CONTENTS

1. Structural Design Procedure for Seismic in Steel Structure (BSL of JAPAN)

1.1. General

Structural design procure which can be used for building is specified in The Building Standard Law of Japan (BSL). Firstly, the building which height is less than or equal to 60m can be design by standard procedure stipulated in BLS. It is so called equivalent lateral force procedure. Standard procedure specified in BLS was compiled in 1981; minor update is conducted almost every year. The validity of the basic concept of this procedure was proved though Kobe Earthquake (1995) and Tohoku Earthquake (2011). The building which height is greater than 60m (hereinafter called high rise building) have to conduct nonlinear dynamic response time-history analyses; the design process should get an endorsement for the scientific committee. The design methodology which have to be used for high rise building is specified by Minister of land, infrastructure and transportation.

Additionally, two following design procedures are also allowed to use by BLS that are equivalent to "*equivalent lateral force procedure*". To achieve performance based design, these procedures were specified.

- The Calculation Method of Response and Limit Strength (2000, 2013); and
- Energy Balance-Based Seismic Resistance Design Procedure (2005, 2013).

Finally, three design procedures are specified in Japan. In the next section, standard design procedure "*equivalent lateral force procedure*" is introduced.

1.2. Equivalent Lateral Force Procedure

Generally, three design procedures are stipulated in BSL for equivalent lateral force procedure. These design procedures are so called "Route" by designers. Following sections will show the detail of each design "Route". "Route 3" is the most sophisticated standard procedure and provide the designer to make their own decisions. "Route 2" is simplified procedure based on "Route 3"; however, limited flexibility due to the simplification. Finally, "Route 1" is the most simplified procedure which is based only on elastic design and only allowed to use in small buildings.

1.2.1. Design Procedure so called "Route 3"

If the designer decided to follow the standard procedure stipulated in BLS, building height greater than 31m and less than or equal to 60m should follow this procedure. Building that height is less than or equal to 31m is also possible to use this design procedure.

Basically, two phases of design should be conducted. "Phase 1" is allowable stress design, and "Phase 2" is ultimate strength design. Following shows each design phase.

Phase 1: Allowable Stress Design (ASD) is conducted under Serviceability Limit State (SLS) and Damage Limitation State (DLS). Moderate seismic event is included in DLS. The return period of seismic event considered in this phase is around 50 years; having around 20% exceedance probability in 10 years (EN1998: 10% exceedance probability in 10 years; 95 years return period). This phase can be also called "Elastic Design".

- Based on basic load combination for SLS and DLS, compute the design forces;
- Following equation should be fulfilled. Long term and short term should be checked, respectively. (Load combinations are tabulated in Table 1);

$$\sigma \le f \tag{1-1}$$

where, f is allowable stress of the structural material (short term and long term values are specified in the BSL), σ is the design stress due to axial, bending, and shear forces (design forces).

- Check severability deflection and/or vibration.

Table 1.2.1 Load combination considered in structural design

Duration of Force	Condition	Combination	
		Standard	Heavy Snow Region
Long term (SLS)	Regular	G + P	G + P
	Snow		G + P + 0.7S
Short term (DLS)	Snow	G + P + S	G + P + S
	Strong Wind	G + P + W	G + P + W
			G + P + 0.35S + W
	Earthquake	G + P + K	G + P + 0.35S + K

where, G is Dead Load effects, P is Live Load effects, S is Snow Load effects, W is Wind load effects, and K is Seismic Load effects.

NOTES:

Table 1.2.2 Example of Allowable Stress

Allowable Stress	Long term (SLS)	Short term (DLS)
Tensile stress, f_t	$\dfrac{F}{1.5}$	1.5×(Long term values)
Shear stress, f_s	$\dfrac{F}{1.5\sqrt{3}}$	

F is the specified strength according to JIS (Japanese Industrial Standard)

Table 1.2.3 Example of Uniformly Distributed Live Loads

Occupancy of Use	Load for floor or secondary beam design (N/mm^2)	Load for girder (Beam for MF), column or secondary beam design (N/mm^2)	Load for Seismic action (N/mm^2)
Offices	2 900	1 800	800
Classrooms	2 300	2100	1 000
Assembly areas			
Fixed seats	2 900	2 600	1 600
Movable seats	3 500	3 200	2 100
Roofs (Usually used)			
school and stores	2 900	2 400	1 300
Others	1 800	1 300	600
Storage of motor vehicles	5 400	1 300	600

Phase 2: Structural safety should be confirmed under Ultimate Limit State (ULS). Significant seismic event should be considered in this phase. The return period of seismic event considered in this phase is around 500 years; having around 10% exceedance probability in 50 years (EN1998: 10% exceedance probability in 50 years; 475 years return period). This phase can be also called "Plastic Design".

- Firstly, story drift ratio (SDR) based on load combination for earthquake in SLS should be fulfil;

$$SDR_i \leq 1/200 \tag{1-2}$$

where, SDR_i is the story drift ratio at i story. This value can be relaxed to 1/120 when the non-structural components are not affected by the structure deformation.

- Design resistance of the building under lateral forces (the horizontal load-carrying capacity), $Q_{u,i}$, should be greater than or equal to the load action, $Q_{un,i}$. Following equation should be fulfilled.

$$Q_{un,i} \leq Q_{u,i} \tag{1-3}$$

where, $Q_{u,i}$ is the lateral design resistance at i story (the horizontal load-carrying capacity), $Q_{un,i}$ is the load action due to horizontal forces at i story. $Q_{un,i}$ is computed as follows;

$$Q_{un,i} = D_{s,i} \cdot F_{es,i} \cdot Q_{ud,i} \tag{1-4}$$

where, $D_{s,i}$ is the ductility reduction factor determined from the member sizes, i.e. compactness (0.25 to 0.50), $F_{es,i}$ is shape factor that will consider the irregularity of the plan and elevation (1.0 to 3.0), and $Q_{ud,i}$ is the load action at i story determined by linear elastic responses. $Q_{ud,i}$ is determined as follows;

$$Q_{ud,i} = C_i \cdot W_i \tag{1-5}$$

where, C_i is the seismic story shear (force) coefficient at i story, and $W_{,i}$ is the total weight supported at i story ($= \sum_{j=i}^{n} w_j$). C_i is determined as follows;

$$C_i = Z \cdot R_t \cdot A_i \cdot C_0 \tag{1-6}$$

where, Z is the region coefficient (0.7 to 1.0), R_t is normalized elastic response acceleration, A_i is the coefficient for lateral force distribution ($\geq$1.0), and C_0 is the standard seismic shear (force) coefficient (for ULS $C_0 \geq 1.0$, for SLS $C_0 \geq 0.2$).

USER NOTES

Ductility reduction factor D_s is considering the inelastic behavior of the structure. This means that in ULS design, structural components are permitted to have inelastic deformations. Compact section delays local buckling behavior, and smaller D_s value can be used in design. (D_s has a similar meaning of *behavior factor q*; however, in inverse value.)

Typically, C_0 =1.0 is used for ULS design, and C_0 =0.2 for SLS and story drift check. A_i is the coefficient to consider the snap effect of the building; A_i should be always greater than or equal to 1.0. Flexible structure will have higher A_i values in the upper stories.

- Structural requirements
  - When steel grade STKR is used for columns, all joints should satisfy the following condition. This requirement can be waived at column base and top level of the multi-

story building.

$$\sum M_{pc} \geq 1.5 \sum M_{bp} \qquad (1\text{-}7)$$

where ΣM_{pc} is the summation of full-plastic moment (design resistance) of the columns that is considering the subjected compressive axial forces, and ΣM_{pb} is the summation of the full-plastic moment (design resistance) of the beam. M_{pc} and M_{pb} are the values at the beam face and column face, respectively. Additionally, the design resistance at the column base should be greater than 1.4 times the design forces.

- When the steel grade of the column is not STKR, each story should satisfy the following condition. This requirement can be waived at column base and top level of the multi-story building.

$$\sum M_{pc} \geq \sum \min\{1.5M_{bp}, 1.3M_{pp}\} \qquad (1\text{-}8)$$

- Otherwise, the story where does not satisfy Eq. (1-8) should be assumed that the design resistance of the columns in that story will be reduced by the factor in Table 1.2.3. The design resistance at each story that are based on the reduced resistance of the column should be greater than the design forces (action). At this moment, resistance of the columns at the bottom of the first story and at the top of the top story should be also reduced by same value.

Table 1.2.3 Reduction Factor for column which does not fulfill Eq. (1-8)

Steel Grade	Reduction Factor
BCR	0.75
BCP	0.80

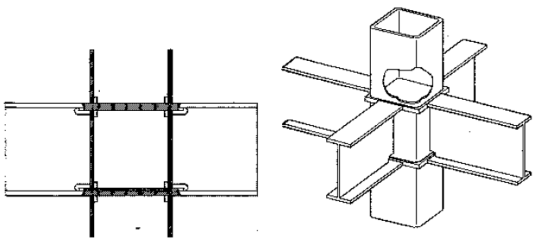


Figure 1.2.1 Typical Beam-to-Column Connection
(Continuity plates are installed in the column, called Diaphragm in Japan)

COMMENTARY

Ductility Reduction Factor D_s

Ductility reduction factor D_s should be determined from Table C1.2.1. Ductility reduction factor will be determined from the classes of moment frame members (beams and columns) and braces. Classification of structural components are determined from Table C1.2.2 to C1.2.4. To adopt this values, following conditions should be fulfilled.

- Brace joint should be full strength;
- Column joint and beam joint at the connection should be full strength; and

- Appropriate lateral bracings are supplied to the beam to prevent drastic strength deterioration.

Table C1.2.1 Matrix of Ductility Reduction Factor

D_s values			Classification of Group of Beams and Columns			
			A	B	C	D
Classification of Group of Braces	A or β_u =0		0.25	0.3	0.35	0.40
	B	$\beta_u \leq 0.3$	0.25	0.3	0.35	0.40
		$0 < \beta_u \leq 0.7$	0.3	0.3	0.35	0.45
		$\beta_u \geq 0.7$	0.35	0.35	0.4	0.50
	C	$\beta_u \leq 0.3$	0.3	0.3	0.35	0.40
		$0 < \beta_u \leq 0.5$	0.35	0.35	0.4	0.45
		$\beta_u \geq 0.5$	0.4	0.4	0.45	0.50

βu is the contribution of the brace to the horizontal design resistance.

