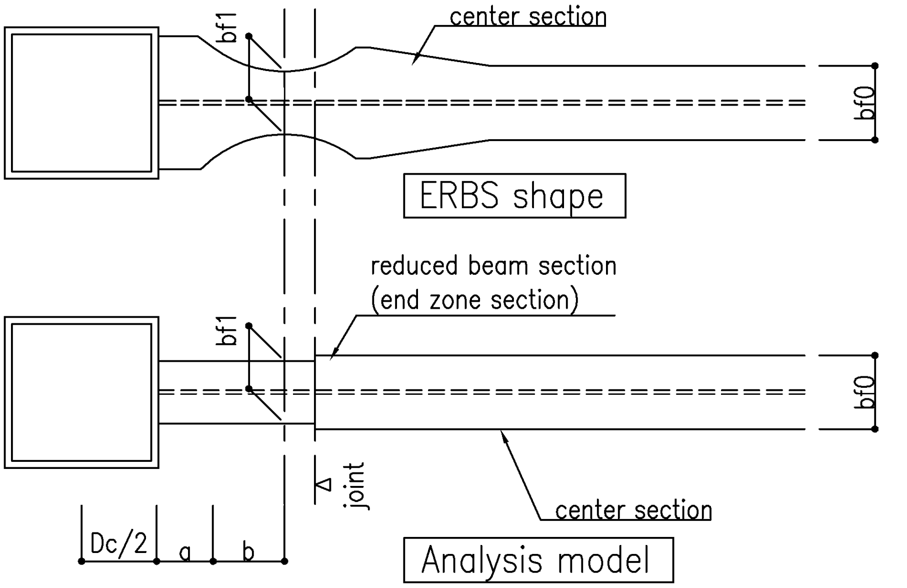
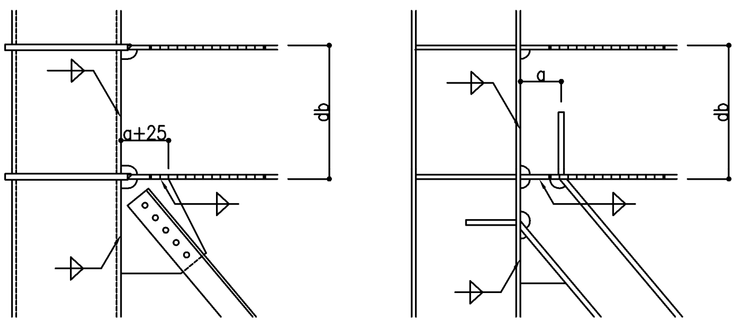
ERBS main frame calculation

- A structural calculation program is used to the design of the main frame in this example.
- The analysis model can be used a three-dimensional analysis model that takes into account bending and shear deformation of members.
- The interstory displacement and vertical deflection of the girder are evaluated as a uniform beam using the central cross section.
- The cross-sectional calculation position is set as "Dc/2+a+b/2" from the column center, and the verification is performed at the reduced section.
- The cross-sectional calculation position is set as "Dc/2+a+b/2" from the column center, and checked using the reduced section.
- Verification of the shape and face cross section of the RBS beam will be calculated later.
- Check the beam joint by another modified calculation data.



Variable cross section beam analysis model

- Remark: If a brace is installed, design must be made to ensure that the deformation around the reduced arc is not disturbed.



Brace installation (example)

1) Paste the beam dead load stress etc. on the sheet “out”.

	A	B	C	D	E	F	G	H	I
1	Title	ERBS example							
2	Girder stress output						Length	bending	
3	case=Dead load						of beam	Left	dumm
4	Layer	frame	axis −	axis	name	case	mm	kNm	kNm
5	RFL	A	1	2	RG1x	L	7000	43	
6	RFL	A	2	3	RG1x	L	7000	61	
7	RFL	B	1	2	RG1x	L	7000	43	
8	RFL	B	2	3	RG1x	L	7000	61	
9	RFL	1	A	B	RG1y	L	7000	24	
10	RFL	2	A	B	RG1y	L	7000	26	
11	RFL	3	A	B	RG1y	L	7000	24	
12	3FL	A	1	2	3G1x	L	7000	47	
13	3FL	A	2	3	3G1x	L	7000	53	
14	3FL	B	1	2	3G1x	L	7000	47	
15	3FL	B	2	3	3G1x	L	7000	53	
16	3FL	1	A	B	3G1y	L	7000	25	
17	3FL	2	A	B	3G1y	L	7000	27	

Readme | Design | List | shs | wf | pipe | str | out

2) Confirm that the member length and QL are set in the sheet "str".

