ELASTO-PLASTIC BEHAVIOR OF HORIZONTAL HAUNCHED BEAM-TO-COLUMN CONNECTION

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SUMMARY

In response to the 1995 Hyogoken-Nanbu earthquake, horizontal haunched beams are being increasingly used in Japan to prevent brittle fractures of beam flanges in steel connections.

In this study, seven simple beam-type specimens with various haunch lengths were statically loaded to determine the elasto-plastic behavior of haunched-beam-to-box-column connections and to recommend rational haunch shapes. Test results were also simulated by an FEM analysis of the load/deflection relationship, and this is discussed on the basis of an analysis using a rupture line to determine effective haunch width. Test results indicate that deformation capacity is increased by incorporating a haunch in the beam end. However, it is also found that a short haunch length cannot completely prevent the beam flange from rupturing. Finally, the rupture line analysis shows that the ratio of haunch length to half clear beam span should be greater than 8.5% to avoid brittle fracture and to assure stable deformation capacity.

INTRODUCTION

In the 1995 Hyogoken-Nanbu earthquake, there were many brittle fractures of beam flanges in the vicinity of steel beam-to-box column moment resisting connections. One cause of these fractures was the stress concentration in welded parts of these connections. This has led to the increasing use of horizontal haunched beams to relieve high stresses by increasing the beam-flange area and to thus prevent their brittle fractures. However, few studies have been carried out on the effect of haunch length on the plastic deformability of haunched beams. In this study, seven simple beam-type specimens with various haunch lengths were statically loaded to determine the elasto-plastic behavior of haunched-beam-to-box-column connections and to recommend rational haunch shapes. The test results are also simulated by an FEM analysis of the load/deflection relationships of beam-to-column connections, and an analysis using a rupture line for effective haunch width is discussed.

This paper presents the structural behavior of the haunched-beam-to-box-column connection obtained from the tests and analyses, and finally proposes suitable haunch lengths.

2. EXPERIMENTAL STUDY

2.1 Specimens

The fundamental configuration of a haunched beam is shown in Fig. 1. The haunched-beam yields first at the start of the haunch when it is subjected to a bending moment generated by a tip load, Pb. Its bending strength increases with amplification of the plasticized zone of the beam until local buckling occurs at the start of the haunch. The local buckling strength factor, s, for a steel grade of SS400 shown in equation (1) is obtained approximately from many stub column tests, calculated from the width-thickness ratio and the mechanical properties of the beam [Japan Society of Steel Construction, 1996]. To avoid brittle fractures, the end of the

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haunched beam is designed to be in the elastic range. Here, l denotes the ratio of the haunch length to half the clear beam span.

$$\frac{1}{s} = \frac{0.4896}{\alpha_f} + \frac{0.046}{\alpha_w} + 0.7606 \qquad (1)$$

where $\alpha_f = \frac{E}{\sigma_{fy}} \times \left(\frac{t_f}{b}\right)^2, \quad \alpha_w = \frac{E}{\sigma_{wy}} \times \left(\frac{2t_w}{d}\right)^2$