Table C1.2.2 Classification of Structural Components

Class of Structural Components	Contribution of Resistance
A	$\gamma_A \geq 0.5$ and $\gamma_C \leq 0.2$
B	$\gamma_C < 0.5$
C	$\gamma_C \geq 0.2$

γ_A is the contribution ratio from BA class or FA class structural components. For brace, the ratio is computed from the summation of class BA brace horizontal resistances with the summation of all brace horizontal resistances. For moment frame members, the ratio is computed from the summation of class FA column horizontal resistances with the summation of all column horizontal resistances excluding class FD columns.

γ_C is the contribution ratio of BC class or FC class structural components. For brace, the ratio is computed from the summation of class BC braces horizontal resistance with the summation of all braces horizontal resistance. For moment frame members, the ratio is computed from the summation of class FC columns horizontal resistance with the summation of all columns horizontal resistance excluding class FD columns.

Classification of brace and moment frame members are tabulated in Table C1.2.3 and C1.2.4, respectively.

Table C1.2.3 Classification of Brace, maximum effective slenderness ratio

Class of Brace	Effective Slenderness Ratio
BA	$\lambda \leq 495\sqrt{F}$
BB	$495\sqrt{F} < \lambda \leq 890\sqrt{F}$ or $1980\sqrt{F} \leq \lambda$
BC	$890\sqrt{F} < \lambda < 1980\sqrt{F}$

λ is the effective slenderness ratio ($=k_c \cdot l_c / r$), r is the radius of gyration of buckling axis.

Table C1.2.4 Classification of Moment Frame Members, maximum width-to-thickness ratio

Class of Beam and Column	Column				Beam	
	Wide flange		Square Hollow	Circular Hollow	Wide flange	
	Flange	Web			Flange	Web
FA	$9.5\sqrt{235/F}$	$43\sqrt{235/F}$	$33\sqrt{235/F}$	$50(235/F)$	$9\sqrt{235/F}$	$60\sqrt{235/F}$
FB	$12\sqrt{235/F}$	$45\sqrt{235/F}$	$37\sqrt{235/F}$	$70(235/F)$	$11\sqrt{235/F}$	$45\sqrt{235/F}$
FC	$15.5\sqrt{235/F}$	$48\sqrt{235/F}$	$48\sqrt{235/F}$	$100(235/F)$	$15.5\sqrt{235/F}$	$48\sqrt{235/F}$
FD	Larger values than shown above.					

F is the specified strength according to JIS (Japanese Industrial Standard)

1.2.2. Design Procedure so called "Route 2" (Simplified)

Simplification can be applied on Phase 2 design, i.e. plastic design. To apply this design procedure, building height should be less than or equal to 31m. Basically, this procedure is based on allowable stress design with some geometrical and structural requirements to guaranty plastic deformation at the expected components; therefore, this is so called "Equivalent allowable stress design". Geometrical and structural requirements will implicitly guaranty the high deformability of structural members. For Moment Resisting Frame system (MRFs), ductility reduction factor D_s is expected to be 0.25 which is used for the most ductile structural system.

Phase 1: Same as Section 2.1.

Instead of conducting plastic design, followings have to be satisfied. Basically, this simplified procedure can be applied to a building which have regularity in elevation and plan.

- Firstly, story drift ratio (SDR) based on load combination for earthquake in SLS should be fulfil;

$$SDR_i \leq 1/200 \quad (1\text{-}9)$$

where, SDR_i is the story drift ratio at i story.

- Eccentricity requirements

<elevation>

$$R_s \geq 0.6 \quad (1\text{-}10)$$

where, R_s is story stiffness ratio. Stiffness uniformity along the height of the structure should be satisfied.

<plan>

$$R_e \leq 0.15 \quad (1\text{-}11)$$

where, R_e is eccentricity ratio. To avoid torsional behavior of the structure, eccentricity between center of mass and the center of stiffness should be limited.

<building shape>

$$H/L \leq 4 \quad (1\text{-}12)$$

where, H is the height, and L is the width of the building. Overturning should be avoided.

- Structural requirements
    - ➢ Design forces should be amplified corresponding to the contribution of the brace resistance to the horizontal design resistance at each story (1.0 to 1.5, MRFs is 1.0);

Table 1.2.4 Design Force Amplification Factor

Brace Contribution	Application Factor
$\beta \leq 5/7$	$1+0.7\beta$
$\beta > 5/7$	1.5

β is the contribution of the brace to the horizontal force resistance.

- Capacity design should be conducted at connections;
- Prevent local buckling. Cross-sectional class should be class FA (class 1);

Table 1.2.5 Maximum Width-to-Thickness Ratio

Member	Cross-section	Element	Maximum width-to-thickness ratio
Column	Wide flange	Flange	$9.5\sqrt{235/F}$
		Web	$43\sqrt{235/F}$
	Square Hollow	-	$33\sqrt{235/F}$
	Circular Hollow	-	$50(235/F)$
Beam	Wide flange	Flange	$9\sqrt{235/F}$
		Web	$60\sqrt{235/F}$

F is the specified strength according to JIS. F should be between 205 N/mm$_2$ to 375 N/mm^2.

- Column Overstrength Factor (COF) should be satisfied (COF $\geq$1.5), see Eq. (1-7). This requirement can be waived at column base and top level of the multi-story building.
  - ✧ Additionally, when steel grade STKR is used for first story column, the design resistance at the column base should be greater than 1.4 times the design forces.
- Design Resistance of the column base joint should be greater than the demand.

1.2.3. Design Procedure so called "Route 1" (Simplified)

This design procedure can be applied to a building that is less than or equal to 13m. Two design branches are prepared; Route 1-1 and Route 1-2. Route 1-1 and 1-2 have geometrical and structural requirements that should be fulfilled, respectively. Basically, "allowable stress design" is only conducted in this design routes. Requirements in "Route 1-2" are between "Route 1-1" and "Route 2".

1.2.3.1. Design Route 1-1

This design route can be used when the building satisfies following configurations.

- Less than or equal to 3 stories;
- Bay width is less than or equal to 6.0m;
- Total area is less than or equal to 500m^2.

Phase 1: Similar to Section 1.2.1. Instead, seismic load effect should be increased 1.5 times larger than other design routes, i.e., seismic load coefficient should be $C_0 \geq 0.3$ (Typically, $C_0 = 0.3$ is used).

- Structural requirements
  - Capacity design should be conducted at connections;

- Design forces at Columns, where are cold-formed rectangular hollow sections, should be amplified. Amplification factor will depend on the steel grade (STKR, BCR, or BCP).

Table 1.2.6 Design Force Amplification Factor for Square Hollow Section

Steel Grade	Typical Beam-to-Column Connection
STKR	1.4
BCR	1.3
BCP	1.2

See Figure 1.2.1 for typical beam-to-column connetion in Japan.

1.2.3.2. Design Route 1-2

This design route can be used when the building satisfies following configurations.

- Less than or equal to 2 stories;
- Bay width is less than or equal to 12m;
- Total area is less than or equal to 500m^2 (one story building can be less than or equal to 3000m^2).

Phase 1: Similar to Section 1.2.1. Instead, seismic load effect should be increased 1.5 times larger than other design routes, i.e., seismic load coefficient should be $C_0 \geq 0.3$ (Typically, $C_0 = 0.3$ is used).

- Structural requirements
  - Eccentricity Ratio (see Eq. 1-11) should fulfil, otherwise the structure should be designed by "Route 3".
  - Prevent local buckling. Cross-sectional class should be class FA (class 1) See Table 1.2.5;
  - Capacity design should be conducted at connections;
  - Design forces at Columns, where are cold-formed rectangular hollow sections, should be amplified. Amplification factor will depend on the steel grade (STKR, BCR, or BCP). See Table 1.2.6 and Figure 1.2.1; and
  - Design Resistance of the column base joint should be greater than the demand.

1.3. Definitions of Symbols

R_s : Story Stiffness Ratio

$$R_s = \frac{r_s}{\bar{r}_s} \tag{1-13}$$

where, r_s is the inverse value of story drift ratio at each story ($=\delta/h$), δ is the interstory drift, and h is story height. $\bar{r}_s$ is the arithmetic mean of r_s.

R_e : Eccentricity Ratio

$$R_e = \frac{e}{r_e} \tag{1-14}$$

where, e is the distance between the center of stiffness and the center of mass, measured normal to the direction of analysis considered, r_e is the square root of the ratio of the torsional stiffness to the lateral stiffness in direction of analysis considered.

F_{es} : Shape Factor (penalty due to irregularity in elevation and plan)

$$F_{es} = F_e \cdot F_s \tag{1-15}$$

where, F_e is the penalty factor to consider the irregularity in elevation, and is the penalty factor to consider the irregularity in plan. Both can be determined by following equations. F_e is determined by R_e, and F_s is determined from R_s, respectively.

$$F_e = \begin{cases} 1.0 & R_e \le 0.15 \\ 0.5 + \dfrac{10}{3} R_e & 0.15 < R_e < 0.30 \\ 1.5 & R_e > 0.30 \end{cases} \tag{1-16}$$

$$F_s = \begin{cases} 2.0 - \dfrac{5}{3} R_s & R_s \le 0.6 \\ 1.0 & R_s > 0.6 \end{cases} \tag{1-17}$$

R_t : Normalized (non-dimensional) elastic acceleration spectrum

$$R_t = 1.0 \qquad T \le T_c \tag{1-18.a}$$

$$R_t = 1 - 0.2\left(\frac{T}{T_c} - 1\right)^2 \qquad T > T_c \tag{1-18.b}$$

where T is the fundamental period of the structure, T_c is transition period of the ground (see Table C1.3.1)

Table C1.3.1 Transition periods of the Ground, T_c

Ground Type	T_c (s)
1 (hard)	0.4
2 (medium)	0.6
3 (soft)	0.8

A_i : Coefficient for lateral force distribution

$$A_i = 1 + \left(\frac{1}{\sqrt{\alpha_i}} - \alpha_i\right) \cdot \frac{2T}{1+3T} \tag{1-19}$$

where α_i is weight ratio that is determined as follow. W is the total weight of the building, and w_j is the weight of j story.

$$\alpha_i = \frac{\sum_{j=i}^{n} w_j}{W} \tag{1-20}$$

T : Fundamental period of the structure

$$T = (0.02 + 0.01\alpha)H \tag{1-21}$$

where H is height of the building in meter, and α is the ratio of the steel structure in the building.
Note: Fundamental period can be computed from eigenvalue analysis; however, the response acceleration computed from this value cannot be smaller than the 75% of the value computed from eq. (1-21).

