicknesses or the widths of the connected plates differ by than 4 mm, a transition slope less than 1:2.5 should be prepared to minimize stress concentration effect.

It is required by the Code for Construction and Quality Acceptance of Steel Structures (GB50205-2001) that the weld quality is classified as Class 3 if only visual inspection is required and qualified. Every weld should be inspected by ultrasonic if Class 1 weld is required while more than 20% welds should be inspected by ultrasonic if Class 2 weld is required. Appropriate weld quality and inspection method should be specified in design documents according to loading conditions and structural systems. Full penetration Class 1 welds are required if the welds are subjected to dynamic and fatigue loading; Class 2 welds are acceptable if the full weld strength is required in the connected parts. Partial penetration Class 3 welds can be used if their resistances are complied with the relevant clauses satisfactorily.

2.2. Fillet welds

The forces transmitted per unit length of fillet welds at a given point is determined from the applied forces and moments through the use of elastic section properties of the welds, depending on their effective throat size h_e and leg size h_f. The design stress in a fillet weld is calculated as the applied force transmitted per unit length of weld, divided by the effective throat size h_e, which is taken as 0.7 h_f for right-angled welds or $h_f \cos\alpha / 2$ for acute and obtuse welds.

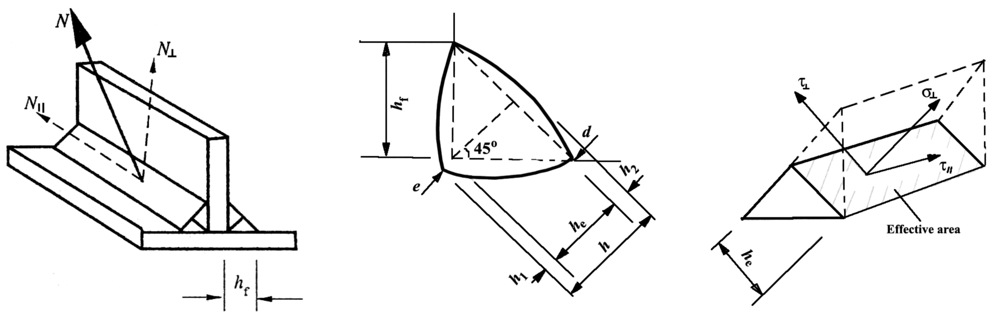


Fig. 1 Fillet weld

When the applied load is parallel to the weld axis (Fig. 1), which is generally referred as side fillet weld, the shear capacity, $N_{||}^{w}$, over the effective length of weld is given by:

$$N_{||}^{w} = h_e l_w f_f^{w} \tag{3}$$

where l_w is the effective length of the weld, taken as the weld length minus 2 h_f for each weld run; and f_f^{w} is the design strength of fillet weld which is determined according to the electrode and the steel grade as given in Table 1.

When the applied load is perpendicular to the weld axis, which is generally referred as end fillet weld, the shear capacity, $N_{\perp}^{w}$, is given by

$$N_{\perp}^{w} = \beta_f h_e l_w f_f^{w} \tag{4}$$

where β_f is the strength enlargement coefficient, which is equal to 1.22 for connection subject to static loading and 1.0 for acute and obtuse welds or under dynamic loading.

GB50017-2003 specifies that the fillet welds will not be allowed to carry load when the angle between the fusion faces of a weld is less than 60° or more than 135°. The leg size h_f of fillet welds should be less than 1.2 times the thickness of the thinner connected plate, but always be larger than $1.5\sqrt{t}$, where t is the thickness of the thicker connected part. The effective length l_w of a load carrying fillet weld should not be less than 8 h_f or 40 mm. In case of load parallel to the weld length, the effective length l_w should be limited and not larger than 60 h_f.

2.3. Partial penetration welds

Partial penetration welds can be used when their capacities are adequate to resist the applied loading. The weld performance is similar to the fillet welds and the design resistance is calculated from Eqs. (1) and (2) where the effective throat thickness h_e is determined from the groove depth s and the profiles as shown in Fig. 2. For single V groove, $h_e = s$ if $\alpha \geq 60°$, $h_e = 0.75\ s$ if $\alpha < 60°$, and $h_e = s$-3 for single and double bevel grooves, but the strength enlargement coefficient, β_f, is taken as 1.0 unless a compression force is perpendicularly applied onto the weld, in which case, $\beta_f = 1.22$.

