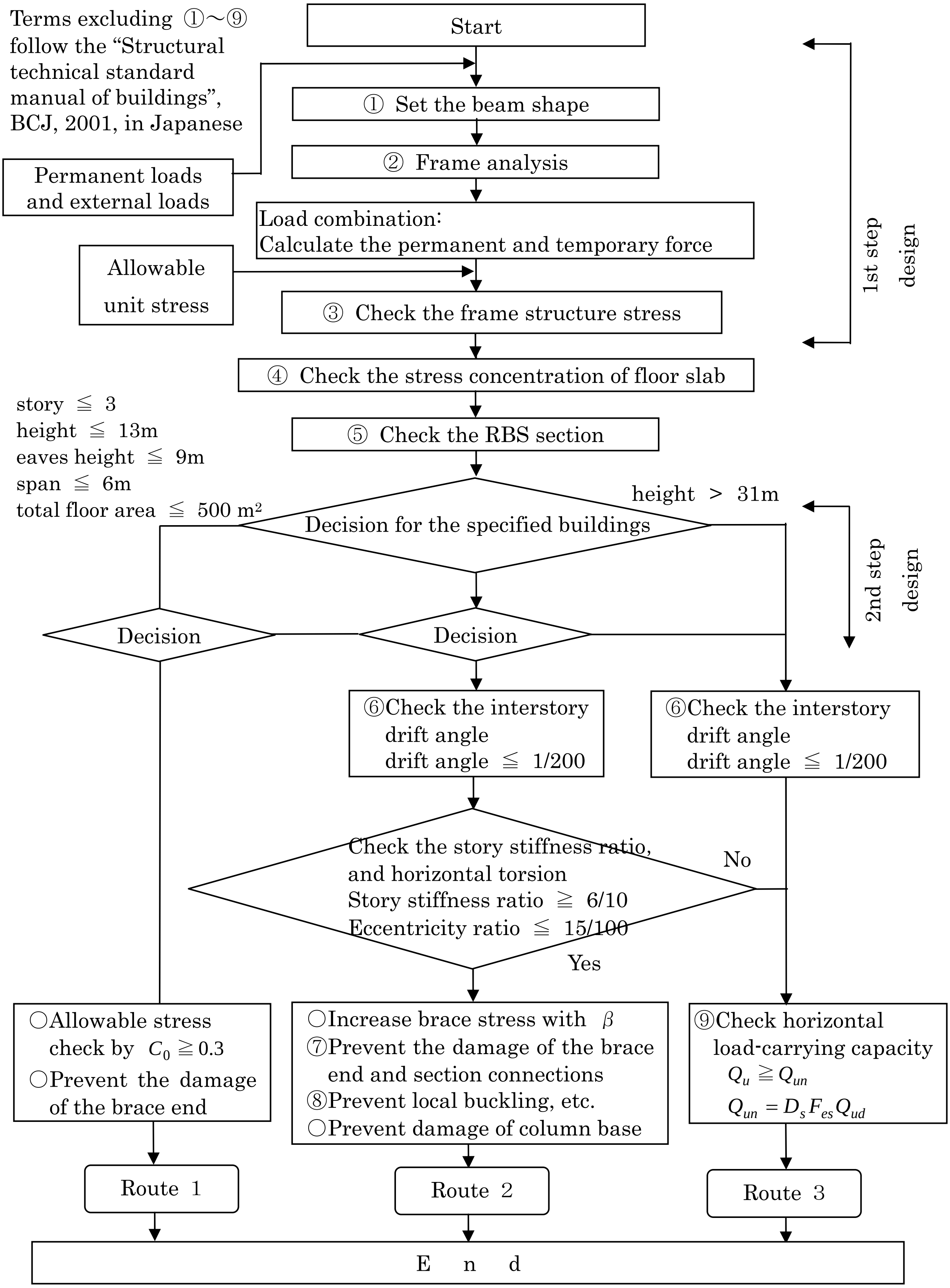
ERBS design procedure



Design flow of ERBS

① Determine beam shape

ERBS shape and size is determined using Table 1~2 and Figure 1~3 by applying experimental and analytical results, and FEMA-350 (Reference 1).