QL at hinge Location

	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R	S	T	U	V
1	1	case=	Dead load					8											19		21	
2	2	Layer	Frame	axis −	axis	name	case	Length	bending									Shear				
3	3							of beam	Left	dumm	JOINT	1/4	Centre	1/4	JOINT	dumm	Right	Left	JOINT	Centre	JOINT	Right
4	4		beam position					mm	kNm	kNm	kNm	kNm	kNm	kNm	kNm	kNm	kNm	kN	kN	kN	kN	kN
5	5	RFL	A	1	2	RG1x	L	7000	43	0	39	0	−53	0	57	0	61	66	66	0	72	72
6	6	RFL	A	2	3	RG1x	L	7000	61	0	57	0	−53	0	39	0	43	72	72	0	66	66
7	7	RFL	B	1	2	RG1x	L	7000	43	0	39	0	−53	0	57	0	61	66	66	0	72	72
8	8	RFL	B	2	3	RG1x	L	7000	61	0	57	0	−53	0	39	0	43	72	72	0	66	66
9	9	RFL	1	A	B	RG1y	L	7000	24	0	22	0	−29	0	22	0	24	38	38	0	38	38
10	10	RFL	2	A	B	RG1y	L	7000	26	0	24	0	−33	0	24	0	26	40	40	0	40	40
11	11	RFL	3	A	B	RG1y	L	7000	24	0	22	0	−29	0	22	0	24	38	38	0	38	38
12	12	3FL	A	1	2	3G1x	L	7000	47	0	44	0	−48	0	50	0	53	65	65	0	67	67
13	13	3FL	A	2	3	3G1x	L	7000	53	0	50	0	−48	0	44	0	47	67	67	0	65	65
14	14	3FL	B	1	2	3G1x	L	7000	47	0	44	0	−48	0	50	0	53	65	65	0	67	67
15	15	3FL	B	2	3	3G1x	L	7000	53	0	50	0	−48	0	44	0	47	67	67	0	65	65

Readme | Design | List | shs | wf | pipe | str | ou

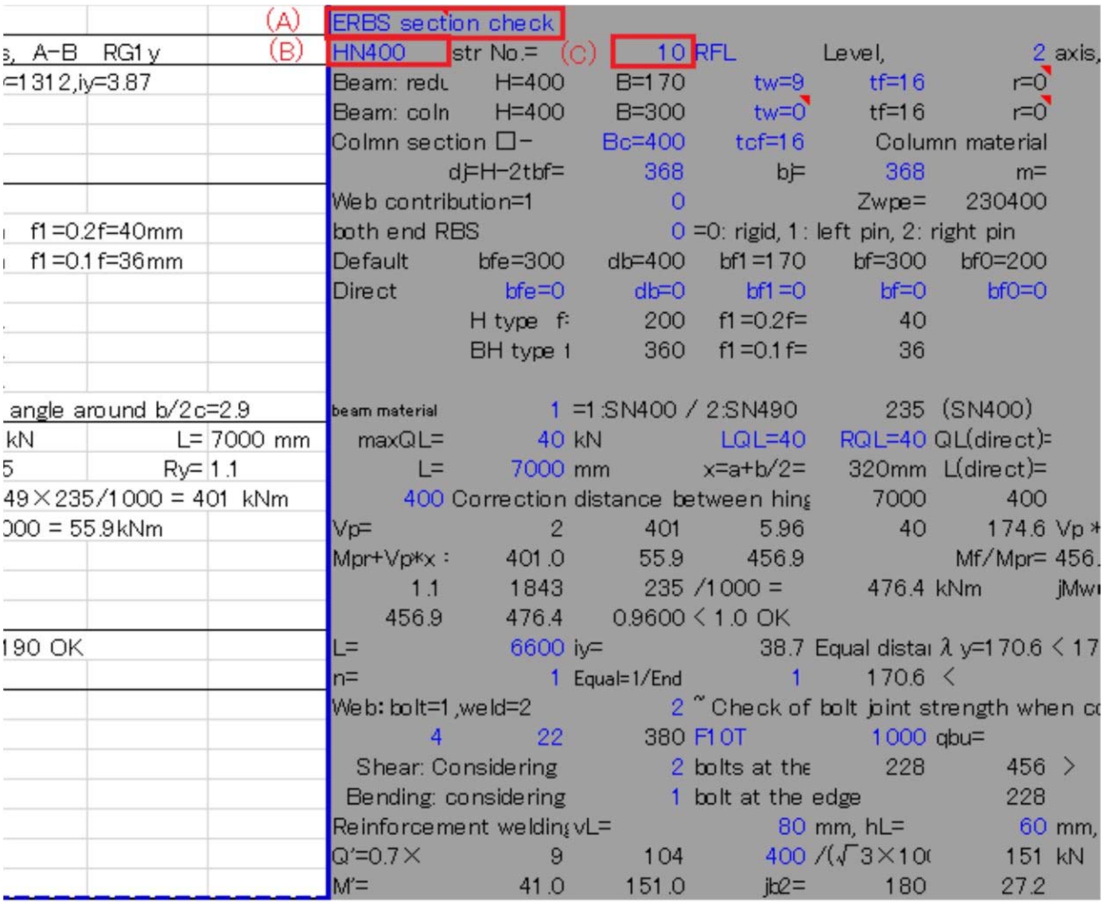
3) Input data into sheet ”shs”, "wf", "pipe". Fill in the blue cells.