2. Recommendation for Design of Connections in Steel Structure (AIJ)

2.1. Beam-to-Column Connection Deign

a) Beam-to-column connection should be designed to have sufficient strength to transmit demand forces in connected beam and column.

b) Column joint that is designed to be rigid should have greater or equal yield and ultimate strength than member strength base on gross area.

c) Beam joint that is designed to be rigid and the connected beam is expected to accommodate plastic deformation in Ultimate Limit State (ULS), beam joint strength should be greater or equal than the force corresponding to the required deformation capacity of the beam. Structural designer should select steel grade, detailing, and fabrication process. Beam joint strength should satisfy following equation:

$$\alpha \cdot {}_bM_p \leq {}_jM_u \tag{2-1}$$

where:

${}_jM_u$ is the maximum bending strength of beam joint;

${}_bM_p$ is the full plastic moment of the beam ($= Z_p \times F_y = W_{pl} \times f_y$);

Z_p is the plastic section modulus;

F_y is the minimum specified yield stress;

α is beam joint coefficient. Without specific consideration, values shown in Table 2.1.1 can be used.

Table 2.1.1 Beam joint coefficient

Steel Grade	Expected Failure Mode	
	Base Metal Fracture	High Strength Bolt Fracture
SS400	1.40	1.45
SM490	1.35	1.40
SN400	1.30	1.35
SN490	1.25	1.30

a) Beam joint that is designed to be semi-rigid or normally pinned, joint should be designed in detail that can accommodate the required rotation capacity with predictable structural behavior until ULS.

COMMENTARY

2.1.1. Component of Beam-to-Column Connection

Beam-to-column connection is composed by three components. Column joint, beam joint, and panel zone. Joints can be classified as rigid, semi-rigid, and normally pinned.

2.1.2. Rigid Beam Joint

In usual portal frame (moment resisting frame), rigid beam joint is assumed in structural analysis model. Rigid beam joint should have negligible local deformation in elastic design, and should have greater strength than full plastic moment of the beam with the capacity to accommodate required plastic deformation of the beam. To achieve rigid beam joint, it is typical in Japan to weld beam flanges by Complete Joint Penetration (CJP) and beam web is connected by either welded or bolted joints. It is also typical that continuity plates are install in hollow section columns in the same plane of beam flanges. Beam joints that can be categorized to rigid joint is summarized in Figure C2.1; however, the most typical detail used in Japan is (b-1).

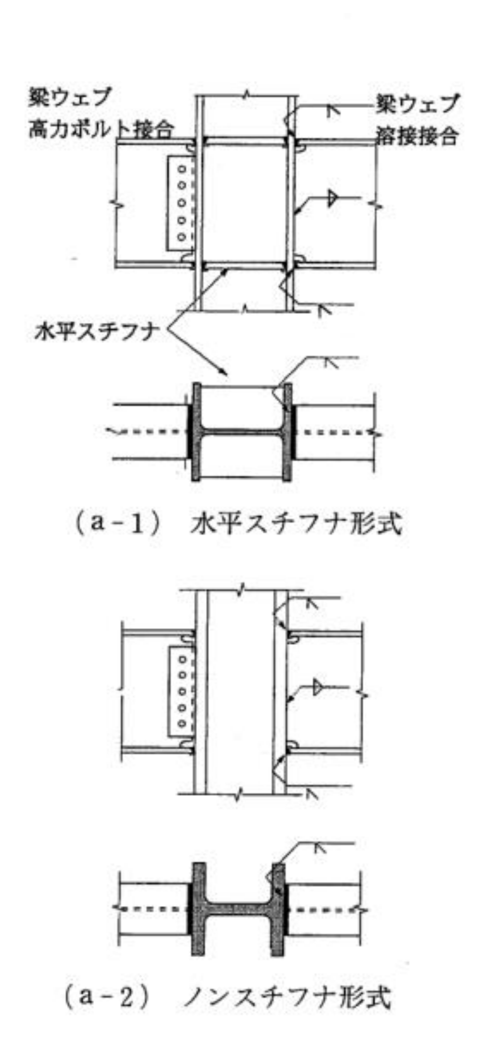
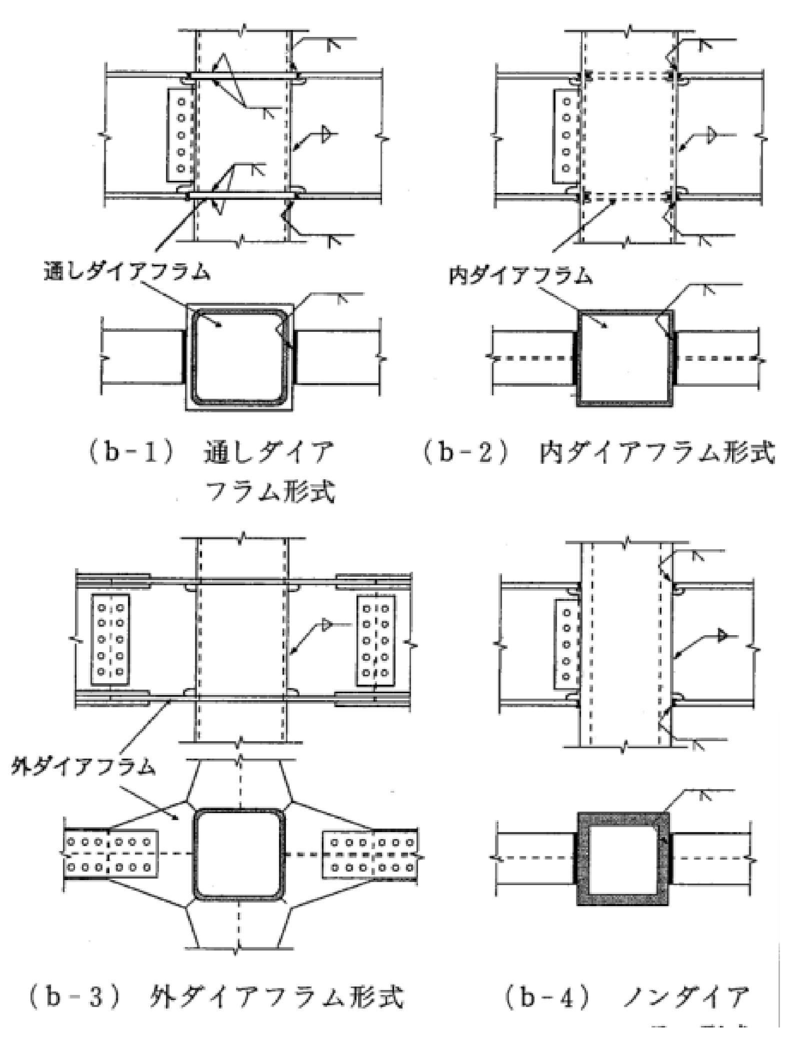
(a)H section column	(b) Hollow section Column
	

Figure C2.1 Typical rigid beam joint

2.1.3. Required Strength in Beam Joint

To dissipate significant seismic event input energy, beams that have rigid joints are expected to form plastic hinges in the beam at ULS. To guaranty this philosophy, beam joint should not allowed to have fracture during seismic event, and strength of the beam joint (Capacity) should be greater or equal than the expected force (Demand). Demand in the beam joint is the result of beam plastification with strain and cyclic hardening.

Based on the nominal full plastic moment of the connected beam, following equation should be satisfied in the beam joint.

$$\alpha \cdot {}_b M_{p(\mathrm{nominal})} = \alpha \cdot \left(Z_p \cdot F_y \right) \leq {}_j M_u \tag{2-2}$$

To prevent fracture at the beam joint, joint strength should be greater or equal than the demand.

$$ {}_{b}M_{\max} \leq {}_{j}M_{u} \tag{2-3}$$

where ${}_{b}M_{\max}$ is the maximum force in the beam. Plastification of the beam will lead strain and cyclic hardening and the demand in the beam joint will be greater than full plastic moment of the beam.

$$ {}_{b}M_{p} \leq {}_{b}M_{\max} \leq {}_{j}M_{u} \tag{2-4}$$

The effect of strain and cyclic hardening will be considered in coefficient ξ, and the difference between specified minimum yield stress and expected yield stress will be considered in coefficient β. Finally, beam joint coefficient will be determined as follows:

$$ \alpha = \xi \cdot \beta = \frac{{}_{b}M_{\max}}{{}_{b}M_{p(\mathrm{nominal})}} \tag{2-5}$$

2.1.3.1. Strain and Cyclic Hardening Effect ξ

In ULS seismic design, beams are expected to dissipate seismic energy. Beams are allowed to have plastification, and beam ends where the forces will be the greatest parts will overcome the full-plastic moment of the beam due to strain and cyclic hardening. The effect of hardening will be characterized by coefficient ξ (see Figure C2.2).

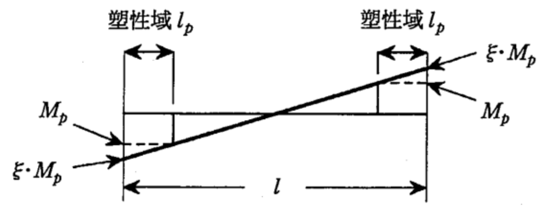


Figure C2.2 Plastification of the Beam ends during seismic event and hardening effects

Figure C2.3 shows the hardening effect of beams based on elasto-plastic portal frame time-history analyses. Vertical axis is hardening ratio (${}_{b}M_{\max}/{}_{b}M_{p}$) and horizontal axis is plastification ratio (${}_{b}\theta_{p,\max}/{}_{b}\theta_{pp}$). Hardening effect corresponds to the amount of plastification; as an example, ξ =1.2 is expected when the beam accommodates 5 times of the elastic deformation.