σ_{fy}, σ_{wy} :yield point of beam flange and beam web, respectively

t_f, t_w :thickness of beam flange and beam web, respectively

E :Young's modulus

b, d :half beam flange width and clear height of beam web, respectively

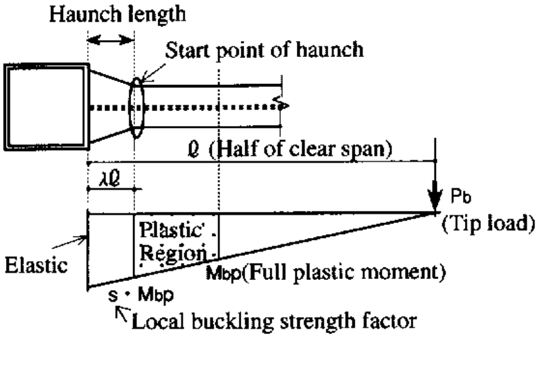


Fig. 1 Horizontal haunched beam in bending

Table 1 List of test specimens

Specimen	Column	Haunch Length ($\lambda \times \ell$)	Web Connection	Comment
BWN	Welded Box	No	Welding	Ordinary Type
BW 3		0.033×ℓ (0.03L)		
BW 5		0.055×ℓ (0.05L)		
BW10		0.111×ℓ (0.10L)		
BB 5		0.055×ℓ (0.05L)	High-Strength Bolt	
CWN	Cold-formed Box	No	Welding	Ordinary Type
CW 5		0.055×ℓ (0.05L)		

(Note) ℓ : Half of clear span length of beam(=1,800mm) ,
L : Half of span length of beam(=2,000mm)

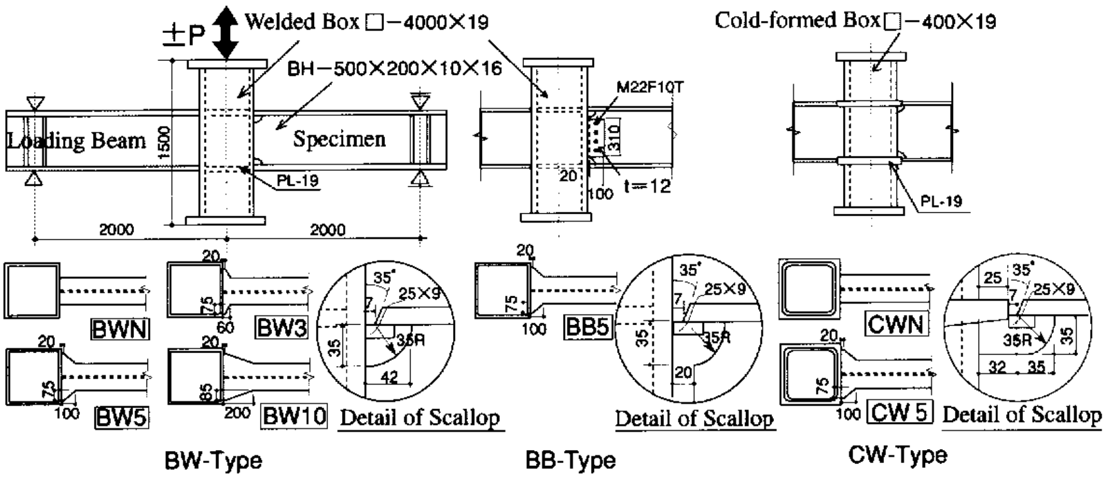


Fig. 2 Test specimens

Seven simple beam-type specimens were loaded, as shown in Table 1 and Fig. 2. Haunch length ratios of 3.3, 5.5, and 11.1 % were selected as main test parameters. The methods of forming the column (both cold forming and welding) and of connecting the beam web to the column (both welding and bolting) were considered, as well as an ordinary connection with a diaphragm. All specimens had ordinary scallops in the beam-to-column welded connection. The mechanical properties of the steel used for the specimens are listed in Table 2. An alternating static load was applied to the column of the specimen that controlled the beam's plastic deformation.

Table 2 Mechanical properties of material

Thickness (mm)	YP (N/mm^2)	TS (N/mm^2)	YR (%)	ε (%)
15.5 (Beam flange)	271	423	64	32
9.7 (Beam Web)	302	435	69	31
18.5 (Column for welded box)	280	437	64	32
18.3 (Column for cold-formed box)	415	467	89	18
18.9 (Diaphragm)	369	533	69	27

(Note) YP : Yield point, TS : Tensile stress, YR : Yield ratio,
ε : Elongation after fracture
Sharpy impact energy of the beam material is 30.0 joule at 0° C
(average of 5 specimens)