(A) Enter a comment.

(B) Enter the beam shape name in "List" (column A).

If the names are the same, the data above takes precedence.

(C) Enter the beam position number "str" (column A).



Check the RBS 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 is determined using the load condition
and mechanism, shown below.
When the permanent load condition is different,
the corresponding shearing force Vp should be calculated.

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

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

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. Cpr=1.15 for RBS.

R_y A coefficient applicable to the beam or girder material, Ry=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

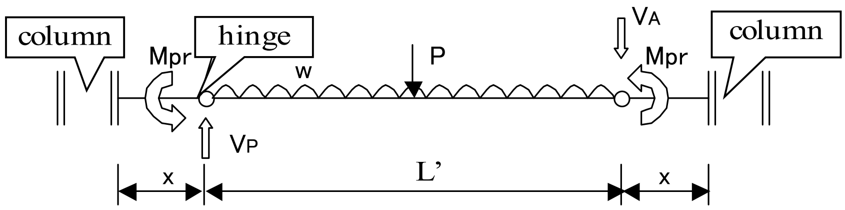
V_p :the shear force at the plastic hinge.

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

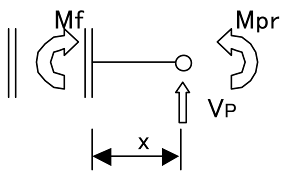
x :the length between the plastic hinge and column face, $x = a + b/2$

p :the concentrated load on the beam center

w :the distributed load on the beam.

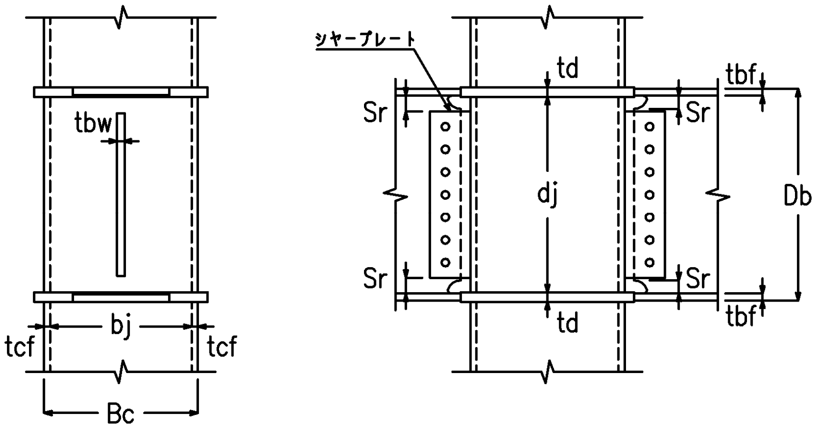


Calculation of shear at a plastic hinge taking into account the gravity loads



Calculation of demands at column face

When evaluating the bending strength of the web, refer to the “AIJ, 2001, Recommendation for Design of Connections in Steel Structures,” etc.



梁端接合部の記号

$${}_jM_{wu} = m \cdot Z_{wpe} \cdot F_{fu}$$

Z_{wpe}: Plastic section modulus of beam web considering loss of scallops etc.

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

m : Nondimensional bending strength of beam web joints

For H-shaped cross section (strong axis direction) m=1

For SHS section $m = min\left\{1, 4\frac{t_{cf}}{d_j}\sqrt{\frac{b_j \cdot F_{cy}}{t_{bw} \cdot F_{wy}}}\right\}$

・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, $Z_b = t_f \cdot b_{fe} \cdot \left(d_b - t_f\right)$

b_j : effective width of column For SHS bj=Bc−2tcf

For steel pipe bj=Bc−tcf

ⅱ) Design for the shear of the connection

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

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

$$\tau = {}^{V_f}/_{t_w}\left(d_b - 2t_f - 2S_\tau\right)$$

$$\tau / f_s \leqq 1.0$$

where, Vp: 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 ″Recommendation for Design of Connections in Steel Structures,” etc.

iii) 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/ne″. The lateral stiffening supports number ″ne″ with a slenderness ratio along the minor axis of the reduced beam section ″ λ ye″ by satisfying the following equation.

