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Strength and ductility of thin-walled beam-columns with low yield steel and round corners

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ABSTRACT: Discussed herein is the load-deformation relationship of bridge piers considering the local failure characteristics. Firstly, the strength and ductility of thin tubular beam-column segments are investigated by two series of experiments keeping a focus on the cross sectional shape and inelastic characteristics of material. Assuming bridge piers subjected to strong ground motions, monotonic bending tests under the constant axial compressive force are carried out. The effect of the yield ratio and use of round corners on the strength and ductility is evaluated. It is found that the box section with round corners and the use of structural steel with low yield ratio is desirable for the ductility improvement. Secondly, by modeling the plastic hinge behavior considering the local instability based on the experimental observation, the deformation characteristics of steel bridge piers are investigated by numerical simulation. It is concluded that the effect of local instability on the strength deterioration is significant and the P-δ effect accelerates the degradation.

1 INTRODUCTION

Recently, the structural steels have been often used for the construction of highways around the urban area in Japan because the more complex structural details are necessary due to requirements to construct the large highway systems. Therefore, from the necessity of the high bending and torsional stiffness of structural members against excessive external loads, thin-walled box sections stiffened longitudinally and transversely have been used frequently. However, excessive seismic loads may cause the catastrophic failure such as the local instability of thin plate/shell elements due to the insufficient deformability. Since the strength design based on the elastic limit seems not to be economical if the structures must survive even against destructive earthquakes, it is reasonable to expect the structures resist against the strong earthquake utilizing the inelastic deformation capability or the energy dissipation capacity.

So, it is an urgent task to investigate the inelastic load-deformation characteristics of thin-walled structures such as bridge piers and towers. Although the ultimate strength of structures can be obtained by nonlinear FEM analysis, to evaluate the cyclic deterioration is not an easy task even with highly advanced computation technology. So the experiments must be utilized to provide the fundamentals. In the past, the seismic performance of steel bridge piers has been evaluated by PWRI(Ministry of Construction, 1984), Fukumoto/Kusama(1985a, 1985b) and the authors(1983,1988, 1991). It has been pointed out that the strength deterioration is significant due to buckling of flange/web plates. In order to provide the sufficient ductility, the authors have made continuous effort to identify the effectiveness of round corners of box cross sections instead of sharp corners in order to avoid the cracking along welding at corners of box sections.

In addition to this physical superiority, its aesthetic view is attractive for the construction in urban area. On the other hand, the inelastic characteristics of structural steel such as the yield ratio and yield plateau, plastic modulus and elongation at the breaking point can be controlled through manufacturing process. Particularly, structural steels with the low yield ratio has been applied for building structures to avoid the local instability and to guarantee the sufficient inelastic deformbility. Its efficiency has been reported by Kuwamura's several research works(1988,1989).

In this study, the effect of material inelastic behavior and cross sectional shape on the strength/ductility improvement is investigated experimentally and the horizontal load-displacement relation of steel bridge piers is simulated taking into account the local instability. Firstly, scaled models of a part of column of bridge pier are tested under the constant axial compression and monotonic bending. Secondly, based on the bending moment-curvature relationship of thin-walled beam-column segments with/without local buckling of plate/shell elements, the horizontal load-displacement relation of bridge piers is simulated by using the plastic hinge method, where the P-δ effect on the overall behavior is taken into consideration.

2 DESCRIPTION OF EXPERIMENTAL PROCEDURE

The bridge pier of urban highways considered in this study is the T-shaped pier as shown in Fig. 1. It consists of three parts, namely, the beam part, the intersection part and the column part. In case that such a bridge pier is subjected to strong ground motion, the plastic hinge may be formed at the bottom of column and the other part may behave only elastically. The length of plastic hinge is relatively small compared with the total height of bridge pier, so that the local sectional behavior can be evaluated by the loading condition as shown in Fig. 2, namely, the uniform bending and the constant axial compressive force.