2.2 Test Results

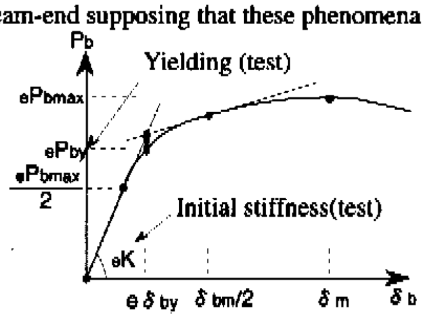
2.2.1 Load-deflection relationships

The load/deflection curves of a typical specimen are shown in Fig. 3, and all test results are compared with calculations in Table 3. In most of the specimens with no haunch or a shorter haunch of less than l of 3 or 5 % the beam flanges ruptured, while in those with a longer haunch the beam flange buckled at the start of the haunch. Photo. 1 shows a typical beam flange rupture, where the rupture extends from the start of the haunch to the opposite-side via a crack-initiated point in the vicinity of the scallop.

Table 3 Results of test and calculation

Specimen	Test results				Calculation results		
	Initial stiffness [1]	Bending yielding [2]	Maximum strength	Type of fracture	Initial stiffness [3]	Bending yielding [4]	Local buckling strenrth [5]
	eK (kN/mm)	ePby (kN)	ePbmax (kN)		cK (kN/mm)	cPby (kN)	cPbsu (kN)
BWN	35.58	269.4	437.2	rupture	38.14 1.07	268.1 1.00	380.2 0.87
BW3	37.70	283.7	447.0	rupture	38.87 1.03	277.4 0.98	393.5 0.88
BW5	38.39	277.6	455.5	buckling	39.57 1.03	285.2 1.03	404.2 0.89
BW10	43.34	298.0	484.1	buckling	41.67 0.96	303.1 1.02	429.4 0.89
BB5	38.01	281.7	432.0	rupture	39.42 1.04	283.8 1.01	402.5 0.93
CWN	36.65	259.2	424.3	rupture	39.56 1.08	271.9 1.05	385.6 0.91
CW5	42.56	281.6	473.5	buckling	40.72 0.96	287.0 1.02	407.1 0.86

(Note) Yielding and buckling strength are calculated at the beam-end supposing that these phenomena occur at the starting point of the haunch.
The ratio of the analytical result to the test result is shown below the analytical resurt.
1),2) : Initial stiffness and yield strength are obtained acccording to the right figure.
3) : Include the part of the haunch
4) : Consider the full section of the beam
5) : Calculation based on the Kato' s formula [Japan Society of Steel Construction, 1996]