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

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

The lateral stiffening interval at the center of the beam is within the range for ″L/ne″. ″nc″ 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.

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

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

,where L: beam length

λ_{ye} : slenderness ratio along the minor axis of the reduced beam section (=L/iye)

λ_{yc} : slenderness ratio along the minor axis of the beam center section (=L/iyc)

i_{ye} : radius of gyration along the minor axis of the reduced beam section, iye = √(Iy1/A1)

i_{yc} : radius of gyration along the minor axis of the beam center section, iyc = √(Iy/A)

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

A_l, 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 Mf. 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, $A_{f0} = b_{f0} \cdot t_f$

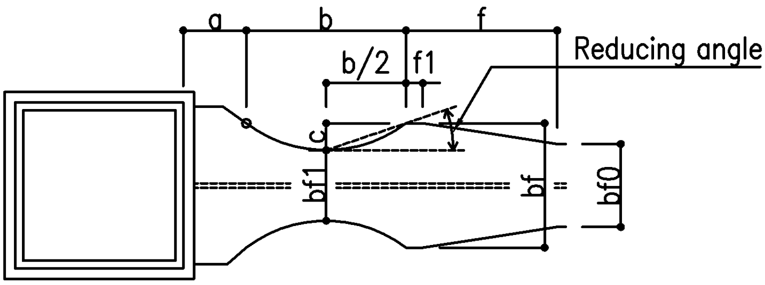
l_{be} :the distance between the supports at the reduced beam section

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

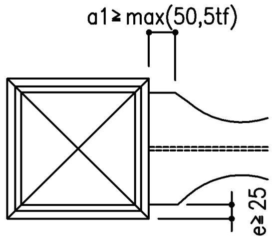
ERBS shape example

Name	beam depth	reduced flange width	inner flange width	center flange width	a	b	c	r	a/bf	b/db	c/bf	bf1/bf0	BH type		H type		reducing angle	inner rib
	db=	bf1	bf=	bf0			0.2~0.25bf		0.5~0.75	0.65~0.85	0.2~0.25	<0.85	f=1.2bf	0.1f	f=2bf/3	0.2f	b/2c	(bf-bf0)/2
HN600	600	170	300	200	180	390	65.0	325	0.60	0.65	0.217	0.850	360	36	200	40	3.00	50.0
HN500rbs	500	110	200	200	150	380	45.0	424	0.75	0.76	0.225	0.550	240	24	133	27	4.22	0.0
HN500	500	170	300	200	150	380	65.0	311	0.50	0.76	0.217	0.850	360	36	200	40	2.92	50.0
HN450	450	170	300	200	150	380	65.0	311	0.50	0.84	0.217	0.850	360	36	200	40	2.92	50.0
HN400	400	170	300	200	150	340	65.0	255	0.50	0.85	0.217	0.850	360	36	200	40	2.62	50.0
HN300	300	126	214	150	120	250	44.0	200	0.56	0.83	0.206	0.840	257	26	143	29	2.84	32.0
HN250	250	105	175	125	110	200	35.0	161	0.63	0.80	0.200	0.840	210	21	117	23	2.86	25.0
HN200	200	84	150	100	100	170	33.0	126	0.67	0.85	0.220	0.840	180	18	100	20	2.58	25.0
HM900	900	240	450	300	240	610	105.0	496	0.53	0.68	0.233	0.800	540	54	300	60	2.90	75.0
HM800	800	240	450	300	240	600	105.0	482	0.53	0.75	0.233	0.800	540	54	300	60	2.86	75.0
HM700	700	240	450	300	230	590	105.0	467	0.51	0.84	0.233	0.800	540	54	300	60	2.81	75.0
HM588	588	230	400	300	210	480	85.0	382	0.53	0.82	0.213	0.767	480	48	267	53	2.82	50.0
HM488	488	230	400	300	200	410	85.0	290	0.50	0.84	0.213	0.767	480	48	267	53	2.41	50.0
HM440	440	230	400	300	200	370	85.0	244	0.50	0.84	0.213	0.767	480	48	267	53	2.18	50.0
HM390	390	230	388	300	200	330	79.0	212	0.52	0.85	0.204	0.767	466	47	259	52	2.09	44.0
HM340	340	180	326	250	180	280	73.0	171	0.55	0.82	0.224	0.720	391	39	217	43	1.92	38.0
HM294	294	150	250	200	150	230	50.0	158	0.60	0.78	0.200	0.750	300	30	167	33	2.30	25.0