Table 1 Shape and Size of the Beam

type	size		figure
ERBS-H type	$a \cong (0.5\sim0.75)b_f$	$a_1 \geq \max(50,5tf)$	figure 1 H type
	$b \cong (0.65\sim0.85)\,d_b$	$c \cong (0.2b_f \leq c \leq 0.25b_f)$	
	$R = (4c \cdot c + b \cdot b)/8c$	$f \geq 2b_f/3$, $f_1 \cong 0.2f$	
	$b_{f1} \leq 0.85b_{f0}$	$t_f + 3 \geq t_{rib} \geq t_f$ (*1)	
	$d_b \leq 600$		
ERBS-BH type	$a \cong (0.5\sim0.75)b_f$	$a_1 \geq \max(50,5tf)$	figure 2 BH type
	$b \cong (0.65\sim0.85)\,d_b$	$c \cong (0.2b_f \leq c \leq 0.25b_f)$	
	$R = (4c \cdot c + b \cdot b)/8c$	$f \geq 1.2b_f$, $f_1 \cong 0.1f$ (*2)	
	$b_{f1} \leq 0.85b_{f0}$		
	$d_b \leq 900$		

(Reference 1) "FEMA-350, Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings, June 2000"

(*1) Align the beam flange bottom and horizontal beam end wing, when $t_{rib} > t_f$ using the back up bar.

(*2) Set the joint at the end of f, or on the inner side toward the beam center.

(*3) Boxing in the H-type inner wing is not always required.

Definition of terms

Reduced section : reduced bottom section where beam flange width is b_{f1}

d_b : beam depth
b_{f0} : center flange width
b_f : inner flange width
c : depth of flange reduction
b_{f1} : reduced flange width
b_{fe} : flange width at face
α : point at flange width is b_f
a : length between face and α
b : length of reduction
f : length of the inner wing
f_1 : parallel part length of the inner wing
r : radius of reduction
t_f : flange thickness
Z_p : plastic section modulus at reduced section (including web)
M_f : end beam moment
Z_b : plastic section modulus at beam end, $Z_b = t_f \cdot b_{fe} \cdot (d_b - t_f)$
A_{f1} : flange cross sectional area at reduced section, $A_{f1} = b_{f1} \cdot t_f$
e : space size at beam flange side

Table 2 Prequalification Data for ERBS Connections

Parameters & Details	Limitations
Beam symmetry	Wing and reduction shape should be symmetric.
Width-thickness ratio of beam	FA rank. Evaluate it using the center flange width b_{f0}, even with the reduced section.
Lateral stiffening of the beam	"Lateral stiffening ensuring horizontal load carrying capacity". Evaluate using A_{f1} and i_y at the reduction length. Gusset plates of stiffening member should not be welded, within the range a+b from the beam end.
Minimum span- to-depth ratio	[span／depth] ≥5
Centre flange width	Centre flange width: $b_{f0} \le 300$
Flange thickness t_f	40mm maximum
Eccentric connection to column	Flange width at face b_{fe} is within the column width.
Floor rigidity	Concrete slab or a metal deck and concrete slab or horizontal bracing should be used.
Welded studs of composite slab	Welded studs should not be placed in the area of the beam flange between the column face and 6 inches beyond the extreme end of the ERBS(a+b+15cm).
Supplementary information about the shape	・Point α provides length a and b. ・When flange width of the face is increased by more than b_f, extend it nearly in the tangential direction from the point α. Flange width at face b_{fe} is more than b_f. ・Beam end size related to the ultrasonic inspection and welding is shown in Figure 3.