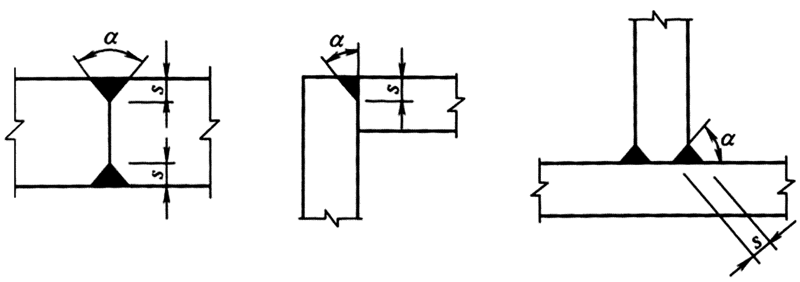


Fig. 2 Partial penetration welds

3. Bolted connections

The behavior of a bolted connection is governed by the resistances of both bolts and connected parts. There are two types of bolts are recommended by GB50017-2003: (1) ordinary bolts, where the loading is transmitted by bearing on the connected parts and shearing through the bolt shank; and (2) high strength bolt, where the loading is transmitted by friction between connected parts or by bearing as an ordinary bolt. In general, high strength bolts with their higher stiffness is often recommended for primary connections, while the ordinary bolts are employed in secondary structures or temporary structures. A rivet as a structural fastener is seldom applied in modern steel construction, but GB50017-2003 has preserved the clauses about rivets which may be beneficial to maintain historical metal structures.

3.1. Ordinary bolts

The ordinary bolts are classified as: (1) Types A and B, which are generally referred as precise bolts, with Grade 5.6 and 8.8; and (2) Type C, which is generally referred as coarse bolts with Grade 4.6 and 4.8. Type C bolts are cheaper and easy in fabrication and erection, but larger deformation is likely to happen under severe loading conditions, Type A or B bolts should be used.

The ordinary bolts can be used for transmitting shear and tensile forces. When a bolted connection is subjected to shear loading, the connection may fail by shearing in bolt shank or bearing in connected plies. The shear capacity of a bolt shank is given by

$$N_v^b = n\frac{\pi d^2}{4}f_v^b \tag{5}$$

where f_v^b is the design shear strength of the bolt material given in Table 2; d is the nominal diameter of the bolt, and n is the number of shear planes in the connection.

The bearing capacity of the connected plies is taken as:

Table 2 Design strength of bolts (N/mm^2)

Type of bolt and steel grade		Ordinary bolts						High strength bolt in bearing-type connection		
		Type C			Type A and Type B					
		Tension strength f_t^b	Shear strength f_v^b	Bearing strength f_c^b	Tension strength f_t^b	Shear strength f_v^b	Bearing strength f_c^b	Tension strength f_t^b	Shear strength f_v^b	Bearing strength f_c^b
Ordinary bolt	Grade 4.6,4.8	170	140							
	Grade 5.6				210	190				
	Grade 8.8				400	320				
High strength bolt in bearing-type connection	Grade 8.8							400	250	
	Grade 10.9							500	310	
Connected parts	Q235			305			405			470
	Q345			385			510			590
	Q390			400			530			615
	Q420			425			560			655

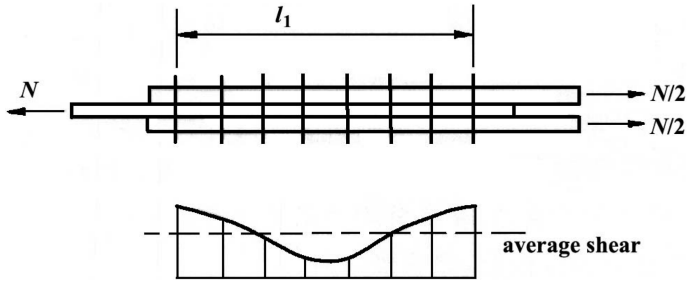


Fig. 3 Long shear joints

$$N_c^b = df_c^b \sum t \tag{6}$$

where f_c^b is the design bearing strength of the connected plies given in Table 2; and $\Sigma\, t$ is the lesser of the sum of the thickness of the connected plies. The shear capacity of an ordinary bolt connection is taken as the lesser of the values given in Eqs. (5) and (6).

The end and edge distances of a bolted connection should be adequate to prevent tearing failure. GB50017-2003 specifies that the distance from the center of a fastener hole to the adjacent end of a ply, measured in the direction of load transfer, should not be less than 2 d_o, where d_o is the hole diameter which is taken as the bolt diameter plus 1 to 2 mm. The distance from the center of a bolt hole to the adjacent edge of a ply, measured perpendicular to the direction of load transfer, should normally be not less than 1.5 d_o for shearing and torch cutting plates, or 1.2 d_o for sawing and flame cutting plates. The minimum spacing between the bolt centers is 3 d_o.