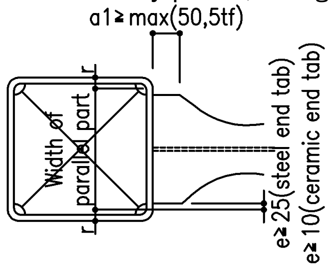
RBS section check



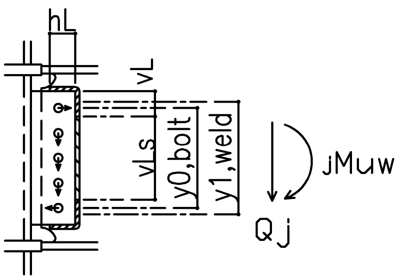
ERBS shape BH type (column: SHS)



continuity plate (through)



continuity plate



- Bolt bending strength
  tg: gusset plate thickness
  qbu: maximum shear strength per bolt
  y0: distance between bolt bending centers of gravity
- Add welding when jMwu>[Bolt bending strength]
  hl: horizontal length of welding
  vl: vertical length of welding
  vls: vertical length of welding (shear)
  y1: distance between weld bending centers of gravity
  jMwu: web shear plate bending strength

Check of web bolt connections (when considering web strength)

ERBS section check

Beam and column shape	Position: RFL Level, 2axis, A–B RG1y

– Beam: reduced section H–400×170×9×16×r0) Zpx=1349,A=87.5, Iy=1312,iy=3.87
– Beam: column face H–400×300×0×16×r0) Zpx=1843
– column section □–400×16 (BCR295)
Internodal distance L=7000 mm

– RBS shape

db= 400mm		H type f=2bf/3=200mm f1=0.2f=40mm
bfe= 300mm	bf= 300mm	BH type f=1.2bf=360mm f1=0.1f=36mm
bf1= 170mm	bf0= 200mm	bf1/bf0=0.85 < 0.85OK
a= 150mm	a=(0.5 to 0.75)bf	a/bf= 0.500 OK
b= 340mm	b=(0.65 to 0.85)db	b/db= 0.850 OK
c= 65mm	c=(0.2 to 0.25)bf	c/bf= 0.217 OK
r= 255mm	r=(4c*c+b*b)/8c	b/2c=2.6 : flange reducing angle around b/2c=2.9

– RBS check σy=235 N/mm2 (SN400) QL= 40 kN L= 7000 mm
x=a+b/2= 150+340/2 = 320mm Cpr= 1.15 Ry= 1.1
L'= 7000–400–2×320=5960mm Mpr= 1.15×1.1×1349×235/1000 = 401 kNm
Vp= 2x401/5.96+40 = 174.6 kN Vp * x= 174.6×320/1000 = 55.9kNm
Mf= 401+55.9 = 456.9 kNm
RyZpσy= 1.1×1843×235/1000 = 476.4kNm
Mf/(RyZbσy)= 456.9 / 476.4 = 0.96< 1.0 OK

– Lateral stiffening check Equal distance λy=170.6 < 170+20×1=190 OK
L=6600mm, iy=38.7mm, n=1

ERBS section check

Beam and column shape | Position: 3FL Level, 2axis, A-B 3G1y

- Beam: reduced section H-500×170×12×16×r0) Zpx=1974,A=110.6, Iy=1317,iy=3.45
- Beam: column face H-500×360×0×16×r0) Zpx=2788
- column section □-400×16 (BCR295)
  Internodal distance L=7000 mm

- RBS shape

db= 500mm		H type f=2bf/3=200mm	f1=0.2f=40mm
bfe= 360mm	bf= 300mm	BH type f=1.2bf=360mm	f1=0.1f=36mm
bf1= 170mm	bf0= 200mm	bf1/bf0=0.85 < 0.85OK	
a= 150mm	a=(0.5 to 0.75)bf	a/bf=	0.500 OK
b= 380mm	b=(0.65 to 0.85)db	b/db=	0.760 OK
c= 65mm	c=(0.2 to 0.25)bf	c/bf=	0.217 OK
r= 311mm	r=(4c*c+b*b)/8c	b/2c=2.9:flange reducing angle around b/2c=2.9	

- RBS check σy=235 N/mm2 (SN400) QL= 38 kN L= 7000 mm
  x=a+b/2= 150+380/2 = 340mm Cpr= 1.15 Ry= 1.1
  L'= 7000-400-2×340=5920mm Mpr= 1.15×1.1×1974×235/1000 = 586.8 kNm
  Vp= 2x586.8/5.92+38 = 236.2 kN Vp * x= 236.2×340/1000 = 80.3kNm
  Mf= 586.8+80.3 = 667.1 kNm
  RyZpσy= 1.1×2788×235/1000 = 720.6kNm
  Mf/(RyZbσy)= 667.1 / 720.6 = 0.93< 1.0 OK