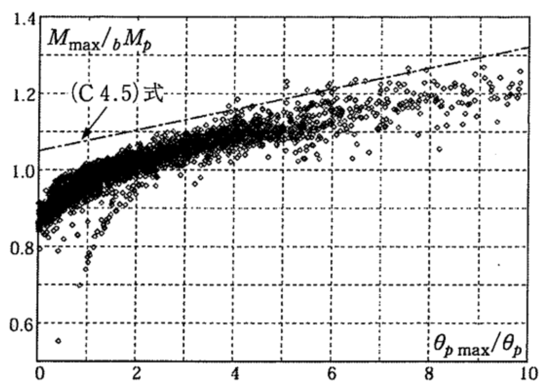


Figure C2.3 Hardening effect due to plastic deformation of the beams

2.1.3.2. Strength Randomness β

Structural design will be conducted based on specified minimum yield stress. However, yield stress of the steel should have random values; it shall be greater than design value. Demand and capacity ration can be expressed as following equation considering actual strength.

$$\xi\left(\beta_b \cdot {}_b M_p\right) \leq \left(\beta_j \cdot {}_j M_u\right) \tag{2-6}$$

where, β_j, β_b are the coefficients to consider actual strength and ${}_jM_u$, ${}_bM_p$ are nominal strength. The Strength relationships can be expressed as follows:

$$\xi \frac{\beta_b}{\beta_j} {}_b M_p \leq {}_j M_u \quad \rightarrow \alpha\, {}_b M_p \leq {}_j M_u \qquad \therefore \alpha = \xi \frac{\beta_b}{\beta_j} = \xi \cdot \beta \tag{2-7}$$

Coefficients β_j will consider the actual maximum strength of the beam joint and it can be express as follow:

$$\beta_j = \frac{\sigma_u}{F_u} \tag{2-8}$$

where, is σ_u ultimate strength, F_u is the specified minimum ultimate strength of the beam.

Coefficients β_b will consider the actual full-plastic moment of the beam and it can be express as follow:

$$\beta_b = \frac{\sigma_y}{F_y} \tag{2-9}$$

where, is σ_y yield stress, F_y is the specified minimum yield stress of the beam.

As the result, strength randomness β can be determined as follow:

$$\beta = \frac{\beta_b}{\beta_j} = \frac{\sigma_y}{F_y}\frac{F_u}{\sigma_u} = \frac{\sigma_y}{\sigma_u}\frac{F_u}{F_y} = \frac{YR}{F_y / F_u} \tag{2-10}$$

where, YR is the yield ratio.

Denominator is the Ratio between the minimum specified values. Minimum specified stresses of the steel are shown in Table C2.1. For 400 grade steel is F_y / F_u =0.588, and for 490 grade steel is F_y / F_u =0.663. In reality, yield ratio (YR) are greater than specified values. Therefore, coefficient for strength randomness β should be greater than 1.0 and shall be considered in the design. Table C2.2 shows the statistical data of each steel grade.

Table C2.1 Minimum specified yield stress and ultimate strength

Steel Grade	F_y (N/mm2)	F_u (N/mm2)
SS400, SN400B	235	400
SM490, SN490B	325	490

Table C2.2 Statistical data of yield ratio (sample)

Steel Grade	$YR = \sigma_y / \sigma_u$		β=(Avg.+SD)/(F_y/F_u)
	Avg.	*SD*	
SS400 (6 < $t \leq$ 40mm)	0.665	0.0651	1.15
SM490 ($t \leq$ 40mm)	0.694	0.0473	1.10
SN400B (12≤ $t \leq$ 40mm)	0.67	0.028	1.19
SN490B (12≤ $t \leq$ 40mm)	0.71	0.021	1.11

2.1.4. Beam Joint Coefficient α Without Specific Consideration

In regular portal frame office buildings, plastification ratio expected in the beams during significant seismic event are at least 4 to 5. It is recommended to consider this amount of plastic deformation. Consequently, as shown in

Figure C2.3, hardening effect shall be taken ξ= 1.20. Based on the steel grade used in the beam, strength randomness β can be evaluated from previous section.

For example, SS400 grade steel should have α = 1.20×1.15≈1.38. As the result rounded value 1.40 is tabulated in Table 1. SN grade steels are new structural steel specified in Japanese Industrial Standard (JIS). These steels are focused to use in building structures; strength randomness are controlled during manufacturing. As shown in Table C2, Standard Deviation (SD) of SN grade steels are less than similar steel grades (SS400, and SM490).

2.2. Design of Welded Beam Joint

Following requirements shall be satisfied when beam flanges at the joint are welded and designed as rigid joint. Otherwise, strength and stiffness of the joint should be evaluated properly according to the load pass.

i) Beam flanges shall be welded by Complete Joint Penetration (CJP); strength of the welded joint should be greater than or equal to the connected flange design resistance.

ii) If needed, continuity plates (in Japan so called stiffener or diaphragm) should be installed in the column and located in the same plane of connected beam flanges. Steel grade and thickness of the continuity plate should be greater than or equal to the connected beam flange.

iii) Shear force in the beam shall be transmitted by beam web joint, beam web joint should be designed to satisfy section 2.4.2. Beam joint can be either bolted or welded.

2.3. Initial Stiffness

Beam joint shall be rigid when continuity plates shown in section 2.2 is satisfied. H-section column without continuity plates (so called stiffeners in Japan) or rectangular hollow section column without continuity plates (so called diaphragms in Japan) should consider the local out-of-plane deformation of the column flange.

2.4. Strength of Beam Joint

2.4.1. Yield Strength of Beam Joint

i) Yield strength ${}_jM_y$ of the beam joint where continuity plates are installed in the column (i.e. rigid joint) should be determined as follows:

$$ {}_jM_y = {}_jZ_e \cdot F_{fy} \tag{2-11}$$

where:

${}_jZ_e$ is the effective section modulus of beam joint;

for H-section column: $${}_jZ_e = \frac{2}{D_b} I_e \tag{2-12.a}$$

for rectangular or circular hollow section: $${}_jZ_e = \frac{2}{D_b}\left\{ I_e - \frac{1}{12} t_{bw}(d_w - 2h_m)^2 \right\} \tag{2-12.b}$$

I_e is the effective moment of inertia considering the weld access holes. In case bolted joint is used in beam web joint, moment of inertia should be determined from the size of shear plate and beam flanges;

d_w is the clear depth of beam web (=D_b-2t_{bf});

D_b is the beam depth;

t_{bf} is the beam flange thickness;

t_{bw} is the beam web thickness;

h_m is the effective region of the beam web;

if $h_m < S_r$ then $h_m = S_r$;

for rectangular hollow section: $$h_m = \frac{b_j}{\sqrt{\dfrac{b_j \cdot t_{wb} \cdot F_{wy}}{t_{cf}^2 \cdot F_{cy}} - 4}} \tag{2-13.a}$$

if $h_m > \dfrac{d_j}{2}$ or $\dfrac{b_j \cdot t_{wb} \cdot F_{wy}}{t_{cf}^2 \cdot F_{cy}} \le 4$ then $h_m = \dfrac{d_j}{2}$

for circular hollow section: $$h_m = \frac{b_j}{\sqrt{\dfrac{k_1}{2}}\sqrt{k_2\sqrt{\dfrac{3k_1}{2}} - 4}} \tag{2-13.b}$$

if $h_m > \dfrac{d_j}{2}$ or $k_2\sqrt{\dfrac{3k_1}{2}} \le 4$ then $h_m = \dfrac{d_j}{2}$

F_{fy} is the specified minimum yield stress of beam flange;

F_{wy} is the specified minimum yield stress of beam web or shear plate;

F_{cy} is the specified minimum yield stress of column;

S_r is the size of weld access hole perpendicular to the beam axis (see Figure 2.1);

b_j is the width of mechanism region of panel zone (see Figure 2.1);

rectangular hollow section: $b_j = B_c - 2t_{cf}$

for circular hollow section: $b_j = B_c - t_{cf}$

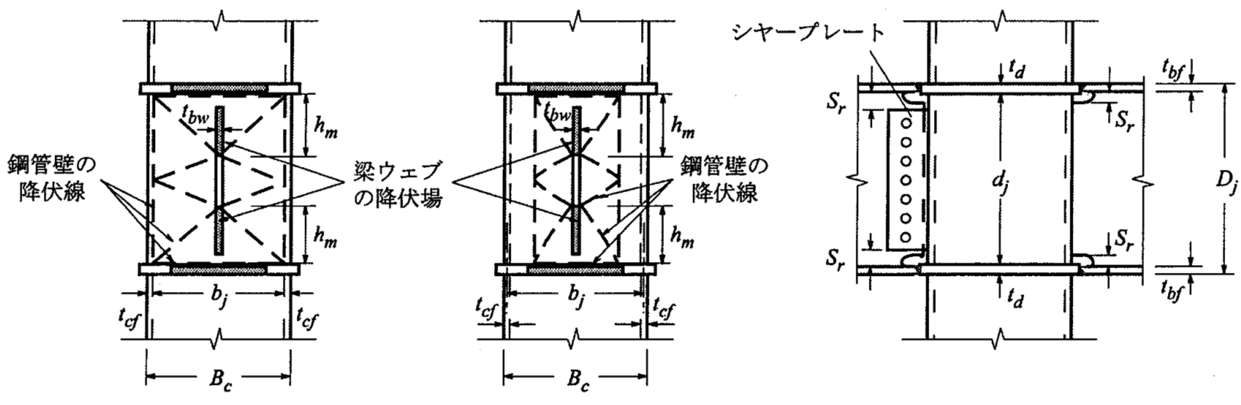
B_c is the hollow section column width;

t_{df} is the column flange thickness;

d_j is the depth of mechanism region of panel zone (see Figure 2.1);

k_1, k_2 is the strength coefficient for circular hollow section.

$$k_1 = \frac{b_j}{t_{cf}}, \quad k_2 = \frac{t_w \cdot F_{wy}}{t_{cf} \cdot F_{cy}} \tag{2-14.a,b}$$



(a) Rectangular Hollow Section (b) Circular Hollow Section (c) Elevation of Connection

Figure2.1 Symbols used in beam joint (yield line mechanism)

ii) Column without continuity plates should consider the local out-of-plane deformation of the column flange to determine the yield strength of the beam joint.