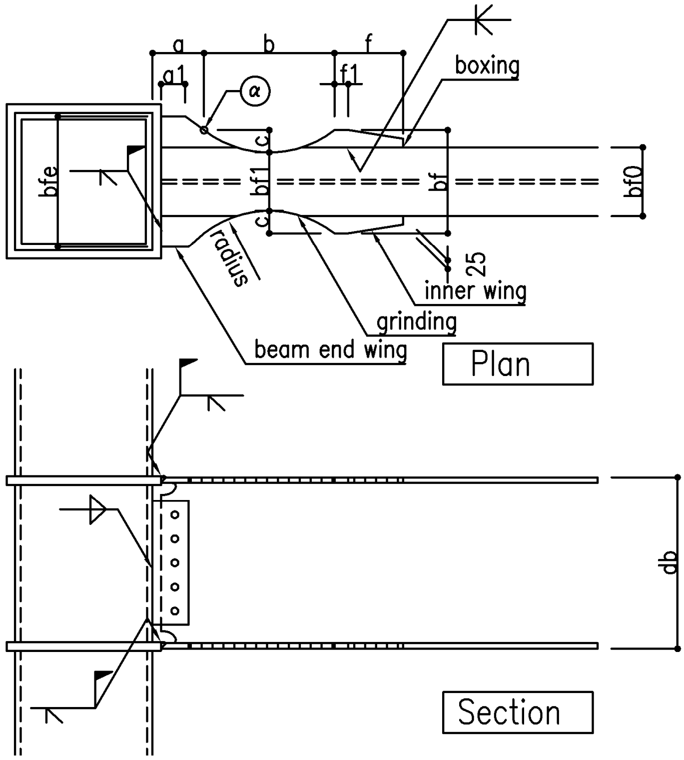


Figure 1 ERBS-H shape and size

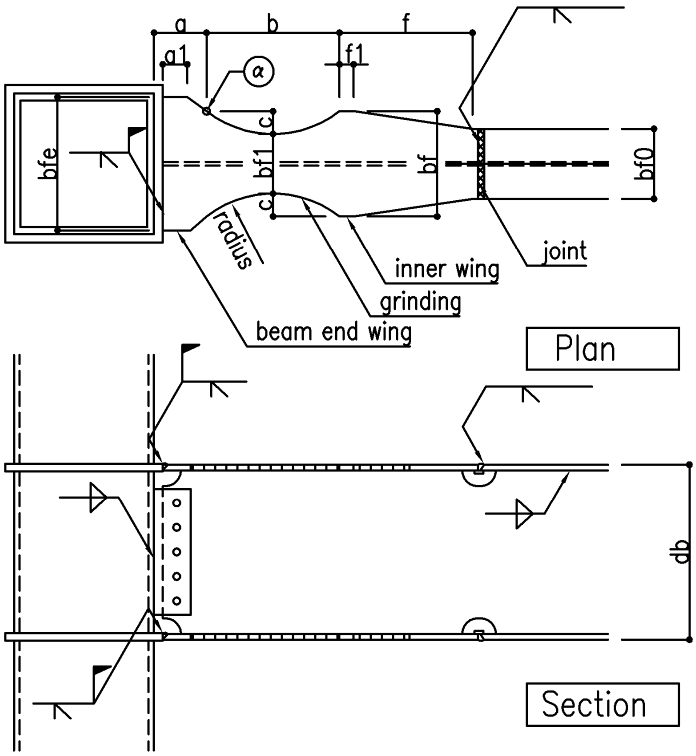


Figure 2 ERBS-BH shape and size

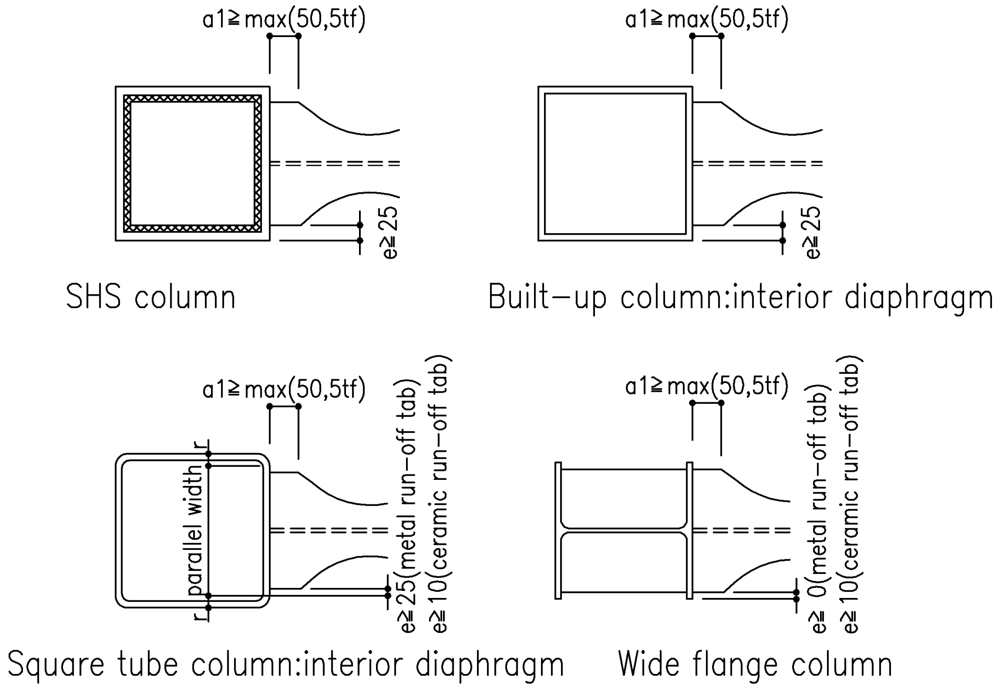


Figure3 Beam end size for various methods

The beam shape design flow of H and BH types is shown in figure 4.