According to the survey by Nakai, et al.(1982) on the steel frame bridge piers designed by 1973 JSHB code, the compressive bending stress is dominant compared with the axial compressive stress and the axial compressive stress is about $0.2\sigma_y$(σ_y is a yield stress). Therefore, in this study, the bending is applied monotonically under constant axial compressive force($0.2P_y$; P_y is a yielding axial force). In order to carry out such a loading test, the testing set-up shown in Fig. 3 is utilized which is consisting of two closed-loop servo-controlled hydraulic actuators and the personal computer for control and data acquisition. The bending moment and axial force diagrams are shown in Fig. 4.

The cross sectional shapes and dimensions of test specimens are summarized in Fig. 5 and Table 1. Firstly, in order to evaluate the yield ratio on the sectional behavior, two different structural steels(yield ratio of H type; 87.5%, yield ratio of L-type; 68.9%) are used. The tensile strength of these steels are the same and their yield stresses are different each other, whose stress-strain curves are shown in Fig.6. Secondly, in order to evaluate the round corners of box sections on the sectional behavior, box sections with round corners(B-type) and stiffened box section with round corners with longitudinal stiffeners(G-type) are made in addition to the standard box section with sharp corners(A-type). The yield ratio of A, B and G types is 60.1%. The generalized width-to-thickness ratio(R) is taken 1.2, 1.0 and 0.8, in order to evaluate the local instability on the sectional behavior.

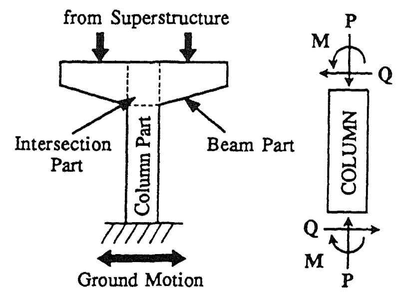

Fig.1. T-shaped steel bridge piers subjected to seismic loads

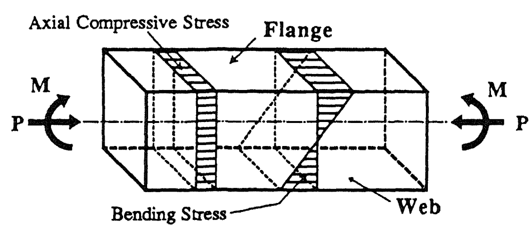

Fig.2. Idealized loading condition of base of bridge piers

Table 1. Dimensions of test specimens (nominal)

Type	Flange Width	Web Height	Corner Radius	Plate Thickness	Length	Stiffener Height	Stiffener Thickness	Cross Sectional Area	Moment of Inertia	Width-to-Thickness Ratio*		
	b(mm)	d(mm)	r(mm)	t(mm)	L_t(mm)	h_s(mm)	t_s(mm)	A(cm^2)	I(cm^4)	R_f	R_w	R_c
H12	197.0	237.0	-----	4.0	600.0	-----	-----	34.2	3027	1.17	1.35	---
H10	168.0	204.0	-----	4.0	600.0	-----	-----	29.3	1910	1.00	1.16	---
H08	139.0	170.0	-----	4.0	600.0	-----	-----	24.2	1094	0.84	0.97	---
L12	222.0	267.0	-----	4.0	600.0	-----	-----	38.6	4340	1.17	1.36	---
L10	190.0	229.0	-----	4.0	600.0	-----	-----	33.0	2725	1.01	1.17	---
L08	157.0	191.0	-----	4.0	600.0	-----	-----	27.3	1563	0.84	0.97	---
A	250.0	300.0	-----	4.5	600.0	-----	-----	48.9	6937	1.01	1.17	---
B	159.0	200.0	50.0	4.5	600.0	-----	-----	45.8	6050	0.63	0.79	0.14
G	159.0	200.0	50.0	4.5	600.0	20.0	4.5	49.4	6381	0.30	0.39	0.14

*Derived from these formulas: $R_f = \frac{b}{t}\sqrt{\frac{12(1-\nu^2)\sigma_y}{K_E E \pi^2}}$, $R_w = \frac{d}{t}\sqrt{\frac{12(1-\nu^2)\sigma_y}{K_E E \pi^2}}$, $R_c = \sqrt{\sqrt{3(1-\nu^2)}\frac{\sigma_y}{E}\frac{r}{t}}$, ($K_E$=4)