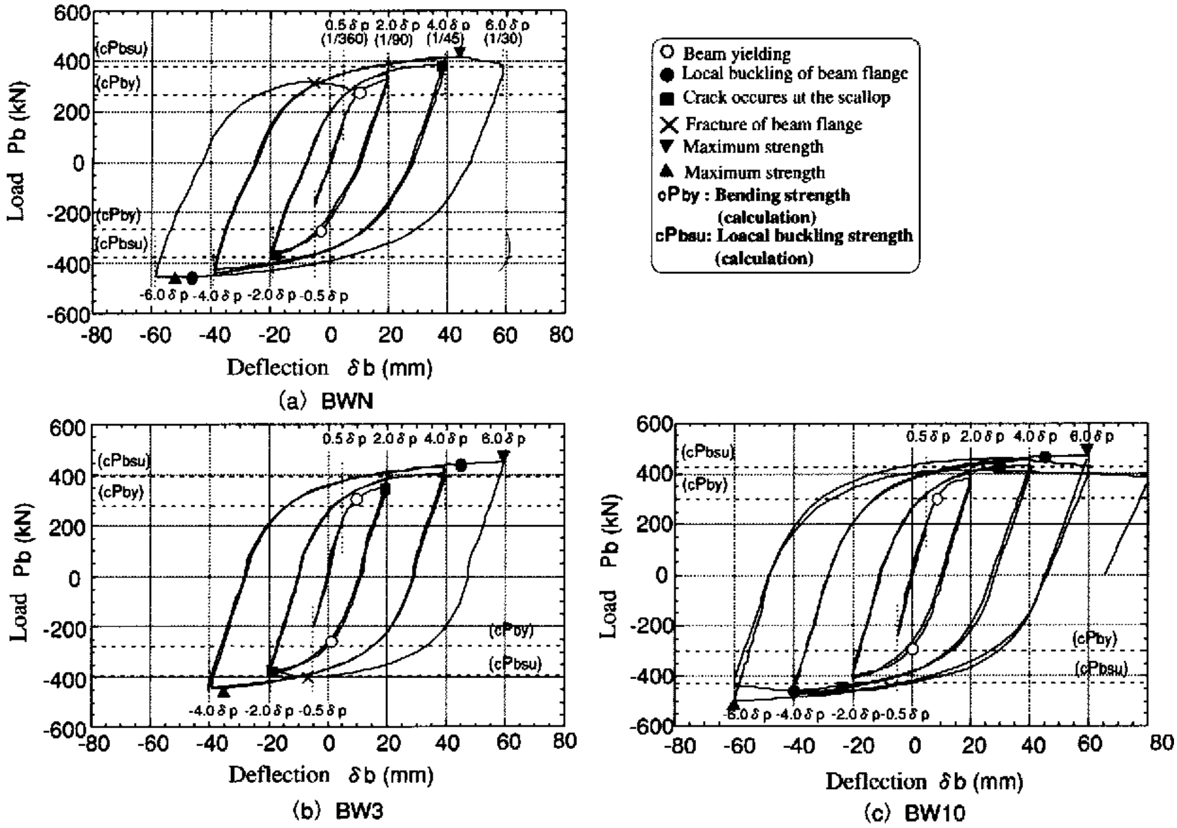


Fig. 3 Load-deflection curves

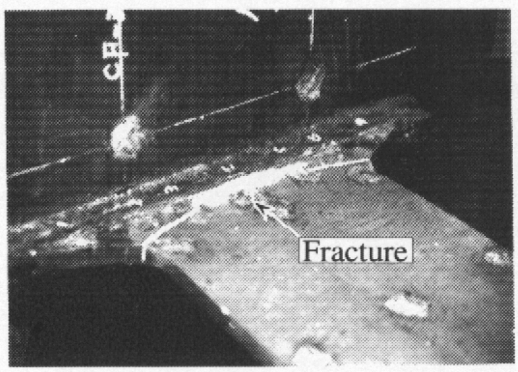


Photo. 1 Rupture in the haunched beam of BW3 with shorter haunch length

The initial stiffness, yield strength and maximum strength all increased with increasing haunch length. The initial stiffness and yield strength can be estimated by the ordinary calculation method by taking into account the sectional properties of the haunch. However, the calculation underestimates the maximum strength for all the specimens. The primary reason for this is that the calculation inherently has a tendency to underestimate the strength due to some built-in safety margins. It can be also considered that the specimens have a small beam width/thickness ratio and this accelerates the increase in strength more than expected. This implies that the generation of excessive stress in the beam ends leads to rupture, because the calculation was used to design the haunch width. Care is therefore necessary in estimating the maximum strength.

2.2.2 Plastic deformability

Fig. 4 shows non-dimensional skeleton curves of the specimens obtained by the method shown in Fig. 5. The method is as follows. First, the load and deflection are normalized by the yield load and deflections. Next, for the positive side of the curves, the part of a hysteresis curve exceeding the last maximum strength is joined in sequence to make a monotonic curve. The negative side is treated in the same manner. All cumulative plastic deformation ratios, h, obtained from this monotonic curve are plotted with l in Fig. 6. The h increases with the increase in l, with the exception of BW3 having the shortest haunch length and BB5 using high-strength bolts for the beam web and the column joint. BB5 has low deformability due to bolt-slippage after beam yielding, because the bolt shear strength was not enough to match the beam shear strength. Therefore, it is indicated that l should be greater than 5% to avoid fracture, and the number of bolts should be equivalent to that required by maximum beam shear strength in bending.