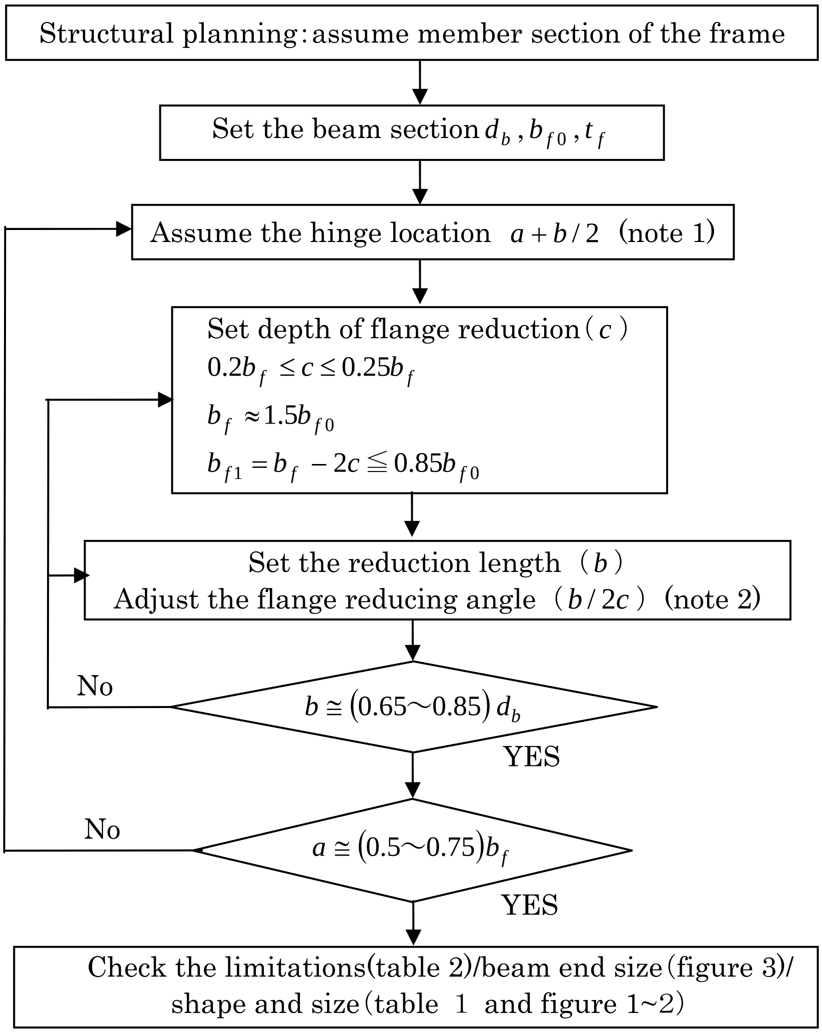
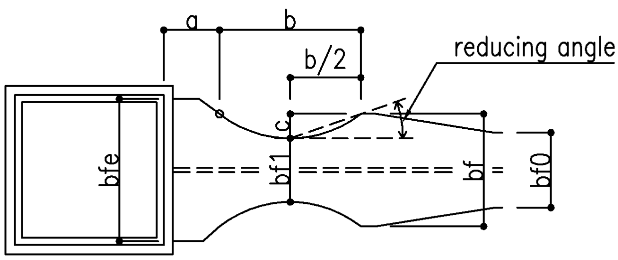


Figure 4 Beam shape design flow

(Note 1) Initial value of the hinge location referenced according to the experiment is shown below.

Section	Hinge location for reference
$d_b / b_{f0} = 3$	$(a + b/2)/ d_b \cong 0.65$
d_b / b_{f0} =2.5(experiment)	$(a + b/2)/ d_b \cong 0.68$
d_b / b_{f0} =2	$(a + b/2)/ d_b \cong 0.80$

(Note 2) Adjust the flange reducing angle, estimated using analysis and experiment.
$b/2c \fallingdotseq 2.9$



② Frame analysis

Frame analysis should be conducted according to the models accurately expressing beam reduction shape characteristics. However, it should be implemented using the following frame analysis method having a rigid zone, when the one described above is not used.

・Model the reduced section and rigid zone using the frame analysis program.
・Set the rigid zone length a + b / 2 from the column face. Set the reduced section length b/2.
・Set the joint location a + b from the column face dividing it into the reduced section and center section.
・The beam stiffness does not consider the composite action with concrete slab.