- Lateral stiffening check Equal distance λy=191.4 < 170+20×2=210 OK
  L=6600mm, iy=34.5mm, n=2

ERBS section check

Beam and column shape | Position: 2FL Level, 2axis, A-B 2G1y

- Beam: reduced section H-600×170×12×19×r0) Zpx=2824,A=132, Iy=1564,iy=3.44
- Beam: column face H-600×360×0×19×r0) Zpx=3974
- column section □-400×19 (BCR295)
  Internodal distance L=7000 mm

- RBS shape

db= 600mm		H type f=2bf/3=200mm	f1=0.2f=40mm
bfe= 360mm	bf= 300mm	BH type f=1.2bf=360mm	f1=0.1f=36mm
bf1= 170mm	bf0= 200mm	bf1/bf0=0.85 < 0.85OK	
a= 180mm	a=(0.5 to 0.75)bf	a/bf=	0.600 OK
b= 390mm	b=(0.65 to 0.85)db	b/db=	0.650 OK
c= 65mm	c=(0.2 to 0.25)bf	c/bf=	0.217 OK
r= 325mm	r=(4c*c+b*b)/8c	b/2c=3:flange reducing angle around b/2c=2.9	

- RBS check σy=235 N/mm2 (SN400) QL= 39 kN L= 7000 mm
  x=a+b/2= 180+390/2 = 375mm Cpr= 1.15 Ry= 1.1
  L'= 7000-400-2×375=5850mm Mpr= 1.15×1.1×2824×235/1000 = 839.5 kNm
  Vp= 2x839.5/5.85+39 = 326 kN Vp * x= 326×375/1000 = 122.3kNm
  Mf= 839.5+122.3 = 961.8 kNm
  RyZpσy= 1.1×3974×235/1000 = 1027.2kNm
  Mf/(RyZbσy)= 961.8 / 1027.2 = 0.94< 1.0 OK

- Lateral stiffening check Equal distance λy=191.9 < 170+20×2=210 OK
  L=6600mm, iy=34.4mm, n=2

RBS section check (Web bending strength considered, lateral buckling: end, web: bolted connection)

Beam and column shape — Position: 3FL Level, 2axis, A-B 3G1y

- Beam: reduced section H-500×110×12×19×r0) Zpx=1646,A=97.2, Iy=428,iy=2.1
- Beam: column face H-500×200×0×19×r0) Zpx=1828
- column section □-400×19 (BCR295) dj=H-2tbf=462, bj=362, m=1, web hight ls=320