2.4.2. Maximum Strength of Beam Joint

Maximum strength of the beam joint, where it is used in ULS evaluation, shall be determined as follows:

$$ {}_jM_u = {}_jM_{fu} + {}_jM_{wu} \tag{2-15}$$

where:

${}_jM_{fu}$ is the maximum bending strength of beam flange joint;

${}_jM_{wu}$ is the maximum bending strength of beam web joint.

i) Maximum Bending Strength of Beam Flange Joint

a) Maximum bending strength of beam flange joint ${}_jM_{fu}$, where continuity plates are installed in the column (i.e. rigid joint), should be determined as follows:

$$ {}_jM_{fu} = A_f \cdot d_b \cdot F_{fu} \tag{2-16}$$

where:

A_f is the area of one side beam flange;

d_b is the distance between beam flange centroids;

F_{fu} is the specified minimum ultimate strength of beam flange;

b) Column without continuity plates should consider the local out-of-plane deformation of the column flange. Details are shown in the notes.

ii) Maximum Bending Strength of Beam Web Joint

a) Maximum bending strength of beam web joint ${}_jM_{wu}$, where continuity plates are installed in the column (i.e. rigid joint) or for H-section column, should be determined as follows:

$$ {}_jM_{wu} = m \cdot Z_{wpe} \cdot F_{wy} \tag{2-17}$$

where:

Z_{wpe} is the effective section modulus of beam web;

$$ Z_{wpe} = \frac{1}{4}(D_b - 2t_{bf} - 2S_r)^2 t_{bw} \tag{2-18}$$

m is the strength coefficient (see Figure C2.4 for symbols);

for H section column: $m = 1.0$ (2-19.a)

for rectangular hollow section: $m = \min\left\{1,\ 4\frac{t_{cf}}{d_j}\sqrt{\frac{b_j \cdot F_{cy}}{t_{bw} \cdot F_{wy}}}\right\}$ (2-19.b)

for circular hollow section: $m = \min\left\{1,\ \frac{8}{\sqrt{3}k_1 \cdot k_2 \cdot r}\left(\sqrt{k_2\sqrt{\frac{3k_1}{2}} - 4} + r\sqrt{\frac{k_1}{2}}\right)\right\}$ (2-19.c)

r is the aspect ratio of mechanism region of panel zone (see Figure 2.1 for symbols).

$$ r = \frac{d_j}{b_j} \tag{2-20}$$

b) Maximum bending strength of beam web joint that are connected by high-strength bolts should be greater than or equal to the capacity (strength) computed from Eq. (2-17).

c) Beam joint where continuity plates are not installed in the column should not count on the bending resistance of the beam web joint.

2.4.3. Shear Strength of Beam Joint

Shear force in the beam shall be transmitted to the column through beam web joint. Beam web joint should be design to have sufficient strength to support the demand. If the beam web joint is considered to resist bending moment based on Eq. (3-7), the web joint should be design under expected shear and bending moment.

Beam web joint should be either welded joint or high-strength bolted joint.

i) Maximum strength of welded joint

Either the beam or shear plate is welded to the column, welded joint should be double sided fillet welds. The maximum strength of the welded joint should be greater or equal than the demand. Maximum strength of the welded joint can be determined in Chapter X.

ii) Maximum strength of high-strength bolted joint

Maximum strength of the high-strength bolted joint, where the beam web is connected to the shear plate, should be greater or equal than the demand. Maximum strength of the bolted joint can be determined in Chapter XX.

For hollow section column, it is permitted to determine the strength of the beam web high-strength bolted joint by following procedure (see Figure 2.2).

a) Shear force in the beam will be carried by the bolts locating near to the natural axis of the beam.

b) Outer sided bolts will be used to resist the bending moment.

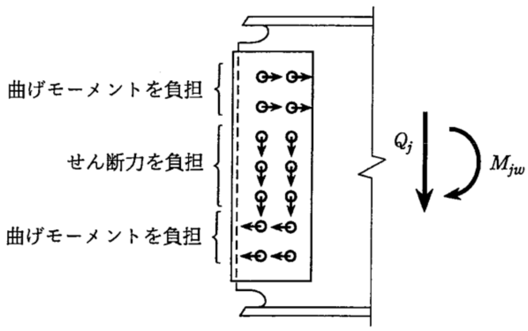


Figure 2.2 Force distribution in the beam web joint (high-strength bolted joint)

COMMENTARY

2.5. Beam-to-column connection with H-section column

2.5.1. Beam joint without continuity plates

Following subclause shows the formula to compute the strength of joints corresponding to the failure mode (see Figure C2.4); however, formulae that can compute the of the stiffness are not included. (Should be similar to component method.)

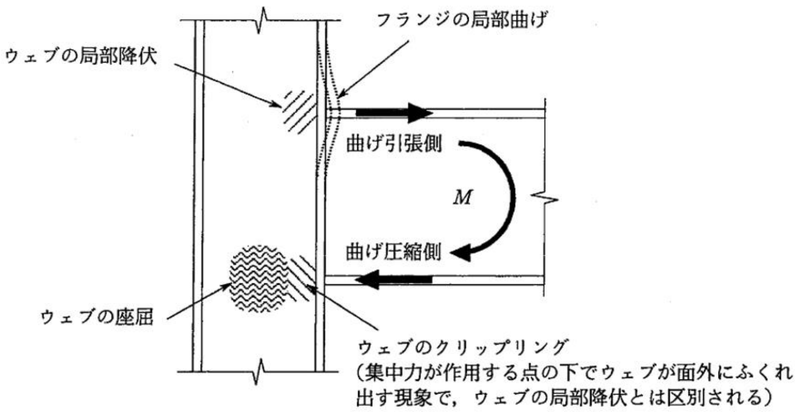


Figure C2.4 Local failure at the beam joint where continuity plates are not installed

2.5.1.1. Local bending of H-section column flange

2.5.1.2. Local yielding of H-section column web

2.5.1.3. Crippling of H-section column web

2.5.1.4. Local buckling of H-section column web

2.5.2. Beam joint with continuity plates

If the strength of the continuity plate is not satisfying the demand, it should be considered in the design.

2.6. Beam-to-column connection with external diaphragms (for hollow section columns)

Square or circular hollow section column is not sliced to assemble panel zone, which is typical in Japanese detail, external diaphragm is welded to the column for beam joint (see Figure 2.3).

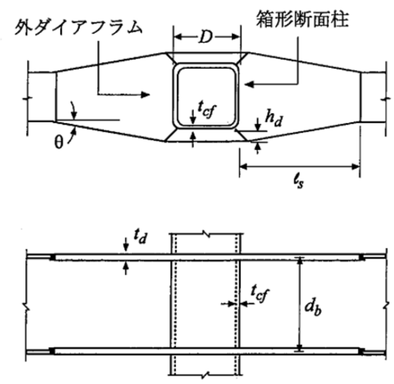


Figure 2.3 External Diaphragms for beam-to-column connection

2.6.1. Yield Strength of Beam Joint

2.6.2. Maximum Strength of Beam Joint

COMMENTARY

Due to the contribution of local deformation from the column plate, it cannot be assumed to be rigid joint in some cases. In such case, the stiffness of the beam joint should be considered in the design.

i) Contribution of the local deformation at the joint should be evaluated. If the amount of local deformation is negligible, it able to assume the joint as rigid.

ii) If the local deformation is not negligible, the joint should be assumed to be semi-rigid. Stiffness of the joint should be modeled as rotational spring in the computational structural model.

2.7. Beam-to-column connection without continuity plates (for hollow section columns)

Formulae that compute the joint strength and stiffness are shown. Stiffness of the joint should include the local deformation from the column flange. Structural model which is used for the analysis should model the joint as a rotational spring.

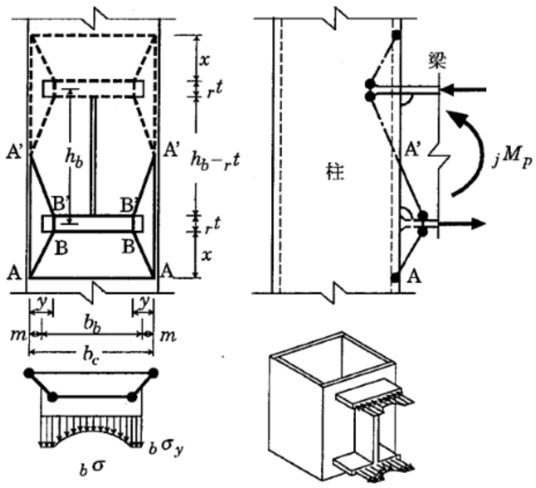
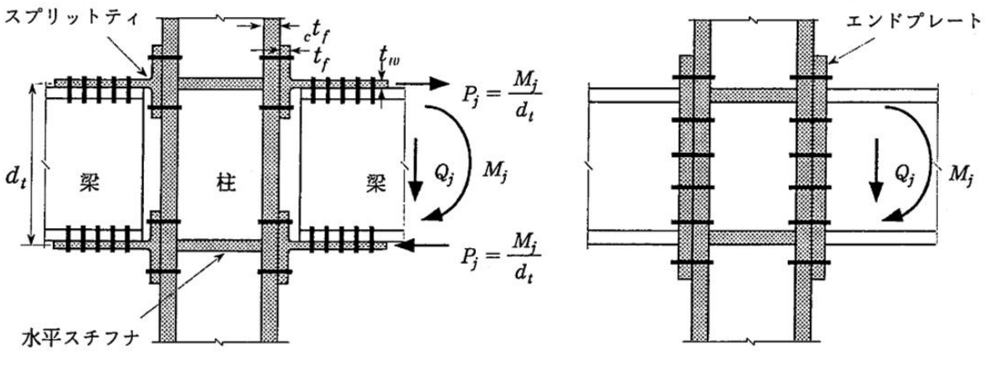


Figure C2.5 Failure mechanism at the column flange

2.8. Beam-to-column connection with high-strength bolts (for H-section columns)

Beam joint with continuity plates are stipulated in this clause. Typical detail are shown in Figure C2.4.



(a) T-stub type (b) End plate type

Figure 2.4 Beam joints by high-strength bolts

2.8.1. Yield Strength of Beam Joint

i) Yield bending strength

ii) Yield shear strength

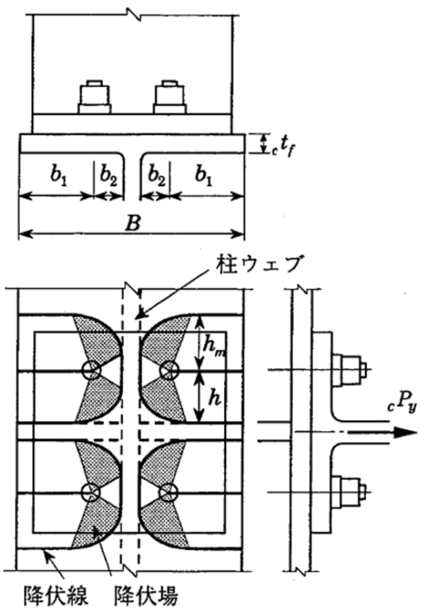


Figure 2.5 Failure mechanism at H-column flange

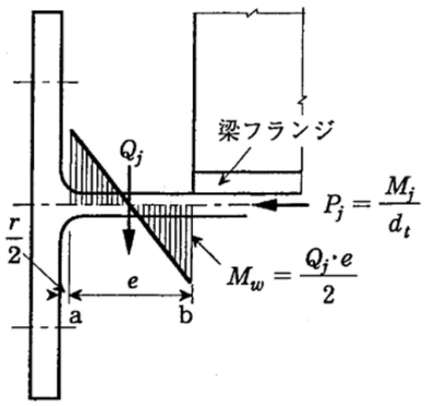


Figure 2.6 Force subject to T-stub

2.8.2. Maximum Strength of Beam Joint

i) Maximum bending strength

ii) Maximum shear strength

2.8.3. Beam joint initial stiffness

T-stub beam joint can be assumed to be rigid when the local deformation at the T-stub is negligible. T-stub failure modes 1 and 2 can be assumed rigid connection. Local deformation mode 3 is not negligible and should be considered (see Figure C2.6 for failure mode definition).