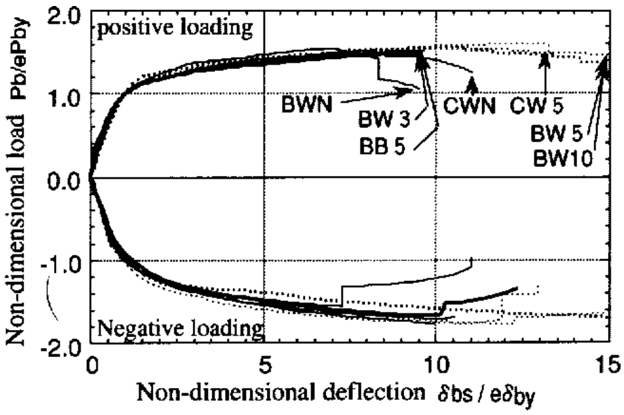


Fig. 4 Non-dimensional skelton curves

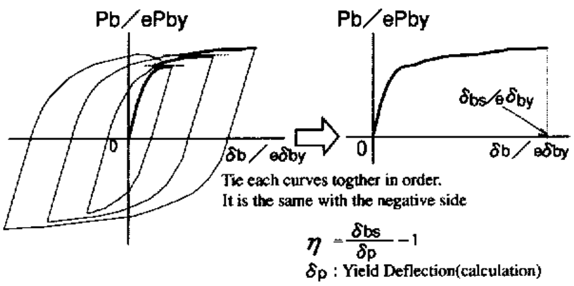


Fig. 5 Definition of skeleton curve and cumulative plastic deformation ratio

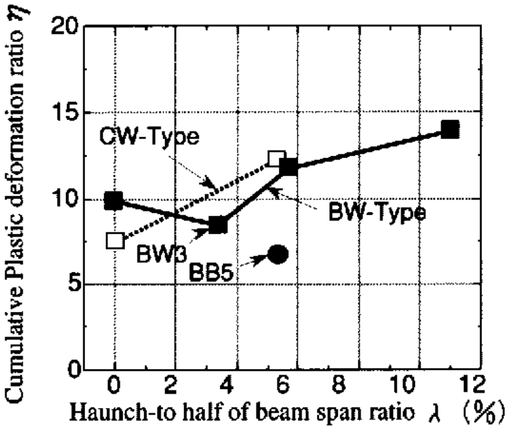


Fig. 6 Plastic deformability

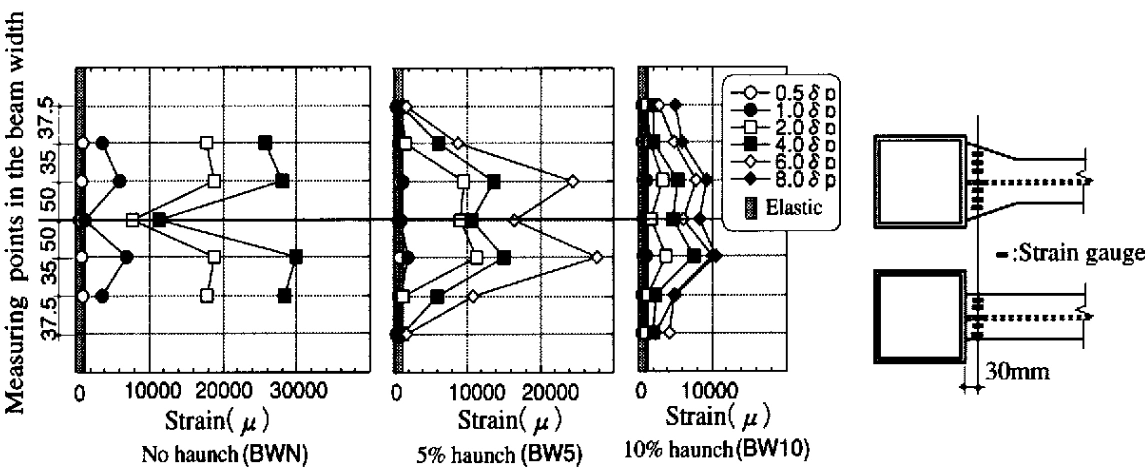


Fig. 7 Strain distribution

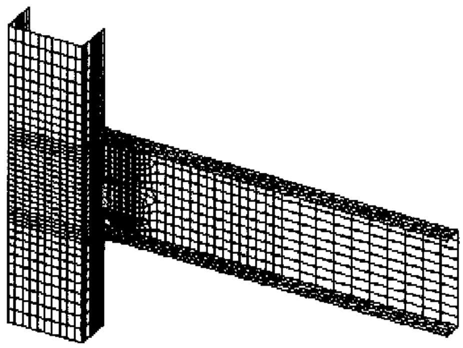
2.2.3 Strain distribution

Typical beam flange strain distributions are shown in Fig. 7. Each plotted line corresponds to a beam deflection from 0.5dp to 8.0dp. Here, dp is yield deflection. It is clear that a haunch