At least two bolts should be used in an effective connection. The tension resistance at the net section with bolt holes shall be checked at the ultimate limit state. Normally, the applied force is assumed uniformly distributed among the bolts. While in a long splice joint as shown in Fig. 3 where the distance l_1 between the end-rows of bolts exceeds 15 d_o, the shear force may not be uniformly distributed, and hence, the shear capacity obtained from Eqs. (5) and (6) should be reduced by

$$\eta = 1.1 - \frac{l_t}{150 d_o} \geq 0.7 \tag{7}$$

When an ordinary bolt is applied to resist tensile force N_t as shown in Fig. 4, the axial tension capacity

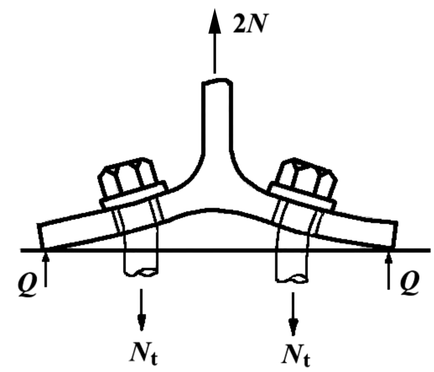


Fig. 4 Bolt in tension

of the bolt is given by

$$N_t^b = A_e f_t^b \tag{8}$$

where f_t^b is the design tensile strength of the bolt, given in Table 2; and A_e is the net area at the threaded portion of the bolt shank. The prying forces Q are neglected in the tension force calculation, but the connections are required to be stiffened, for example applying stiffeners or increasing plate thickness, to minimize any prying effect.

If the bolt is subjected to combined shear force N_v and tension force N_t, the following interaction formula should be satisfied:

$$\sqrt{\left(\frac{N_t}{N_t^b}\right)^2 + \left(\frac{N_v}{N_v^b}\right)^2} \le 1 \tag{9a}$$

and

$$N_v \le N_c^b \tag{9b}$$

The design resistance of the bolt is checked by Eq. (9a) while the bearing resistance of the connected plies is checked by Eq. (9b).

3.2. High strength bolts

Two types of high strength bolt grades are commonly available, namely, Grades 8.8 and 10.9, and they can be manufactured as hexagon or tension control bolts. Pretension should be applied during installation. The bolted connections can be designed as a friction type joint where the shear force is transferred between connected parts by friction on faying surface, or a bearing type joint where the shear force is transferred as an ordinary bolt. The frictional force is provided by the clamping action of the preloaded bolts. For a connection subjected to impact, dynamic loads and seismic actions where rigidity is essential, the friction type joint is recommended. For both frictional and bearing types of joints, pretension can be applied to the bolts either by a torque wrench or part-turn method for hexagon bolts, or sheared-off pintail method for a tension control bolt as specified in JGJ82-91.

The high strength bolts can be used for transmitting both shear and tensile forces. The shear resistance of a bolt in a frictional type joint is justified as a slip resistance, N_v^b, which is given by

$$N_v^b = 0.9 n_f \mu P \tag{10}$$

where P is the specific pretension of a high strength bolt which varies with bolt diameter and bolt grade as given in Table 3; μ is the slip coefficient which depends on the steel grade and the surface treatment of the connected parts ranging from 0.3 to 0.5, as shown in Table 4; and n_f is the number of shear

Table 3 Pretension in high strength bolt P (kN)

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

Table 4 Slip coefficient μ

Surface preparation	Steel grade		
	Q235	Q345 and Q390	Q420
Blast-cleaned	0.45	0.50	0.50
Blast-cleaned and Zinc-coated	0.35	0.40	0.40
Rusted after blast-cleaned	0.45	0.50	0.50
Hand-cleaned	0.30	0.35	0.40

planes, taken as either 1 or 2 in practice.

When a high strength bolt in a fictional type joint is required to carry external tension force, the transmitted design tension force, N_t^b, should not exceed the following value:

$$N_t^b = 0.8P \tag{11}$$

When a high strength bolt with pretension is applied in a bearing type joint, the design resistance is determined as an ordinary bolt from Eq. (5) to (9) but using the design strength of a high strength bolt.

The application of axial tension force on a preloaded bolt in a frictional type joint will reduce the clamping force between the connected parts, and a linear interaction formula is used in GB50017-2003 to check the combined actions:

$$\frac{N_t}{N_t^b} + \frac{N_v}{N_v^b} \le 1 \tag{12}$$

4. Beam-column moment connections

A beam-column moment connection in steel frames is required to have adequate resistance and to transmit the beam end moment without generating significant rotation deformation. The beam can be connected to the flange and the web of the column. The column web stiffeners can be applied in line with both flanges of the beam to increase the joint rigidity. Normally the beam flanges should be welded to the column with full penetration butt welds, while the beam web may be bolted or welded onto the column. GB50017-2003 provides clauses for checking the moment resistance of an I-shape beam to column flange connection.

4.1. Unstiffened column webs

If the column web is unstiffened as shown in Fig. 5, there are two critical regions to be considered: the compression zone and the tension zone. The column web may fail by yielding or buckling in the compression zone due to the compression force in the beam flange. The thickness of the column web should satisfy the following requirement:

$$b_e t_w f_c \ge A_{fc} f_b \tag{13a}$$

or

$$t_w \ge \frac{A_{fc} f_b}{b_e f_c} \tag{13b}$$

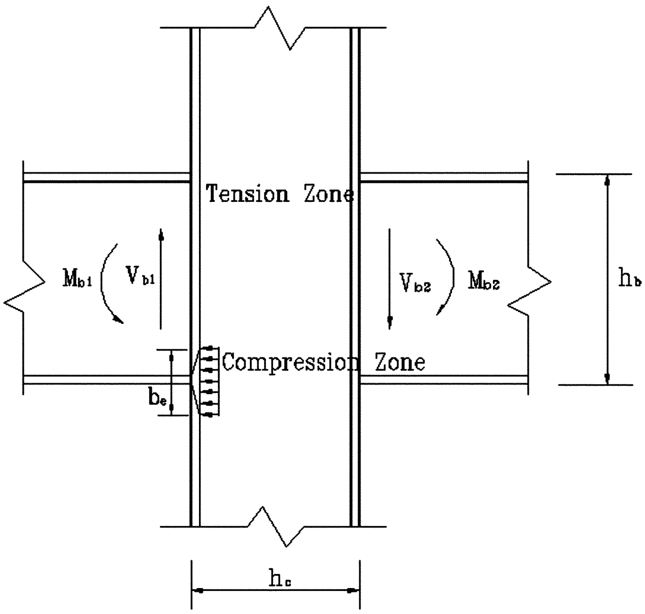


Fig. 5 Unstiffened web panel

where b_e is the effective width of the column web in the compression zone and taken as $b_e = t_b + 5h_y$; h_y is the distance from the beam end to the edge of effective depth of the column web which may be taken as the column flange thickness t_c; t_b is the thickness of the beam flange; A_{fc} is the area of the beam flange in compression, equal to $b_b t_b$; and f_b, f_c are the design strengths of the beam and the column respectively.

In addition, the web buckling should be prevented in the compression zone by limiting the column web depth to thickness ratio as follows:

$$h_c / t_w \leq 30\sqrt{\frac{235}{f_{yc}}} \tag{14a}$$

or

$$t_w \geq \frac{h_c}{30}\sqrt{\frac{f_{yc}}{235}} \tag{14b}$$

where h_c and t_w are the web depth and the thickness of the column in the joint panel; and f_{yc} is the yielding strength of the column web.

In the tension zone, the force is transmitted from the beam flange to the column through the column flange and (partially) through the column web. Assuming the column flange failed along the yielded line ABCD as shown in Fig. 6, the minimum column flange thickness t_{fc} required to resist the beam flange tension force is given as follows:

$$t_{fc} \geq 0.4\sqrt{A_{ft} f_b / f_c} \tag{15}$$

where A_{ft} is the area of the beam flange in tension.

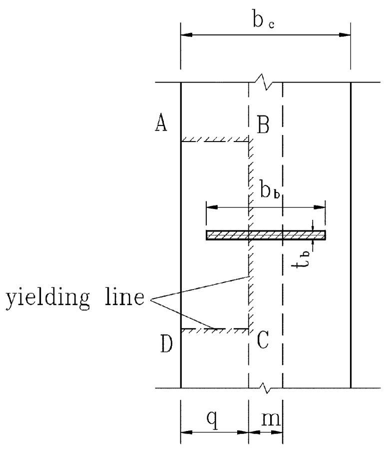


Fig. 6 Column flange in tension zone

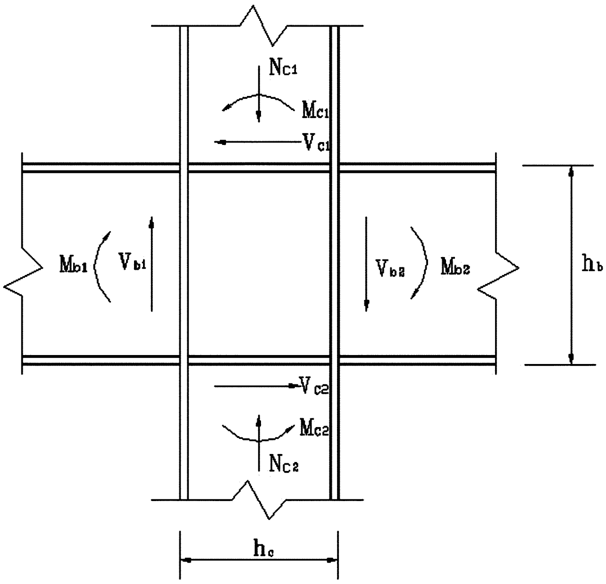


Fig. 7 Stiffened web panel

Case studies indicated that for unstiffened column webs, both the web and the flange thicknesses of a column should be increased significantly in the joint region, and leading to inefficient and uneconomical fabrication procedures. For connections subjected to seismic action, horizontal stiffeners must be provided with a thickness not less than that of the beam flange according to GB50011-2001.

4.2. Stiffened column webs

A stiffened column web could provide higher resistances in both the tension and the compression zones of a beam-column moment connection as shown in Fig. 7, but high shear force may be developed in the joint panel, causing shear yielding or buckling of the column web. The shear force induced on the joint panel, V, can be estimated as follows:

$$V = \frac{M_{b1} + M_{b2}}{h_b} - \frac{V_{c1} + V_{c2}}{2} \tag{16}$$

where h_b is the depth of the beam web; M_{b1}, M_{b2} are the applied moments at the beam ends, and V_{c1}, V_{c2} are the applied shear forces at the column ends.