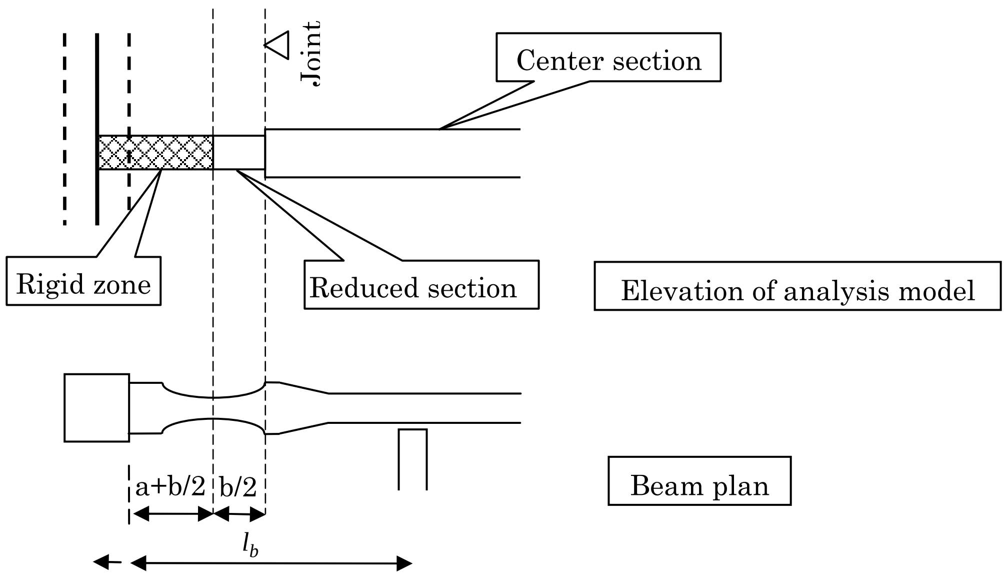


Figure 5 Frame analysis with a rigid zone

③ Check stress on frame structure

・The allowable unit stress of buckling of the bending member shall be evaluated using b_{f1} and t_f in the reduced section. The interval between the support points of the compression flange l_b must be the length between the column face (or column center) and first support point from the column face where the reduced section is located.
・Set the location for the design of the section at the rigid zone end.
・The allowable unit stress of buckling of the compression member of the beams shall be evaluated using the radius of gyration of the reduced section, where the reduced section is located.

④ Check stress concentration of the floor slab

Welded studs should not be placed in the beam flange area between the column face and 6 inches beyond the extreme end of the ERBS (a + b + 15 cm). The shearing force transfer of the floor slab should be designed taking into account the fewer studs effects, where an extra shear force occurs at the brace. The allowable strength of studs meets the requirement of AIJ, 1985, Design Recommendations for Composite Constructions.

⑤ Check the RBS the section

Verify that the bending and shear strength satisfy the RBS design procedure. The beam end connection bears the moment of the flange and the shear force of the web.

i) Design for the beam end moment

・Evaluation of the bending strength of connections

Bending moment working at the end connection of the beam is calculated as follows.
In this procedure, the beam end moment M_f is determined using the load condition and mechanism, as shown in figure 6~7. When the permanent load condition is different, the corresponding shearing force V_p should be calculated.

$$M_f = M_{pr} + V_p x = C_{pr} R_y Z_p F_y + V_p x \quad (1)$$

$$M_{pr} = C_{pr} R_y Z_p F_y \quad (2)$$

M_f : the moment demands at the column face.

M_{pr} : the probable peak plastic hinge moment

C_{pr} : the peak connection strength coefficient, including strain hardening, local restraint, additional reinforcement, and other connection conditions. $C_{pr} = 1.15$ for RBS.

R_y : A coefficient applicable to the beam or girder material, $R_y = 1.1$

Z_p : effective plastic modulus of the section at the location of the plastic hinge (including the web).

F_y : the specified design strength

V_p : the shear force at the plastic hinge.

$$V_p = \frac{M_{pr} + M_{pr} + PL'/2 + wL'^2/2}{L'} \quad (3)$$

x : the length between the plastic hinge and column face, $x = a + b/2$

p : the concentrated load on the beam center

w : the distributed load on the beam.