COMMENTARY

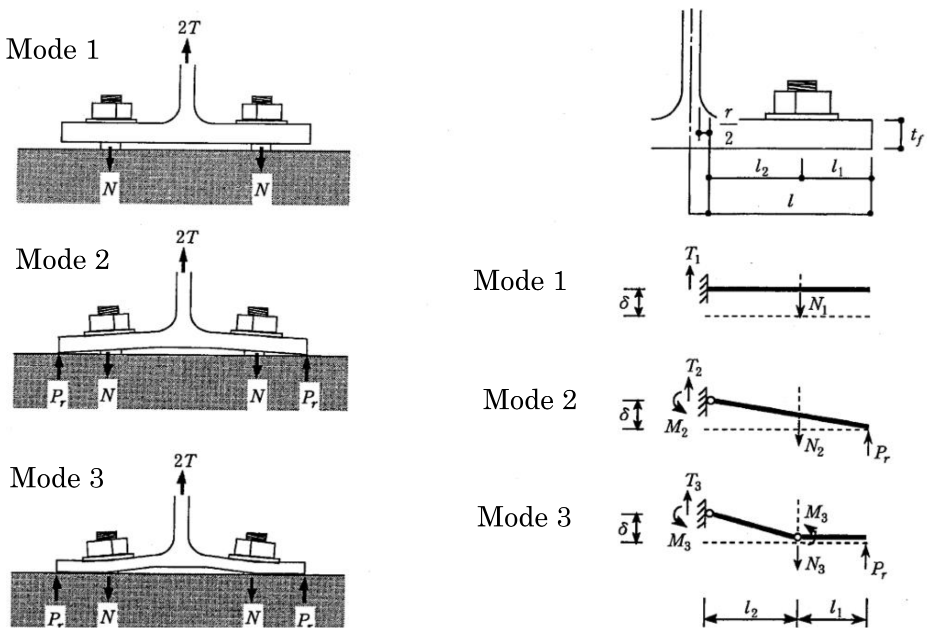


Figure C2.6 Local deformation and Failure modes at T-stub

3. Recommendation for Limit State Design of Steel Structure (LRFD, AIJ)

3.1. Classification of Structure

Building structure can be classified in four structural classes as shown in Table 3.1. Classification shown in this table can be used in ULS. Class S-I-1 should be only applied for seismic design. Cross-section, beam, and column classes are shown in subsequent sections.

Table 3.1 Structural Class

Structural Class	Structural Components Requirements
S-I-1	Cross-section: class P-I-1, Beam class: L-I, Column class: C-I Joints should fulfill Section 2
S-I-2	Cross-section: class P-I-2, Beam class: L-I, Column class: C-I Joints should fulfill Section 2
S-II	Cross-section: class P-II, Beam class: L-II, Column class: C-II Joints should fulfill Section 2
S-III	Cross-section: class P-III, Beam class: L-III, Column class: C-III

COMMENTARY

In ULS, the design procedure and structural class can be summarized in Table C3.1. Plastic design (analysis) should be used in class S-I, elastic design (analysis) should be used in class S-II and S-III. Elastic design means allowable stress design is only allowed in these classes.

Table C3.1 Design Matrix

<table>
<tr><th colspan="2" rowspan="2">Structural Class</th><th colspan="4">Class of Cross-section</th></tr>
<tr><th>P-I-1</th><th>P-I-2</th><th>P-II</th><th>P-III</th></tr>
<tr><td rowspan="3">Class of Beam & Column</td><td>L-I
C-I</td><td>S-I-1 <J>
[plastic design]</td><td>S-I-2 <J>
[plastic design]</td><td rowspan="2">S-II <J>
[elastic design]</td><td rowspan="3">S-III
[elastic design]</td></tr>
<tr><td>L-II
C-II</td><td colspan="2"></td></tr>
<tr><td>L-III
C-III</td><td colspan="3"></td></tr>
</table>

<J> means the joints should fulfill Section 2.

3.2. Classification of Structural Components

3.2.1. Classification of cross sections

Five classes of cross-sections are defined as follows. Table 3.2 is for beam classification, and Tables 3.3, 3.4, and 3.5 are for column classification.

- Class P-I-1 cross-sections are those which can form a plastic hinge with the plastic rotation capacity greater than or equal to 4.0
- Class P-I-2 cross-sections are those which can form a plastic hinge with the plastic rotation capacity greater than or equal to 2.0 but less than 4.0.

- Class P-II cross-sections are those which can develop their plastic moment resistance, but not having rotation capacity due to local bucking.
- Class P-III cross-sections are those in which the stress in the extreme compression fiber of the steel member assuming an elastic distribution of stress can reach the yield strength, but local buckling is liable to prevent development of the plastic moment resistance.
- Class P-IV cross-sections not included in above classes are those in which local buckling will occur before the attainment of yield stress in one or more parts of the cross section.

Table 3.2 Classification for wide flange Beam

Class	SN400	SN490
P-I-1	$\dfrac{(b/t_f)^2}{(0.49\sqrt{E/F_y})^2}+\dfrac{(d/t_w)^2}{(3.2\sqrt{E/F_y})^2}\le 1$	$\dfrac{(b/t_f)^2}{(0.57\sqrt{E/F_y})^2}+\dfrac{(d/t_w)^2}{(2.6\sqrt{E/F_y})^2}\le 1$
	but $d/t_w \le 2.2\sqrt{E/F_y}$	
P-I-2	$\dfrac{(b/t_f)^2}{(0.60\sqrt{E/F_y})^2}+\dfrac{(d/t_w)^2}{(3.8\sqrt{E/F_y})^2}\le 1$	$\dfrac{(b/t_f)^2}{(0.73\sqrt{E/F_y})^2}+\dfrac{(d/t_w)^2}{(3.2\sqrt{E/F_y})^2}\le 1$
	but $d/t_w \le 2.2\sqrt{E/F_y}$	
P-II	$\dfrac{(b/t_f)^2}{(0.71\sqrt{E/F_y})^2}+\dfrac{(d/t_w)^2}{(4.6\sqrt{E/F_y})^2}\le 1$	$\dfrac{(b/t_f)^2}{(0.88\sqrt{E/F_y})^2}+\dfrac{(d/t_w)^2}{(3.9\sqrt{E/F_y})^2}\le 1$
	but $d/t_w \le 2.4\sqrt{E/F_y}$	
P-III	$0.38\sqrt{E/F_y} < b/t_f \le 0.82\sqrt{E/F_y}$ $2.4\sqrt{E/F_y} < d/t_w \le 6.0\sqrt{E/F_y}$	

Table 3.3 Classification for wide flange Column

Class	SN400	SN490
P-I-1	$\dfrac{(b/t_f)^2}{(0.49\sqrt{E/F_y})^2}+\dfrac{(d_e/t_w)^2}{(1.5\sqrt{E/F_y})^2}\le 1$	$\dfrac{(b/t_f)^2}{(0.57\sqrt{E/F_y})^2}+\dfrac{(d_e/t_w)^2}{(1.4\sqrt{E/F_y})^2}\le 1$
	but $d/t_w \le 1.5\sqrt{E/F_y}$	

P-I-2	$\frac{(b/t_f)^2}{(0.60\sqrt{E/F_y})^2}+\frac{(d_e/t_w)^2}{(1.9\sqrt{E/F_y})^2}\le 1$	$\frac{(b/t_f)^2}{(0.73\sqrt{E/F_y})^2}+\frac{(d_e/t_w)^2}{(1.7\sqrt{E/F_y})^2}\le 1$
	but $d/t_w \le 1.5\sqrt{E/F_y}$	
P-II	$\frac{(b/t_f)^2}{(0.71\sqrt{E/F_y})^2}+\frac{(d_e/t_w)^2}{(2.3\sqrt{E/F_y})^2}\le 1$	$\frac{(b/t_f)^2}{(0.88\sqrt{E/F_y})^2}+\frac{(d_e/t_w)^2}{(2.1\sqrt{E/F_y})^2}\le 1$
	but $d/t_w \le 1.6\sqrt{E/F_y}$	
P-III	$0.38\sqrt{E/F_y} < b/t_f \le 0.82\sqrt{E/F_y}$ $1.6\sqrt{E/F_y} < d/t_w \le 3.5\sqrt{E/F_y}$	

where $d_e=\frac{1}{2t_w}\left[2\frac{N_{Ed}}{N_y}A_f+\left(1+\frac{N_{Ed}}{N_y}\right)A_w\right]$, but when $\frac{A_w}{2A_f+A_w}\le\frac{N_{Ed}}{N_y}<1 \rightarrow d=d_e$, $A_f=2\cdot b\cdot t_f$, $A_w=d\cdot t_w$

Table 3.4 Classification for Square Hollow Section Column (SN400, SN490)

Class	Hot Finished	Cold Formed
P-I-1	$\frac{B}{t}\le 1.1\sqrt{\frac{E}{F_y}}$	$\frac{B}{t}\le 24$
P-I-2	$\frac{B}{t}\le 1.3\sqrt{\frac{E}{F_y}}$	$\frac{B}{t}\le 28$
P-II	$\frac{B}{t}\le 1.6\sqrt{\frac{E}{F_y}}$	$\frac{B}{t}\le 36$
P-III	$1.6\sqrt{E/F_y} < B/t \le 1.8\sqrt{E/F_y}$	$\frac{B}{t}\le 36$

Table 3.5 Classification for Circular Hollow Section Column (400, 490 grade steel)

Class	Hot Finished	Cold Formed
P-I-1	-	$\frac{D}{t}\le 36$
P-I-2	-	$\frac{D}{t}\le 54$
P-II	-	$\frac{D}{t}\le 90$
P-III	-	$\frac{D}{t}\le 90$

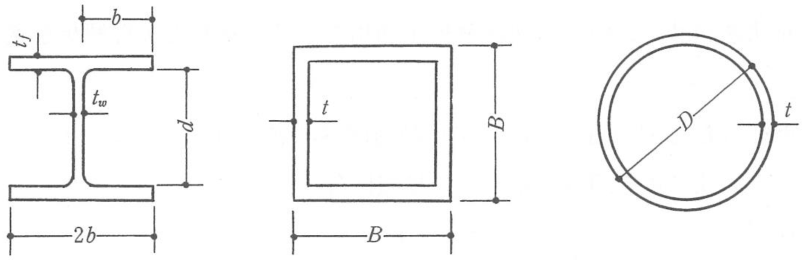


Figure 3.1 Geometrical symbols defined for cross-sections

COMMENTARY

Figure C3.1 shows the schematic relationships between resistance and cross-section classification. Due to imperfections, elastic buckling will be limited at point D. The outer most fiber that reached to yield stress in average are represented by point C. The range between point C to point B can develop certain amount of plastification in the section, and it is classified as P-II. However, in this class, cross-section cannot expect plastic rotation. Plastic rotation can be expected when the compactness of the element is smaller than point B where full-plastic resistance is reached. Point A is the compactness where can guaranty plastic rotation capacity greater than or equal to 4.0.