Assuming that the applied axial forces and bending moments are resisted by the column flanges, and the shear force, V, given by Eq. (16) is uniformly distributed within the column web of the joint panel, the shear stress, τ, should be limited to

$$\tau = \frac{M_{b1} + M_{b2}}{h_b \cdot h_c t_w} - \frac{V_{c1} + V_{c2}}{2 \cdot h_c t_w} \leq f_v \tag{17a}$$

Neglecting the shear forces V_{c1}, V_{c2}, the shear stress τ will be slightly overestimated. Experimental results suggested that web stiffeners can also increase the shear resistance of the column web according to GB50017-2003, therefore, the design shear strength may be multiplied by a factor of 4/3 to enhance the structural adequacy. Thus, the shear stress of a joint panel is governed by:

$$\frac{M_{b1} + M_{b2}}{V_p} \leq \frac{4}{3} f_v \tag{17b}$$

where M_{b1}, M_{b2} are the applied moments at the beam ends; V_p is the volume of the joint panel given by $h_b h_c t_w$ for H sections and 1.8 $h_b h_c t_w$ for box sections. It is required that the web stiffeners should be provided at both the levels of the upper and the lower flanges of the beam, welded to the column flanges and web with full penetration welds.

The high shear force may also cause web buckling in the column in the joint panel, which can be prevented by controlling the column web thickness as follows:

$$t_w \geq \frac{h_c + h_b}{90} \tag{18}$$

If the shear capacity of a column web is found to be inadequate according to Eqs. (17) and (18), the web thickness should be increased at the joint panel or supplementary plates or diagonal stiffeners can be added to stiffen the web. The supplementary plate should be as wide as the column web and connected to the column flange by fillet welds. The stiffened region is required to cover and extend more than 150 mm above the top beam flange and below the bottom beam flange.

5. End-plate connections

End-plate connections are commonly used in portal frames in China where a beam-column moment connection and a beam-beam splice can be achieved by bolted end-plate connections. Their use in multi-storey frames is becoming more common recently. Extended end-plates are usually designed as rigid connections, but flushed end-plates are seldom used in China. It should be noted that the end-plate is butt-welded to a beam and bolted to a column with preloaded high strength bolts which is designed as a frictional type, with or without stiffeners. The design of an end-plate largely depends on the performance in the zone adjacent to the beam tension flange and its interaction with bolts. The design rules are

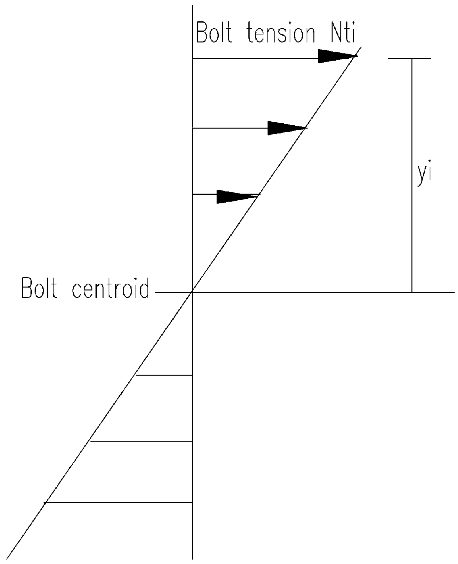


Fig. 8 Distribution of loads among bolts

presented in CECS102:2002 and JGJ82-91. It is essential to note that both the bolts and the end-plate should be checked with adequate resistances against internal forces. For a typical joint subjected to an axial force N, a shear force V and a moment M, JGJ82-91 assumes that the axial forces in the bolts at ultimate limit state are proportional to their distances from the center of rotation, which is assumed to be the centroid of the bolt group, as shown in Fig. 8. Both the shear and the axial forces are uniformly distributed among the bolts. The tensile force generated by the moment and transferred by a bolt in the tension zone is given by

$$N_{ti} = \frac{My_i}{\sum_j y_j^2} \tag{19}$$

where y_i denotes the distance of the bolt measured from the centroid of the bolt group. The maximum value of N_{ti} should be less than 0.8 P. The shear resistance of the bolt group, considering the interaction between the shear and the tensile forces is given by

$$N_v^b = 0.9n_f\mu\sum_i(P - 1.25N_{ti}) \tag{20}$$

where N_{ti} is taken as 0 for bolts in compression zone.