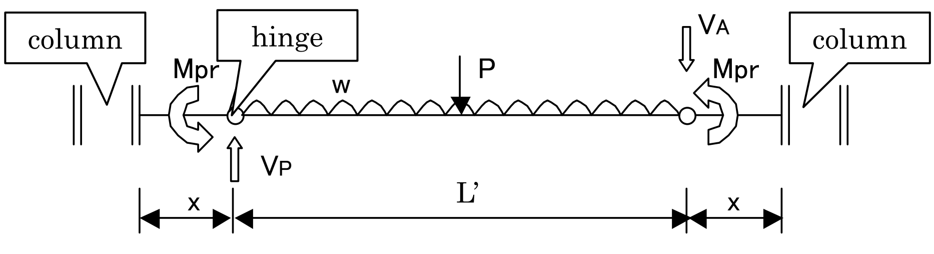


Figure 6 Sample calculation of shear at a plastic hinge taking into account the gravity loads

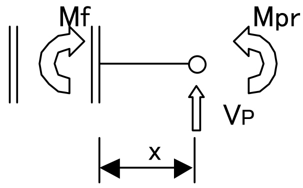


Figure 7 Calculation of demands at column face

・Check the bending strength of the connection

If the following equation is satisfied, the design is acceptable.

$$M_f < R_y Z_b F_y \qquad (4)$$

Z_b : plastic section modulus at beam end, $Z_b = t_f \cdot b_{fe} \cdot (d_b - t_f)$

ii) Design for the shear of the connection

Calculate the shear at the column face V_f , according to the equation:

$$V_f = 2\frac{M_f}{L - d_c} + Q_L \qquad (5)$$

$$\tau = V_f \big/ t_w (d_b - 2t_f - 2S_r)$$

$$\tau / f_s \leqq 1.0$$

where, Q_L : shear due to permanent load.

f_s : allowable shear stress under temporary forces

S_r : height of weld access hole

For the rest, panel zone and continuity plates should be calculated according to "Design Standard for Steel Structures", "AIJ, 2001, Recommendation for Design of Connections in Steel Structures," etc.

⑥ Check the interstory drift angle

When the interstory drift angle is evaluated using the "frame analysis method with rigid zone" in frame analysis, the calculated displacement is smaller than the actual one with respect to the less flexural length of the beam. Therefore, it should be checked minutely by the following the simple calculation of safety side estimation. (In order to calculate the displacement accurately, it should be evaluated using another non-uniform beam section model, etc. The calculated result of frame displacement with a non-uniform beam section element is shown in reference A.)

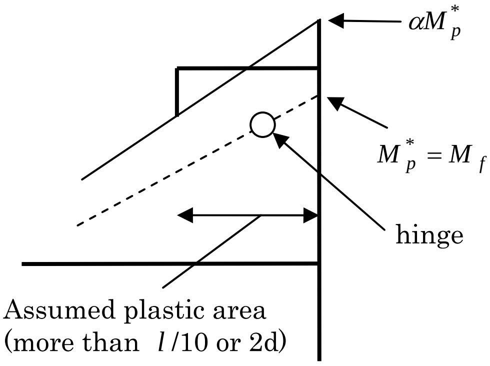
・Model the beam using the uniform section element. The section is the center of the steel beam section. The beam stiffness does not take into account the composite action with concrete slab in the same way as the frame analysis.

・Check that the interstory drift angle is within the allowable range of the calculated displacement.

This frame displacement having a uniform section element provides safety estimation, because the displacement is greater than that of the actual accurate RBS section analysis. The beam deflection caused by permanent loads is safe, because the displacement is greater than the actual one.

⑦ Prevent the damage of the brace end and section connection

Check the "connection to ensure the horizontal load carrying capacity" at the beam end as follows.



where、l:length of beam, d:beam depth

・The moment demands of the connection

The bending moment acting at the beam end of the horizontal load-carrying capacity is the moment demanded at the column face(M_f).

Check that the connection does not break by multiplying the safety factor with the moment. While calculating this moment M_f, the permanent load and C_{pr} is considered as a supplementary factor.

The moment demands of connection αM_p^* is given by the following expression

$$\alpha M_p^* = \alpha M_f = \alpha\left(C_{pr} R_y Z_p F_y + V_p x\right)$$

・Check the ultimate bending moment of the connection

Check that the connection does not break, assuming that the moment demands of connection is αM_p^*. However, it is always satisfied using the following calculation, because the RBS section check is executed using the allowable stress.

Multiplying the safety factor α with the RBS equation,

$$\alpha\left(R_y Z_b F_y\right) > \alpha M_f = \alpha M_p^*$$

(400N quality carbon steel:α=1.3)

$Z_b \sigma_U$ = $400 Z_b$ > $\alpha\left(R_y Z_b F_y\right) = 1.3 \times 1.1 Z_b \times 235 = 336\ Z_b$ > αM_p^* OK

(490N quality carbon steel:α = 1.2)

$Z_b \sigma_U$ = $490 Z_b$ > $\alpha\left(R_y Z_b F_y\right) = 1.2 \times 1.1 Z_b \times 345 = 455\ Z_b > \alpha M_p^*$ OK

⑧ Prevent local buckling, etc.