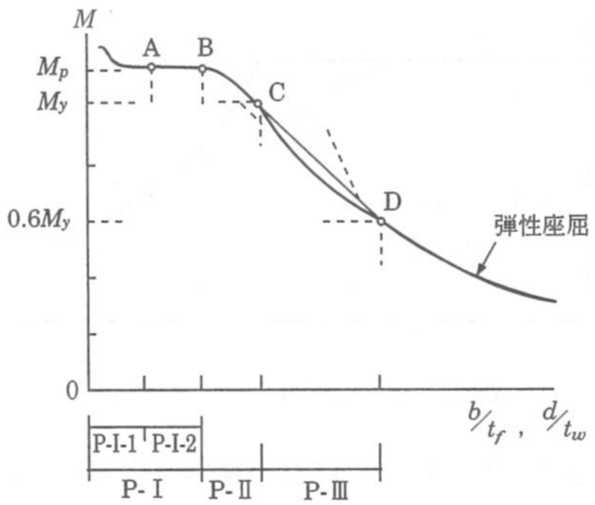


Figure C3.1 Classification of Cross Section

3.2.2. Classification of Beam

Three classes of beams are defined as follows. The beam portion which is expected to reach full plastic moment should be either L-I or L-II depending of the demand of rotational capacity, i.e. plastification level.

Table 3.6 Classification of Beam

Beam Class	Beam non-deimetional Slenderness Ratio	Plastic Deformation Capacity
L-I	$\lambda_b \leq 0.75\ {}_p\lambda_b$	$R \geq 6$
L-II	$0.75{}_p\lambda_b < \lambda_b \leq\ {}_p\lambda_b$	$2 \leq R < 6$
L-III	${}_p\lambda_b < \lambda_b$	-

where $R = \theta_u/\theta_p$ -1, where θ_u is the maximum rotation, θ_p is the elastic rotation corresponding to M_p., λ_b is non-dimensional lateral buckling slenderness ratio that is defined as Eq. (3-1), ${}_p\lambda_b$ is plastic limit slenderness that is considering the moment distribution as defined by Eq. (3-2), M_p is full plastic moment of the beam, Z_p is plastic section modulus, F_y is the minimum specified yield stress (nominal yield stress).

$$\lambda_b = \sqrt{\frac{M_P}{M_e}} = \sqrt{\frac{Z_p \cdot F_y}{M_e}} \tag{3-1}$$

$${}_p\lambda_b = 0.6 + 0.3\frac{M_2}{M_1} \tag{3-2}$$

where M_1, M_2 are end moments coincides to the laterally brace location. $|M_1| > |M_2|$; M_2/ M_1 will be positive when double curvature bending is subjected to the considered portion of the beam (double curvature bending will be negative in EN1993-1-1), and M_e is elastic buckling resistance for wide-flange beam which shall be determined from following equation:

$$M_e = C_b\sqrt{\frac{\pi^4 EI_Y \cdot EI_W}{{}_k l_b^4} + \frac{\pi^2 EI_Y \cdot GJ}{l_b^2}} \quad \text{(for wide flange beam)} \tag{3-3}$$

COMMENTARY

Figure C3.2 shows the schematic relationships between resistance and beam classification. Where the non-dimensional lateral buckling slenderness ratio, λ_b, is less than or equal to plastic limit slenderness, ${}_p\lambda_b$, can develop full-plastic moment. Small value than ${}_p\lambda_b$, are classified as L-1 and L-II.

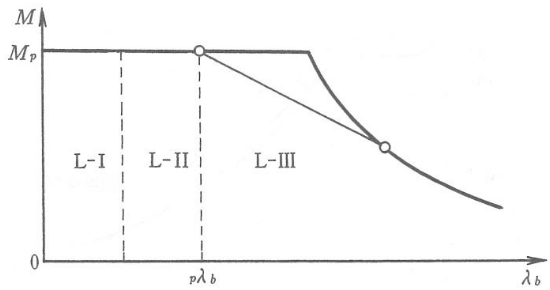


Figure C3.2 Classification of Beam

3.2.3. Classification of Column

Three classes of columns are defined as follows.

- **Class C-I** columns are those which fulfill the followings;

<Compressive Axial Force>

$$\frac{N_{Ed}}{N_y} \le 0.75 \tag{3-4}$$

<Slenderness Ratio>

$$\lambda_b \le 0.75\,{}_p\lambda_b \quad \text{only for strong axis bending on wide-flange column} \tag{3-5}$$

<Combination of Compressive axis force with bending moment>

$$\frac{N_{Ed}}{N_y} \cdot {}_f\lambda_c^2 \le 0.25 \tag{3-6}$$

where λ_b, ${}_p\lambda_b$ are determined from Eqs. (3-1), (3-2), ${}_f\lambda_c$ is determined from Eq. (3-1) which is the value of considered plane.

Moreover, the column which class is C-1 and is expected to form plastic hinge should fulfill following;

$-0.5 < M_2/M_1 \leq 1.0$

$$\frac{N_{Ed}}{N_y}\cdot\lambda_{c0}^2 \leq 0.10\left(1+\frac{M_2}{M_1}\right) \tag{3-7}$$

$-1.0 \leq M_2/M_1 \leq -0.5$

$$\frac{N_{Ed}}{N_y}\cdot\lambda_{c0}^2 \leq 0.05 \tag{3-8}$$

where λ_{c0} is non-dimensional flexural slenderness ratio that is defined as Eq. (3-9).

$$\lambda_c = \sqrt{\frac{N_y}{N_e}} = \sqrt{\frac{A\cdot F_y}{N_e}} \tag{3-9}$$

where N_0 is the Euler's buckling resistance computed from member length as follow:

$$N_0 = \frac{\pi EI}{(l_c)^2} \tag{3-10}$$

- **Class C-II** columns are those which fulfill the followings and not included in class C-I;

$$\frac{N_{Ed}}{N_{cr}} \leq 0.75 \tag{3-11}$$

$$\lambda_b \leq {}_p\lambda_b \qquad \text{only for strong axis bending on wide-flange column} \tag{3-12}$$

where N_{cr} is design buckling resistance.

- **Class C-III** columns are those which fulfill Eq. (3-11) and not included in class C-I and C-II.

For hollow sections, class C-III column do not exist.

3.3. Resistance of Members

3.3.1. Uniform Member in Compression

The design resistance of the compression member is the smaller value of flexural buckling resistance or local buckling resistance. The design resistance of the compression member, $N_{b,Rd}$, shall be determined from following equation. ϕ_c is the resistance factor.

$$N_{b,Rd} = \phi_c \cdot N_{cr} \tag{3-13}$$

3.3.1.1. Nominal Resistance of Flexural Buckling

Nominal flexural buckling resistance N_{cr} should be taken as:

$$\lambda_c \leq {}_p\lambda_c \qquad N_{cr} = N_y \tag{3-14.a}$$

$${}_p\lambda_c < \lambda_c \leq {}_e\lambda_c \qquad N_{cr} = \left(1.0 - 0.5\frac{\lambda_c - {}_p\lambda_c}{{}_e\lambda_c - {}_p\lambda_c}\right)N_y \tag{3-14.b}$$

$$\lambda_c > {}_e\lambda_c \qquad N_{cr} = \left(\frac{1}{1.2\cdot\lambda_c^2}\right)N_y \tag{3-14.c}$$

where λ_c is non-dimensional flexural slenderness ratio that is defined as Eq. (5-3), ${}_p\lambda_c$ is plastic limit slenderness (=0.15), ${}_e\lambda_c$ is elastic limit slenderness ($=1/\sqrt{0.6}$), N_y is the yield strength (=$A\cdot F_y$), F_y is the minimum specified yield stress (nominal yield stress).

$$\lambda_c = \sqrt{\frac{N_y}{N_e}} = \sqrt{\frac{A\cdot F_y}{N_e}} \tag{3-15}$$

where N_e is the elastic buckling resistance computed as follow:

$$N_e = \frac{\pi EI}{(k_c \cdot l_c)^2} \tag{3-16}$$

where k_c is effective buckling length factor, l_c is member length.

3.3.1.2. Nominal Resistance of Local Buckling

Nominal resistance of local buckling N_{cr} should be taken as:

1) Cross-section class: P-I and P-II

$$N_{cr} = N_y \tag{3-17}$$

2) Cross-section class: P-III

$$N_{cr} = F_{cr} \cdot A \tag{3-18}$$

a) Wide-flange section

$$F_{cr} = \min\left[{}_f F_{cr},\ {}_w F_{cr}\right] \tag{3-19}$$

where,

$${}_f F_{cr} = \left(670 - 453\frac{b}{t_f}\sqrt{\frac{F_y}{E}}\right) \cdot 410\frac{F_y}{E} \qquad 0.38\sqrt{\frac{E}{F_y}} < \frac{b}{t_f} \leq 0.82\sqrt{\frac{E}{F_y}} \tag{3-20.a}$$

$${}_w F_{cr} = \left(1720 - 453\frac{d}{t_w}\sqrt{\frac{F_y}{E}}\right) \cdot 201\frac{F_y}{E} \qquad 1.55\sqrt{\frac{E}{F_y}} < \frac{d}{t_w} \leq 2.45\sqrt{\frac{E}{F_y}} \tag{3-20.b}$$

b) Hot finished and Square-welded Hollow section

$$F_{cr} = \left(1670 - 453\frac{B}{t}\sqrt{\frac{F_y}{E}}\right) \cdot 218\frac{F_y}{E} \qquad 1.61\sqrt{\frac{E}{F_y}} < \frac{B}{t} \leq 2.45\sqrt{\frac{E}{F_y}} \tag{3-21}$$

c) Cold-formed hollow section (for square $B/t \leq 36$, for Circular $D/t \leq 90$)

$$F_{cr} = F_y \tag{3-22}$$

where t_w is the web thickness, and t_f is the flange thickness.