CECS102:2002 recommends that if the end-plate is relatively thin, or less stiffeners are used, the joint may fail in plastic yielding of the end-plate around the bolts. An analysis model based on yielded line theory is proposed to limit the minimum plate thickness. Considering the restraints provided by the stiffeners and the beam section to the end-plate, four types of boundary conditions are modeled in assessing the failure modes of the end-plate connection (Fig. 9). The minimum plate thickness should be determined as follows:

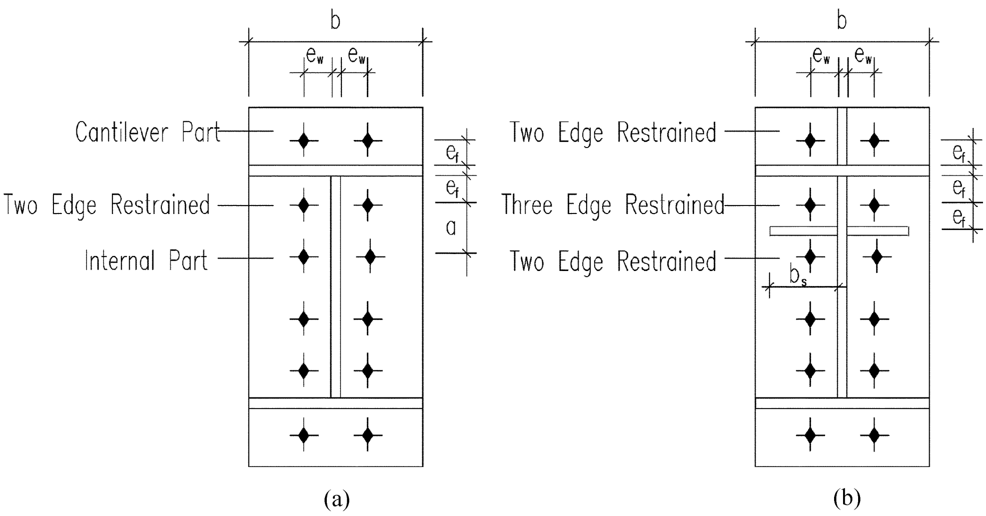


Fig. 9 End-plate in tension zone

For cantilever part as shown in Fig. 9(a):

$$t \geq \sqrt{\frac{6e_f N_t^b}{bf}} \tag{21a}$$

For internal zone as shown in Fig. 9(a):

$$t \geq \sqrt{\frac{3e_w N_t^b}{(0.5a + e_w)f}} \tag{21b}$$

For two edge restrained zone as shown in Fig. 9(b):

$$t \geq \sqrt{\frac{6e_w e_f N_t^b}{[e_w b + 2e_f (e_f + e_w)]f}} \tag{21c}$$

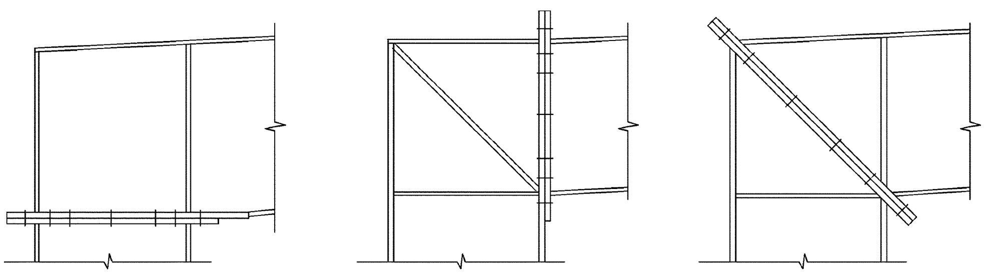
For three edge restrained zone as shown in Fig. 9(b):

$$t \geq \sqrt{\frac{6e_w e_f N_t^b}{[e_w(b + 2b_s)+4e_f^2]\, f}} \tag{21d}$$

where N_t^b is the design tension resistance of a high strength bolt determined from Eq. (11); e_w, e_f are the distances from the bolts to the stiffeners or to the beam flange respectively as shown in Fig. 9; b, b_s are the width of the end-plate and the stiffener; a is the bolt gauge; and f is the design strength of the end-plate.

The minimum thickness of the end-plate should not be less than 16 mm. Whenever possible, the end-plate should be stiffened by stiffeners to reduce any additional forces due to prying action.

The main application of the end-plate connections is in portal frames, such as the eave joint and the apex joint. The flange and the web of the beam or the column section are required to weld onto the end-plate with full penetration welds. The end-plates can be located horizontally, vertically and obliquely as shown in Fig. 10. The shear resistance of an unstiffened column web panel in a connection between the



(a) (b) (c)

Fig. 10 Column-rafter joint

column and the rafter should be checked as follows:

$$\frac{1.2M}{d_b d_c t_c} \leq f_v \tag{22}$$

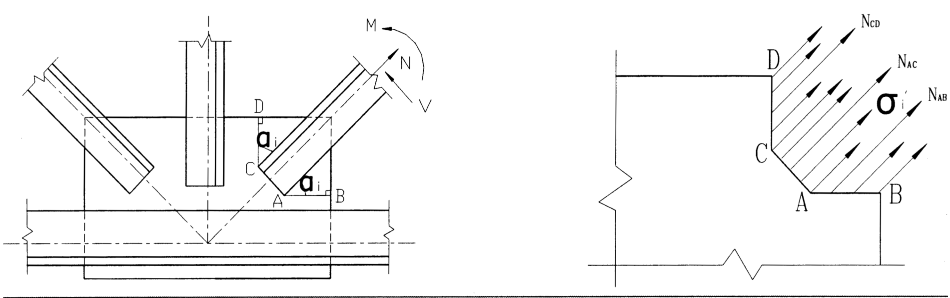
in which d_c, t_c are the width and the thickness of the web panel; d_b is the depth of the web panel; M is the bending moment resisted by the joint; and f_v is the shear strength of the web panel. If the strength of the web panel is inadequate, it is required to increase the web thickness or add diagonal stiffeners as shown in Fig. 10(b).