To prevent lateral buckling, the procedure for checking the beam is as follows:

・Applying the case "secure the lateral stiffening supports at equal distance along the full length of the beam"

The lateral stiffening supports the distance at the beam end within the range for l/n_e. The lateral stiffening supports number n_e with a slenderness ratio along the minor axis of the reduced beam section λ_{ye} by satisfying the following equation.

$\lambda_{ye} \leqq 170 + 20n_e$ (400 N quality carbon steel)

$\lambda_{ye} \leqq 130 + 20n_e$ (490 N quality carbon steel)

The lateral stiffening interval at the center of the beam is within the range for l/n_c. n_c is the lateral stiffening number with a slenderness ratio along the minor axis of the beam center section λ_{yc} by satisfying the following equation.

$\lambda_{yc} \leqq 170 + 20n_c$ (400 N quality carbon steel)

$\lambda_{yc} \leqq 130 + 20n_c$ (490 N quality carbon steel)

,where l :beam length

λ_{ye} :slenderness ratio along the minor axis of the reduced beam section ($= l / i_{ye}$)

λ_{yc} :slenderness ratio along the minor axis of the beam center section ($= l / i_{yc}$)

i_{ye} :radius of gyration along the minor axis of the reduced beam section, $i_{ye} = \sqrt{I_{y1}/A_1}$

i_{yc} :radius of gyration along the minor axis of the beam center section, $i_{yc} = \sqrt{I_y/A}$

I_{y1}, I_y :moment of inertia of the reduced beam section and beam center section along the minor axis.

A_1, A :area of the reduced beam section and the beam center section.

・Applying the case "secure the lateral stiffening member mainly near the beam end"

Secure the lateral stiffening supports at the distance calculated using the following equation in the area where the bending moment exceeds the yield moment. The moment distribution used to calculate the lateral stiffening supports is also estimated, assuming that the moment at the column face is M_f. Furthermore, the yield moment should be calculated using the reduced beam section at the end and the beam center section at the center of the beam.

Multiply the safety factor α with the moment distribution for calculating the lateral stiffening supports. The safety factor α is 1.2 for 400 N quality carbon steel, and 1.1 for 490 N quality carbon steel.

For 400 N quality carbon steel,

the distance between the supports at the end: $\dfrac{l_{be} \cdot d_b}{A_{f1}} \leqq 250$ and $\dfrac{l_{be}}{i_{ye}} \leqq 65$

the distance between the supports at the center: $\dfrac{l_{bc} \cdot d_b}{A_{f0}} \leqq 250$ and $\dfrac{l_{bc}}{i_{yc}} \leqq 65$

For 490 N quality carbon steel,

the distance between the supports at the end: $\dfrac{l_{be} \cdot d_b}{A_{f1}} \leqq 200$ and $\dfrac{l_{be}}{i_{ye}} \leqq 50$

the distance between the supports at the center: $\dfrac{l_{bc} \cdot d_b}{A_{f0}} \leqq 200$ and $\dfrac{l_{bc}}{i_{yc}} \leqq 50$

A_{f1} : flange cross-sectional area at reduced section, $A_{f1} = b_{f1} \cdot t_f$

A_{f0} : flange cross-sectional area at the center, $A_{f0} = b_{f0} \cdot t_f$

l_{be} : the distance between the supports at the reduced beam section

l_{bc} : the distance between the supports at the center

⑨ Check the horizontal load–carrying capacity

Assume that the plastic hinges of the beam develop at the reduced beam section while calculating the horizontal load carrying capacity. C_{pr} is not multiplied while calculating the horizontal load-carrying capacity.

If it is calculated using the frame analysis program, it can be executed using the same rigid zone model employing the frame analysis method of the 1st step design.

＜Reference A＞ An example of the frame displacement calculation

An example of the frame displacement calculation using the non-uniform beam element is shown below.

The element and model used in the analysis is shown in Figure A-1, and the analyzed cases are shown in Table A-1.