  Internodal distance L=7000 mm tg=12mm Zwpe=307200, jMwu=91kNm

- RBS shape

  db= 500mm H type f=2bf/3=133mm f1=0.2f=27mm

  bfe= 200mm bf= 200mm BH type f=1.2bf=240mm f1=0.1f=24mm

  bf1= 110mm bf0= 200mm bf1/bf0=0.55 < 0.85OK

  a= 150mm a=(0.5 to 0.75)bf a/bf= 0.750 OK

  b= 380mm b=(0.65 to 0.85)db b/db= 0.760 OK

  c= 45mm c=(0.2 to 0.25)bf c/bf= 0.225 OK

  r= 424mm r=(4c*c+b*b)/8c b/2c=4.2：flange reducing angle around b/2c=2.9

- RBS check σy=235 N/mm2 (SN400) QL= 38 kN L= 7000 mm

  x=a+b/2= 150+380/2 = 340mm Cpr= 1.15 Ry= 1.1

  L'= 7000-400-2×340=5920mm Mpr= 1.15×1.1×1646×235/1000 = 489.3 kNm

  Vp= 2x489.3/5.92+38 = 203.3 kN Vp ＊ x= 203.3×340/1000 = 69.1kNm

  Mf= 489.3+69.1 = 558.4 kNm

  RyZpσy+jMwu= 1.1×1828×235/1000 +91=472.5+91=563.5kNm

  Mf/(RyZbσy)= 558.4 / 563.5 = 1< 1.0 OK

- Lateral stiffening check End of beam Lb･H/Af=1000×500/2090=240 < 250 OK

  L=6600mm, iy=21mm, n=5 Lb/iy=1000/21=47.7 < 65 OK

- Check web bolts strength

  5-M22, Ab=380, F10T, σu=1000N/mm2, qbu=228kN Bolt bending center distance y=240

  Shear: Considering 3bolts at the center Qu=228×3=684kN > Vp=203.3 OK

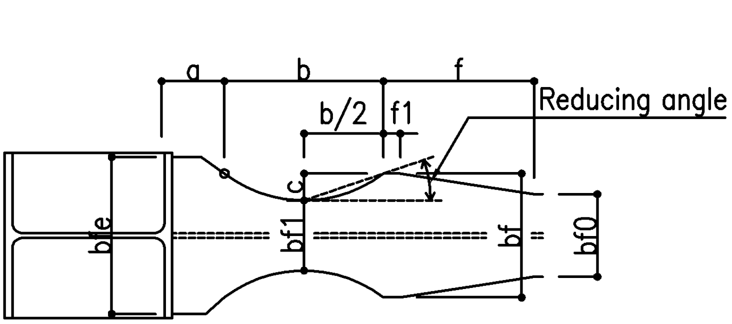
  Bending: considering1bolt at the edge1×228×240/1000=54.7 > jMwu=91kNm NG

  Reinforcement welding,vL=80mm, hL=60mm, s=12mm, Le=92mm, Weld distance jb2=240mm

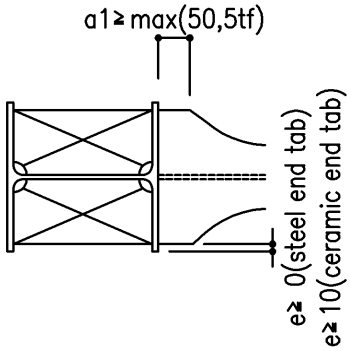
  Q'=0.7×12×92×400/($\sqrt{3}$×1000)=178kN

  M'=54.7+178×240/1000=54.7+42.7=97.4 > 91kNm OK

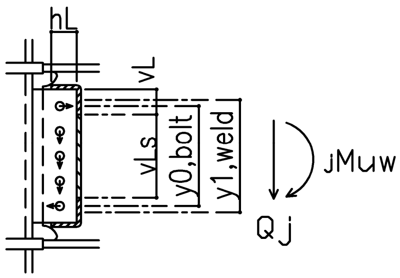
RBS section check



ERBS shape BH type (column: wide flange)



Wide flange column



- Bolt bending strength
  tg: gusset plate thickness
  qbu: maximum shear strength per bolt
  y0: distance between bolt bending centers of gravity
- Add welding when jMwu>[Bolt bending strength]
  hl: horizontal length of welding
  vl: vertical length of welding
  vls: vertical length of welding (shear)
  y1: distance between weld bending centers of gravity
  jMwu: web shear plate bending strength

Check of web bolt connections (when considering web strength)

ERBS section check

Beam and column shape — Position: RFL Level, 2axis, A-B RG1y

- Beam: reduced section H-400×170×9×16×r0) Zpx=1349,A=87.5, Iy=1312,iy=3.87
- Beam: column face H-400×300×0×16×r0) Zpx=1843
- column section H-400 (SN400) web hight ls=260
  Internodal distance L=7000 mm tg=9mm Zwpe=152100, jMwu=36kNm

- RBS shape

db= 400mm — H type f=2bf/3=200mm f1=0.2f=40mm
bfe= 300mm — bf= 300mm — BH type f=1.2bf=360mm f1=0.1f=36mm
bf1= 170mm — bf0= 200mm — bf1/bf0=0.85 < 0.85OK
a= 150mm — a=(0.5 to 0.75)bf — a/bf= 0.500 OK
b= 340mm — b=(0.65 to 0.85)db — b/db= 0.850 OK
c= 65mm — c=(0.2 to 0.25)bf — c/bf= 0.217 OK
r= 255mm — r=(4c*c+b*b)/8c — b/2c=2.6 : flange reducing angle around b/2c=2.9

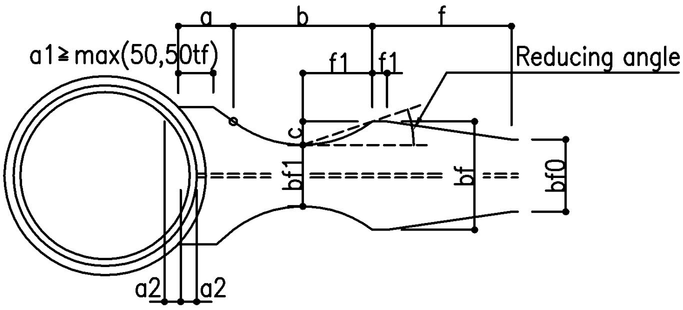
- RBS check — σy=235 N/mm2 (SN400) — QL= 40 kN — L= 7000 mm