smoothes the strain distribution, effectively using all of the haunch, while there were irregular distributions and larger strains in the specimens without a haunch. This indicates that in the specimens with shorter haunches, the haunch width is partly utilized for bending.

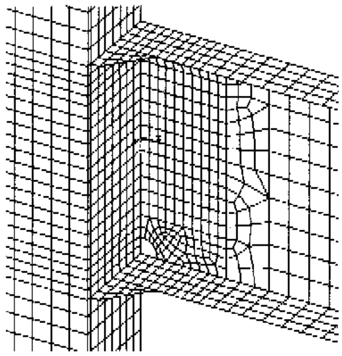
3. ANALYTICAL STUDY

3.1 Nonlinear FEM Analysis

3.1.1 Analytical Method

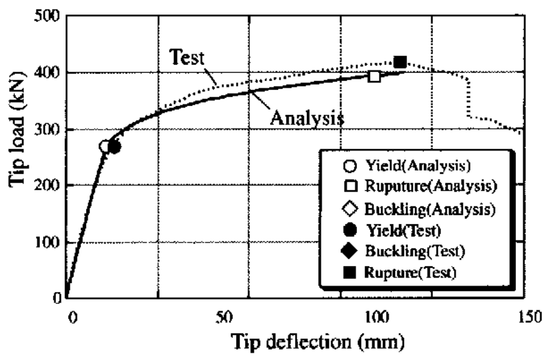


(a)Mesh distribution

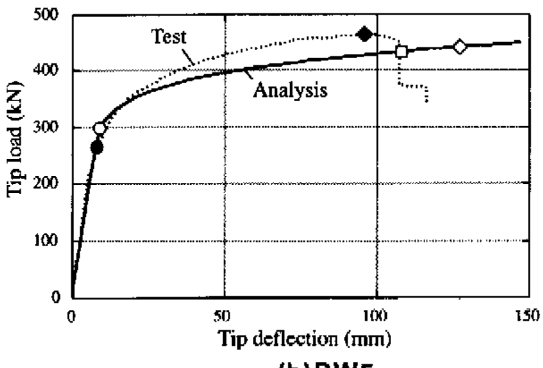


(a)Detail

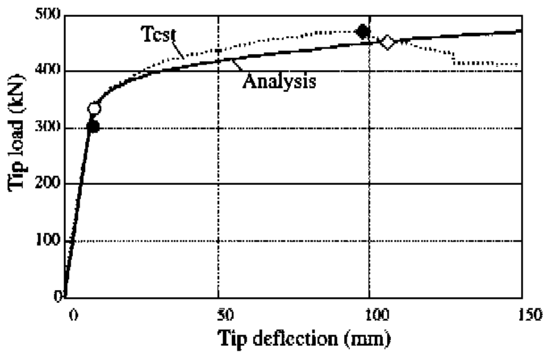
Fig. 8 Analytical model for the specimen BW5