Table A-1 Analyzed cases

Case	End model	Hinge model	Center model	Notes
1 non-uniform beam section	ERBS beam	ERBS beam	Center section	Calculation by actual shape
2 uniform beam section	Center section			For displacement calculation
3 rigid zone + steel beam	Rigid zone	Reduced section	Center section	For strength calculation

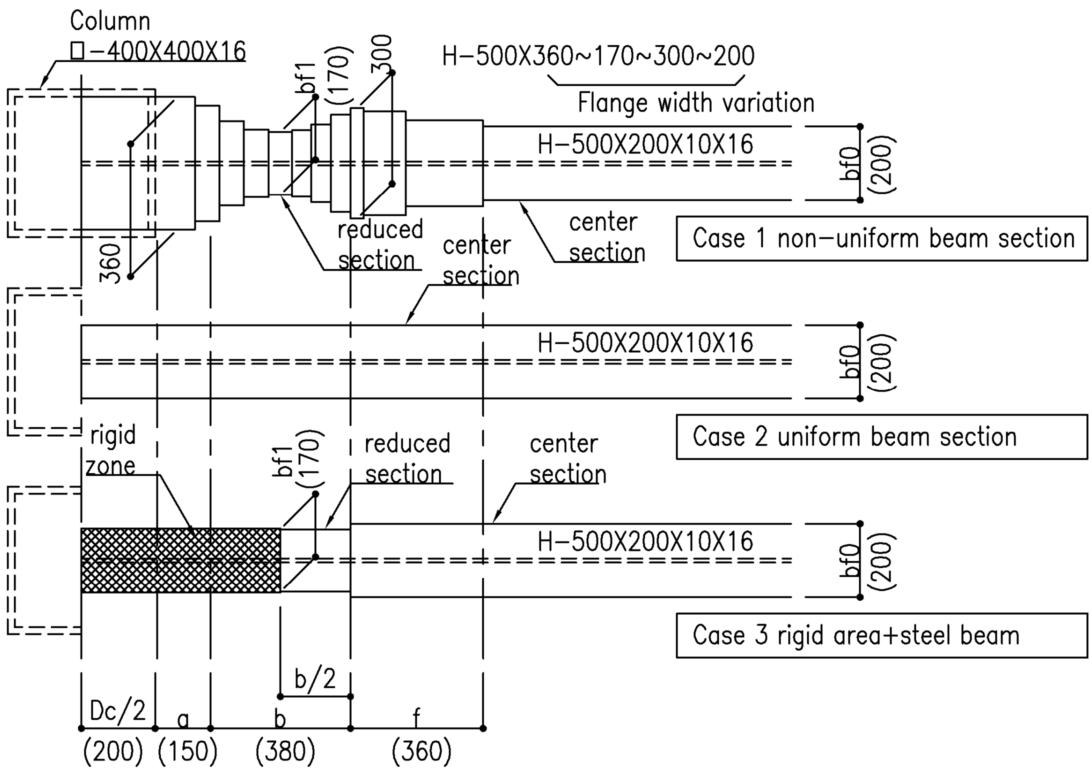


Figure A-1 Element and model used in the analysis

The input load and frame model is shown in Figure A-2.

"Case 2: uniform beam section" gives greater displacement than "Case 1: non-uniform beam section" using the calculated result shown in Table A-2, and it provides an estimate of safety.

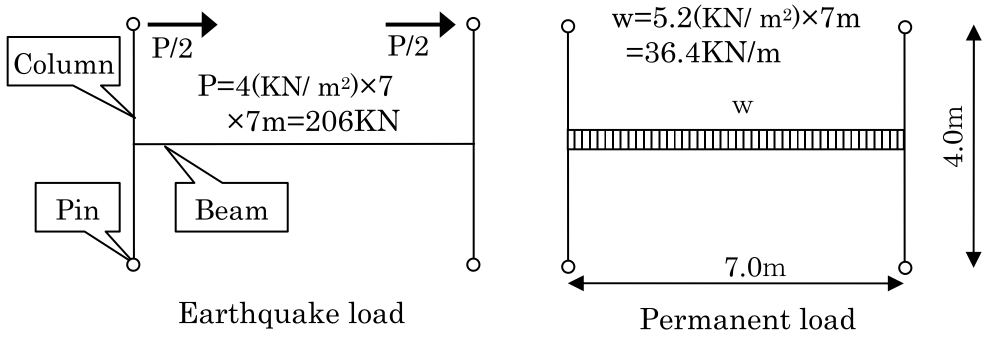


Figure A-2 Input load and frame model

Table A-2 Calculated Results

Case	Horizontal displacement in earthquake		Vertical deflection in permanent load	
	displacement (cm)	ratio (%)	displacement (cm)	ratio (%)
1. non-uniform beam section (Calculation by actual shape)	2.17	100	0.28	100
2. uniform beam section (For displacement calculation)	2.46	113	0.31	111
3. rigid zone + steel beam (For strength calculation)	1.71	79	0.22	79

＜Reference B＞ Flange reduction fabrication

The side of an arc along the reduction length b should be ground conforming both H and BH types. In H type, the point of intersection of the reduced arc and beam flange (marked ○ in the figure) should be included in the welding area, as shown in the following figure.