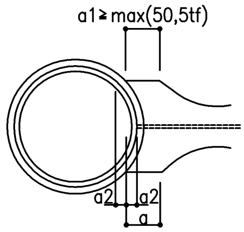
x=a+b/2= 150+340/2 = 320mm — Cpr= 1.15 — Ry= 1.1
L'= 7000-400-2×320=5960mm — Mpr= 1.15×1.1×1349×235/1000 = 401 kNm
Vp= 2x401/5.96+40 = 174.6 kN — Vp * x= 174.6×320/1000 = 55.9kNm
Mf= 401+55.9 = 456.9 kNm
RyZpσy+jMwu= 1.1×1843×235/1000 +36=476.4+36=512.4kNm
Mf/(RyZbσy)= 456.9 / 512.4 = 0.9< 1.0 OK

- Lateral stiffening check — End of beam Lb･H/Af=1650×400/2720=243 < 250 OK
  L=6600mm, iy=38.7mm, n=1 — Lb/iy=1650/38.7=42.7 < 65 OK

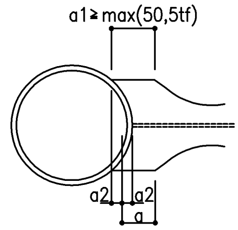
- Check web bolts strength
  4-M20, Ab=314, F10T, σu=1000N/mm2, qbu=188.4kN Bolt bending center distance y=180
  Shear strength: Bolt 2 +weld=Qu=Qu,b+Qu,w=376.8+0=376.8 > Vp=174.6 OK
  Bending: considering 1bolt at the edge, Mu=1×188.4×180/1000=33.9 > jMwu=36kNm NG
  Reinforcement welding,vL=80mm, hL=60mm, s=9mm, Le=104mm, Weld distance jb2=180mm
  Q'=0.7×9×104×400/(√3×1000)=151kN
  M'=33.9+151×180/1000=33.9+27.2=61.1 > 36kNm OK

RBS section check



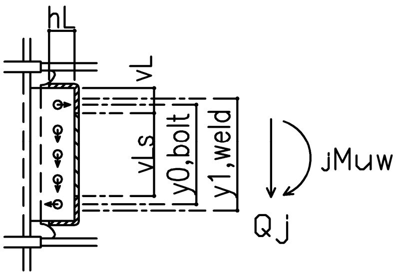


continuity plate (through)



ERBS shape (column: pipe)

continuity plate



- Bolt bending strength
  tg: gusset plate thickness
  qbu: maximum shear strength per bolt
  y0: distance between bolt bending centers of gravity
- Add welding when jMwu>[Bolt bending strength]
  hl: horizontal length of welding
  vl: vertical length of welding
  vls: vertical length of welding (shear)
  y1: distance between weld bending centers of gravity
  jMwu: web shear plate bending strength

Check of web bolt connections (when considering web strength)

ERBS section check

Beam and column shape | Position: RFL Level, 2axis, A-B RG1y

- Beam: reduced section H-400×170×9×16×r0) Zpx=1349,A=87.5, Iy=1312,iy=3.87
- Beam: column face H-400×300×0×16×r0) Zpx=1843
- column section φ-406.4×16(STK400)
  Internodal distance L=7000 mm

- RBS shape

db= 400mm   H type f=2bf/3=200mm   f1=0.2f=40mm
bfe= 300mm   bf= 300mm   BH type f=1.2bf=360mm   f1=0.1f=36mm
bf1= 170mm   bf0= 200mm   bf1/bf0=0.85 < 0.85OK
a= 150mm   a=(0.5 to 0.75)bf   a/bf= 0.500 OK   a2= 33mm
b= 340mm   b=(0.65 to 0.85)db   b/db= 0.850 OK
c= 65mm   c=(0.2 to 0.25)bf   c/bf= 0.217 OK
r= 255mm   r=(4c*c+b*b)/8c   b/2c=2.6 : flange reducing angle around b/2c=2.9

- RBS checl   σy= 235 N/mm2 (SN400)   QL= 40 kN   L= 7000 mm
x=a+b/2-a2= 150+340/2-33 = 287mm   Cpr= 1.15   Ry= 1.1
L'= 7000-406.4-2×287=6019.6mm   Mpr= 1.15×1.1×1349×235/1000 = 401 kNm
Vp= 2x401/6.0196+40 = 173.2 kN   Vp * x= 173.2×287/1000 = 49.7kNm
Mf= 401+49.7 = 450.7 kNm
RyZpσy= 1.1×1843×235/1000 = 476.4kNm
Mf/(RyZbσy)= 450.7 / 476.4 = 0.95< 1.0 OK

- Lateral stiffening check   End of bea Lb・H/Af=1648.4×400/2720=243 < 250 OK
L=6593.6mm, iy=38.7mm, n=1   Lb/iy=1648.4/38.7=42.6 < 65 OK