6. Truss joints

Joints between truss members should be capable of transmitting internal forces. In addition to the determination of the number of bolts and the weld length required in each joint, GB50017-2003 requires the gusset plate to be checked against rupture failure and local buckling.

6.1. Tension rupture

For a tension web member connected to a gusset plate by welding as shown in Fig. 11, tension rupture failure at the gusset plate should be prevented by checking its effective resistance. From the typical joint as shown in Fig. 11(a) where double angles are fillet-welded to the gusset plate, one possible



(a) (b)

Fig. 11 Gusset rupture failure

failure mode is section tearing along line BA-AC-CD, where lines BA and CD are perpendicular to the edge of the gusset plate. Assuming the tensile force on any failure section is N_i, the tensile stress and the shear stress along the failure line are given in Fig. 11(b) as follows:

$$\sigma_i = \sigma_i' \sin \alpha_i = \frac{N_i}{l_i t} \cdot \sin \alpha_i \qquad \tau_i = \sigma_i' \cos \alpha_i = \frac{N_i}{l_i t} \cdot \cos \alpha_i \tag{23}$$

The equivalent stress is

$$\sigma_{red} = \sqrt{\sigma_i^2 + 3\tau_i^2} = \frac{N_i}{l_i t}\sqrt{\sin^2\alpha_i + 3\cos^2\alpha_i} = \frac{N_i}{\eta_i l_i t} \tag{24}$$

where t is the thickness of the gusset plate; l_i is the length of ith failure line; α_i is the angle between the member axis and the failure line; and η_i is a parameter equal to $1/\sqrt{1+2\cos^2\alpha_i}$.

If the maximum equivalent stress σ_{red} is limited to the design strength of the gusset plate, f, the tension resistance of each failure section is given by

$$N_{ti} = \eta_i l_i t f \tag{25}$$

Therefore, the member tensile force N should not exceed the gusset rupture resistance, N_{Rt}, which is given by

$$N_{Rt} = \sum(\eta_i l_i t) f \tag{26}$$

For irregular gusset plates with quadrilateral or trapezoid plates, the length l_i of the critical failure line is difficult to estimate, and an alternative approach recommended in GB50017-2003 may be adopted where the direct stress at the end of each member should satisfy the following requirement:

$$N / b_e t \le f \tag{27}$$

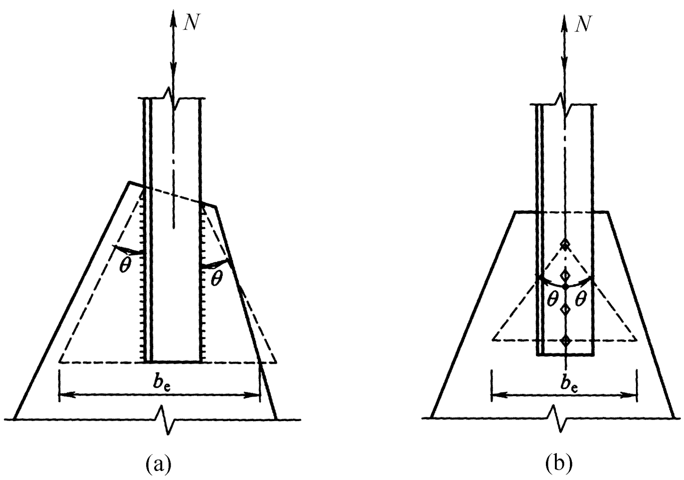


Fig. 12 Effective width of gusset plate

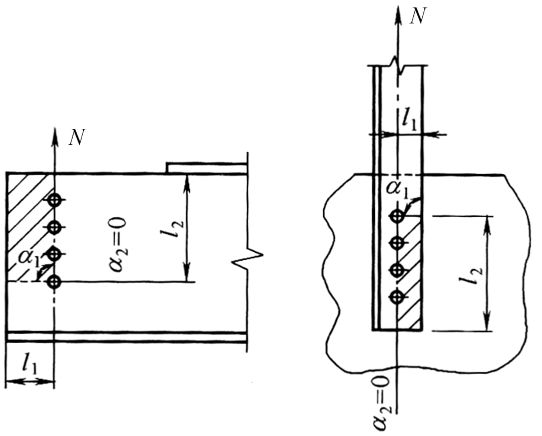


Fig. 13 Block shear failure

where b_e is the effective width within the gusset plate, and can be obtained by assuming the load disperse at θ=30° on each side of the member, as shown in Fig. 12. Eq. (27) is applicable to both fillet-welded and bolted connections.