(a)BWN



(b)BW5



(c)BW10

Fig. 9 Analytical results

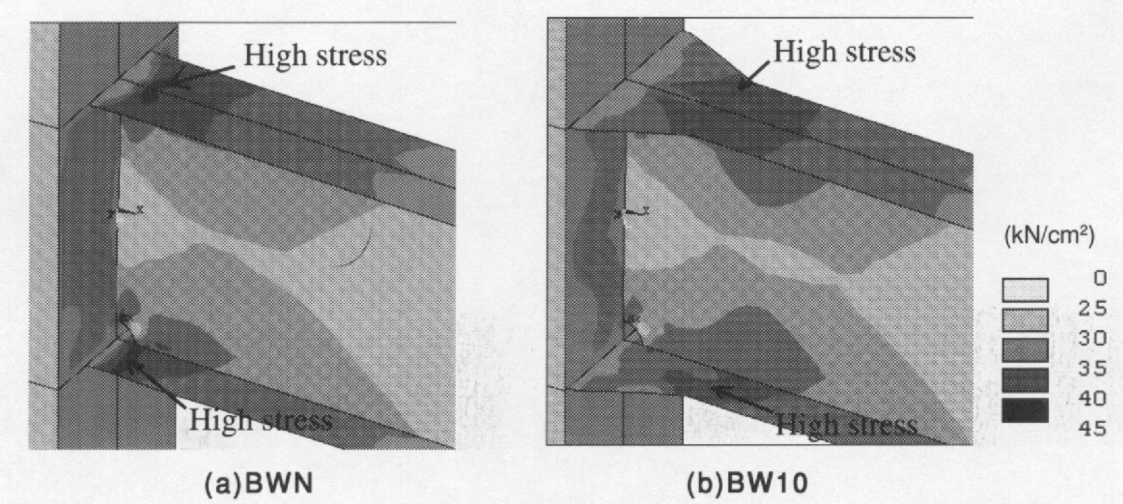


Fig. 10 Von mises countours

The specimen is modeled by four-node shell elements (SHELL 43) using the ANSYS finite element package [ANSYS, 1997] and considering symmetry in the beam web plane. The element mesh of the connection part of the specimen is shown in Fig. 8. The material properties were modeled by Von Mises criterion with the associated plastic-hardening rule based on the Prandtl-Reuss equation. The stress/strain relationship of the steel was of the bi-linear type, where the second stiffness was assumed to be 0.01E (E is Young's Modulus) on the basis of the stress/strain relationship of the steel used for the specimen. The model was given an incremental displacement at its beam tip, while it was supported at both ends of the column; one end was pin-rollered and the other was pined.

3.1.2 Results

The test and analytical skeleton curves for the specimens are presented in Fig. 9. In the analysis, it was assumed that buckling occurs when the wave height of the beam flange buckling reaches 5 mm and rupture occurs when the beam flange stress reaches the maximum steel stress. The analyses accord well with the test results.

Perspective views of the connection stress distribution are shown in Fig. 10. In both figures, the Von Mises stress contours were used at the same deflection, about 74 mm, when specimen BWN without haunches broke out. There was a stress concentration in the vicinity of the beam flange scallop for specimen BWN (indicated by the arrow in the figure). However, for the specimens with the haunches, the location of the high stress concentration moved to the haunch start point and its intensity decreased according to the increase in haunch length

3.2 Analysis for Effective Haunch Width

3.2.1 Analytical Method

The tests and the FEM analytical results show that the haunch length needs to be larger than a certain length to assure plastic deformability by avoiding brittle fractures. Here, the effective haunch width ratio, Rh, which refers to the effective haunch width to beam end haunch width, is introduced based on two rupture lines as shown in Fig. 11. First, the strength of the each rupture line is evaluated. If the strength of rupture line 2 is small, it indicates that the beam flange has little possibility of breaking out because the beam end is designed to be in the elastic range. If the flange breaks at rupture line 1, Rh is denoted as be/bh. be is obtained by converting the strength of rupture line 1 to that of the beam end. The stress is calculated from the sectional properties of the haunch. An Rh of 100% denotes that the entire width of the haunch is utilized for stress relief of the beam end.