3.3.2. Uniform Member is Bending

The design resistance of the bending member is the smaller value of lateral torsional buckling resistance or local buckling resistance. The design resistance of the bending member, $M_{b,Rd}$, shall be determined from following equation. ϕ_b is the resistance factor.

$$M_{b,Rd} = \phi_b \cdot M_{cr} \tag{3-23}$$

3.3.2.1. Nominal Resistance of Lateral Torsional Buckling

Nominal buckling strength N_{cr} should be taken as:

$$\lambda_b \leq {}_p\lambda_b \qquad M_{cr} = M_p \tag{3-24.a}$$

$${}_p\lambda_b < \lambda_b \leq {}_e\lambda_b \qquad M_{cr} = \left(1.0 - 0.4\frac{\lambda_b - {}_p\lambda_b}{{}_e\lambda_b - {}_p\lambda_b}\right)M_p \tag{3-24.b}$$

$$\lambda_c > {}_e\lambda_b \qquad N_{cr} = \frac{1}{\lambda_b^2}M_p \tag{3-24.c}$$

where λ_b is non-dimensional lateral buckling slenderness ratio that is defined as Eq. (3-25), ${}_p\lambda_b$ is plastic limit slenderness that is considering the moment distribution as defined by Eq. (3-26), ${}_e\lambda_b$ is elastic limit slenderness

($=1/\sqrt{0.6}$), M_p is full plastic moment of the beam, Z_p is plastic section modulus.

$$\lambda_b = \sqrt{\frac{M_P}{M_e}} = \sqrt{\frac{Z_p \cdot F_y}{M_e}} \tag{3-25}$$

$$_p\lambda_b = 0.6 + 0.3\frac{M_2}{M_1} \tag{3-26}$$

where M_e is elastic buckling resistance for wide-flange beam shall be determined from following equation:

$$M_e = C_b\sqrt{\frac{\pi^4 EI_Y \cdot EI_W}{{}_k l_b^4} + \frac{\pi^2 EI_Y \cdot GJ}{l_b^2}} \quad \text{(for wide flange beam)} \tag{3-27}$$

where C_b is moment distribution coefficient that is determined from Eq. (3-28) (Correction factors kc; EN1993-1-1); uniform bending will be 1.0. C_b for non-intermediate loading at considered portion:

$$C_b = 1.75 + 1.05\left(\frac{M_2}{M_1}\right) + 0.3\left(\frac{M_2}{M_1}\right)^2 \le 2.3 \tag{3-28}$$

where M_1, M_2 are beam end moments, and $|M_1| > |M_2|$. M_2/M_1 will be positive when double curvature bending is subjected to the considered portion of the beam (double curvature bending will be negative; EN1993-1-1).

3.3.2.2. Nominal Resistance of Local Buckling

Nominal resistance of local buckling M_{cr} should be taken as:

1) Cross-section class: P-I and P-II

$$M_{cr} = M_p \tag{3-29}$$

2) Cross-section class: P-III

$$M_{cr} = F_{cr} \cdot Z_e \tag{3-30}$$

a) Wide-flange section

$$F_{cr} = \min\left[{}_f F_{cr},\ {}_w F_{cr}\right] \tag{3-31}$$

i) Under strong axis bending

$${}_f F_{cr} = \left(670 - 453\frac{b}{t_f}\sqrt{\frac{F_y}{E}}\right) \cdot 410\frac{F_y}{E} \tag{3-32.a}$$

$${}_w F_{cr} = \left(5190 - 453\frac{d}{t_w}\sqrt{\frac{F_y}{E}}\right) \cdot 50\frac{F_y}{E} \tag{3-32.b}$$

ii) Under weak axis bending

$${}_f F_{cr} = 1.6\left(670 - 453\frac{b}{t_f}\sqrt{\frac{F_y}{E}}\right) \cdot 410\frac{F_y}{E} \le F_y \tag{3-33.a}$$

$${}_w F_{cr} = F_y \tag{3-33.b}$$

b) Square Hollow section

$$F_{cr} = \left(1240 - 453\frac{B}{t}\sqrt{\frac{F_y}{E}}\right) \cdot 621\frac{F_y}{E} \tag{3-34}$$

3.3.3. Uniform Member in Compression with Bending (combined loading)

The design resistance (limit state resistance) of the combined loaded member is the smallest value of full-plastic resistance, flexural resistance, and local resistance. ϕ_p, ϕ_c, ϕ_b are resistance factors.

3.3.3.1. Full Plastic Resistance

1) Wide flange section under strong axis bending and square hollow section

$$\frac{M_{Ed}}{\phi_p \cdot M_p} \leq 1.0 \qquad \frac{N_{Ed}}{\phi_p \cdot N_y} \leq 0.15 \tag{3-35.a}$$

$$\frac{N_{Ed}}{\phi_p \cdot N_y} + 0.85\frac{M_{Ed}}{\phi_p \cdot M_p} \leq 1.0 \qquad \frac{N_{Ed}}{\phi_p \cdot N_y} > 0.15 \tag{3-35.b}$$

2) Wide flange section under weak axis bending

$$\frac{M_{Ed}}{\phi_p \cdot M_p} \leq 1.0 \qquad \frac{N_{Ed}}{\phi_p \cdot N_y} \leq 0.40 \tag{3-35.c}$$

$$\left(\frac{N_{Ed}}{\phi_p \cdot N_y}\right)^2 + 0.84\frac{M_{Ed}}{\phi_p \cdot M_p} \leq 1.0 \qquad \frac{N_{Ed}}{\phi_p \cdot N_y} > 0.40 \tag{3-35.d}$$

3) Circular hollow section

$$\frac{M_{Ed}}{\phi_p \cdot M_p} \leq 1.0 \qquad \frac{N_{Ed}}{\phi_p \cdot N_y} \leq 0.20 \tag{3-35e}$$

$$\frac{N_{Ed}}{\phi_p \cdot N_y} + 0.80\frac{M_{Ed}}{\phi_p \cdot M_p} \leq 1.0 \qquad \frac{N_{Ed}}{\phi_p \cdot N_y} > 0.20 \tag{3-35.f}$$

3.3.3.2. Flexural Resistance

1) Wide flange section under strong axis bending

a) In-plane limit state resistance

In-plane limit state resistance should fulfill following; however, the column which class in C-I and fulfill the requirement to form plastic hinge should fulfill section 3.3.3.1.

$$\frac{N_{Ed}}{\phi_c \cdot N_c} + 0.85\varphi\frac{M_{Ed}}{\phi_p \cdot M_p} \leq 1.0 \tag{3-36}$$

where

$$\varphi = 1.0 \qquad (N_{Ed}/N_y)\cdot\lambda_{c0}^2 \leq 0.25(1+M_2/M_1) \tag{3-37.a}$$

$$\varphi = \frac{1-0.5(1+M_2/M_1)\sqrt{N_{Ed}/N_0}}{1-N_{Ed}/N_0} \geq 1.0 \qquad (N_{Ed}/N_y)\cdot\lambda_{c0}^2 > 0.25(1+M_2/M_1) \tag{3-37.b}$$

b) Out-of-plane limit state resistance

Class C-III columns should also fulfill the out-of-plane limit state resistance.

$$\frac{N_{Ed}}{\phi_c \cdot N_{cY}} + 0.85\frac{M_{Ed}}{\phi_b \cdot M_c} \leq 1.0 \quad \text{but} \quad \frac{M_{Ed}}{\phi_b \cdot M_c} \leq 1.0 \tag{3-38}$$

2) Wide flange section under weak axis bending and hollow sections (i.e. Square and Circular)

Limit state resistance should fulfill 3.3.3.2 1)-a). Moreover, Class C-I columns should fulfil full-plastic resistance (see section 3.3.3.1).

3.3.3.3. Local Resistance

1) Wide flange section

a) Cross-section class: P-I

See section 3.3.3.1. (full plastic resistance)

b) Cross-section class: P-II

$$\frac{N_{Ed}}{\phi_p \cdot N_y} + \frac{M_{Ed}}{\phi_p \cdot M_p} \le 1.0 \tag{3-39}$$

c) Cross-section class: P-III

$$\frac{N_{Ed}}{\phi_c \cdot N_c} + \frac{M_{Ed}}{\phi_b \cdot M_c} \le 1.0 \tag{3-40}$$

where, $N_c = F_{cr} \cdot A$, $M_c = F_{cr} \cdot Z_e$, A is the gross cross section, and Z_e is elastic section modulus. Dcr is determined as follow:

$$F_{cr} = \min\left[{}_f F_{cr},\ {}_w F_{cr}\right] \tag{3-41}$$

$${}_f F_{cr} = \left(670 - 453 \frac{b}{t_f} \sqrt{\frac{F_y}{E}}\right) \cdot 410 \frac{F_y}{E} \tag{3-42.a}$$

$${}_w F_{cr} = \left(2875 - 453 \frac{d}{t_w} \sqrt{\frac{F_y}{E}}\right) \cdot 94.3 \frac{F_y}{E} \tag{3-42.b}$$

2) Square hollow section

a) Cross-section class: P-I

See section 3.3.3.1. (full plastic resistance)

b) Cross-section class: P-II

See section 3.3.3.3 1)-b) Eq.(3-39) (i.e. same as wide flange section)

c) Cross-section class: P-III

See section 3.3.3.3 1)-c) Eq.(3-40) . But Fcr should be determined as follow:

i) Hot finished section

$$F_{cr} = \left(980 - 453 \frac{B}{t} \sqrt{\frac{F_y}{E}}\right) \cdot 820 \frac{F_y}{E} \tag{3-43}$$

ii) Cold formed section

$$F_{cr} = F_y \tag{3-44}$$

3) Circular hollow section

d) Cross-section class: P-I

See section 3.3.3.1. (full plastic resistance)

e) Cross-section class: P-II

See section 3.3.3.3 1)-b) Eq.(3-39) (i.e. same as wide flange section)

f) Cross-section class: P-III

$$\frac{N_{Ed}}{\phi_p \cdot N_y} + \frac{M_{Ed}}{\phi_p \cdot M_y} \le 1.0 \tag{3-45}$$

COMMENTARY