The thickness of gusset plates can be selected according to the member internal force. A reference table was suggested in the commentary of GB50017-2003 to assist structural engineer. The gusset plate should be large enough to accommodate the necessary weld length or bolts.

6.2. Block shear

If a truss member is connected to a gusset plate with ordinary bolts, tearing-out failure may occur through a group of bolt holes at a free edge, as shown in Fig. 13. Assuming failure occurs at the net section along bolt holes accompanied by tensile rupture along the end bolt row, the block shear resistance can be calculated from Eq. (26), where a_i is taken as 0° for a failure line along the bolt group and 90° for a failure line perpendicular to the bolt line. Both truss members and notched beams should be considered with this failure mode.

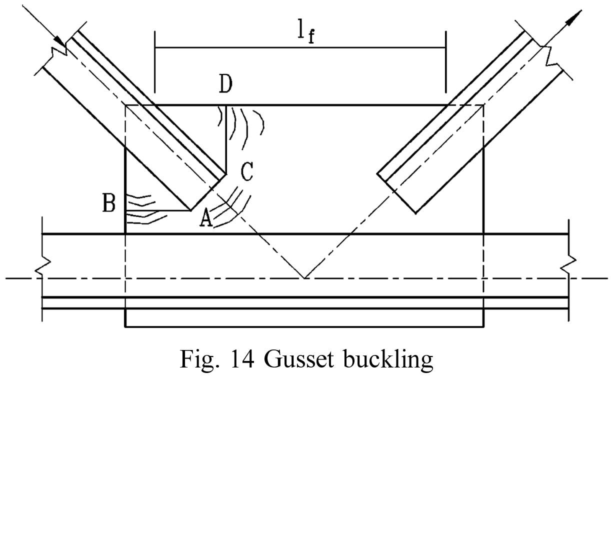


Fig. 14 Gusset buckling

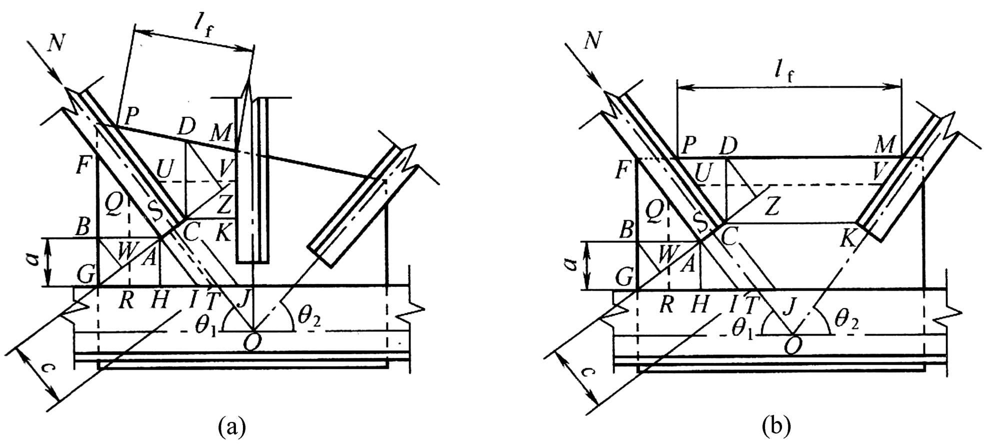


Fig. 15 Truss joint details

6.3. Buckling of gusset plates

In case of a thin gusset plate connected to a compression web member, the gusset plate may locally buckle and fail to transmit its internal force. From typical joint tests, it is noted that local buckling may be developed at the free edge of a gusset plate around the end of a compression member, as shown in Fig. 14.

GB50017-2003 specifies that if the vertical web member does not exist as shown in Fig. 15, buckling along a free edge of the gusset plate can be effectively prevented, provided that the unrestrained length l_f satisfies the following requirement:

$$l_f/t \le 60\sqrt{235/f_y} \tag{28}$$

where f_y is the yielding strength of the gusset plate. Otherwise longitudinal stiffeners should be provided.

It is found that buckling failure around a member end largely depends on the clear distance c between the end of each web member and the chord. It is suggested that in order to eliminate local buckling in the gusset plate, the ratio c/t should be limited to the following and local buckling can be avoided:

For a gusset plate connected to both diagonal and vertical web members as shown in Fig. 15(a):

$$c/t \le 15\sqrt{235/f_y} \tag{29a}$$

For a gusset connected to diagonal members only as shown in Fig. 12(b):

$$c/t \le 10\sqrt{235/f_y} \tag{29b}$$

7. Conclusions

In this paper, the newly implemented design rules for steel joints and connections commonly used in China are outlined. Comparing to the previous versions, more clauses on beam-column connections and truss joints have been provided in the new codes as effective design guidance to structural engineers. The new codes are more easy to use with less ambiguity, and provide better specification for structural steel design.

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