3.2.2 Results

The calculated results are shown in Fig. 12 with the test results obtained from the strain distribution when the beam deflection is 2.0dp to 4.0dp . Each calculation line depends on how the stress calculated from the sectional properties. Here, three conditions are considered: the full beam section is effective, the scallop is considered, and

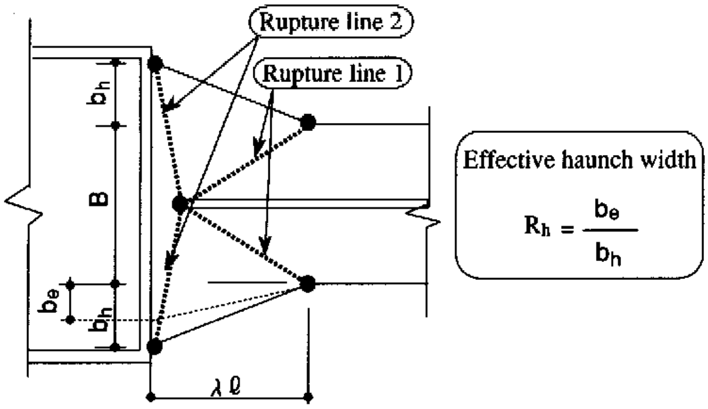


Fig. 11 Rupture lines supposed in the analysis

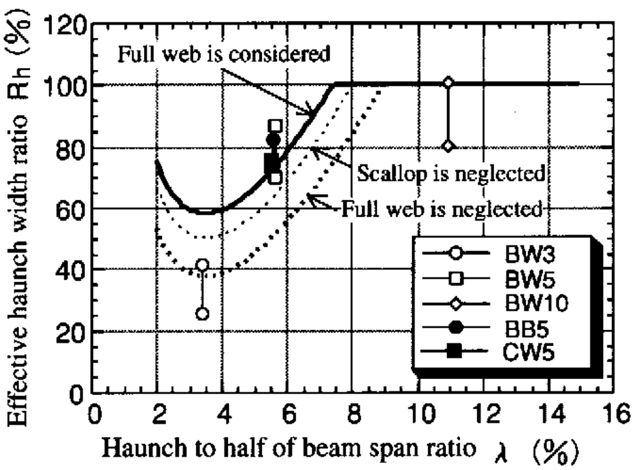


Fig. 12 Analytical results for effective haunch length

full web is neglected. On the condition that the haunched beam can avoid rupture when Rh is 100% or more, where the web is neglected, l should be larger than about 9%, although l should be as large as about 7.5% when the full beam properties are considered. The test results indicate that the beam web acts against bending in a condition between all-web-neglected and scallop-considered. It is thus concluded that the haunch length ratio l should be larger than 8 or 9%.

4. CONCLUDING REMARKS

Through tests and the analyses, the following conclusions are derived:

a) Deformation capacity is increased by attaching a haunch to a beam-end. This is achieved by decreasing the stress and smoothing its distribution. This is confirmed by FEM analysis. However, it is also found that a beam with shorter haunch cannot necessarily avoid rupture. This is because a short haunch doesn't work over the width against bending stress.

b) The rupture line analysis for the effective haunch width accords well with test results. It indicates that the haunch length ratio, l, needs to be over 8.5% to avoid brittle fracture and to assure stable deformation capacity.

c) The elastic stiffness and yield strength are evaluated by the conventional method. However, the calculation underestimates the maximum strength. This implies generation of excessive stress in the beam ends leading to rupture. Care is therefore necessary in estimating the maximum strength.

5. REFERENCES

ANSYS, Inc., ANSYS User's Manual, Rev. 5.4, 1997.

Japan Society of Steel Construction, "Kobe Earthquake Damage to Steel Moment Connections and Suggested Improvement", JSSC Technical Report No. 39, 1996.