Note: $\sigma_y = 385.4(N/mm^2)$, $\sigma_u = 441.3(N/mm^2)$ for H-type, $\sigma_y = 308.9(N/mm^2)$, $\sigma_u = 448.2(N/mm^2)$ for L-type, $\sigma_y = 206.5(N/mm^2)$, $\sigma_u = 343.7(N/mm^2)$ for A,B,G-type

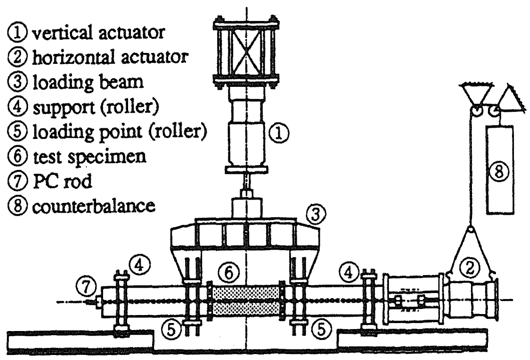

Fig. 3. Test setup

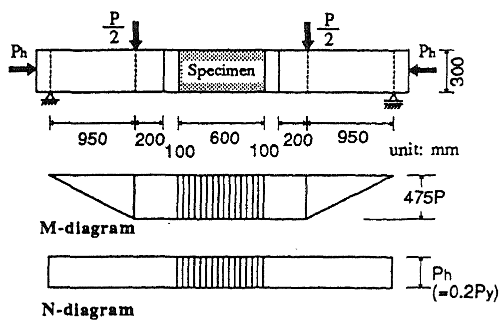

Fig. 4. Bending moment and axial force diagram

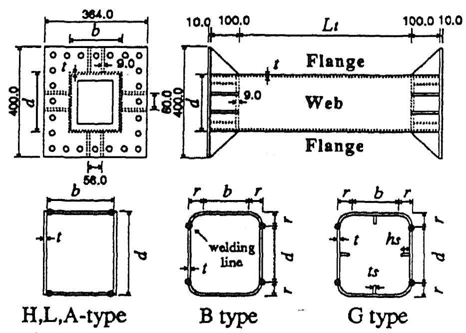

Fig. 5. Cross sectional shapes and dimensions of test specimens (nominal, unit: mm)

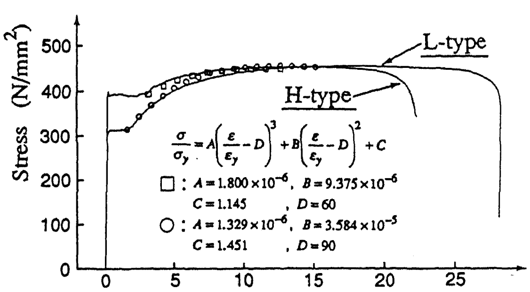

Fig. 6. Stress-strain curves of structural steels with different yield ratios

3 EXPERIMENTAL RESULTS AND DISCUSSIONS

The inelastic behavior considering the local instability is firstly discussed in this section. The bending moment(M)-curvature(ϕ) relationships obtained by the loading tests are shown in Figs. 7 and 8. In these figures, the bending moment and curvature are normalized by their yield values, M_y and ϕ_y, because it makes possible to compare the inelastic characteristics among specimens with different dimension and material properties. It may be reasonable to adopt this normalization of M-ϕ curves by σ_y as a basis of the strength, because the current design procedure of JSHB(1991) is based on σ_y.

In Fig. 7, M-ϕ curves of test specimens with different yield ratios(H and L-types) are shown. Symbols used in this figure can be referred to those given in Table 1. Between H and L-types, there exists no difference of the ultimate strength, but the slight difference of strength degradation after its ultimate strength point. The strength deterioration of H-type is more significant than that of L type for each R value. Considering the loading condition of uniform bending, the strength deterioration may be thought to be affected by the magnitude of the strain-hardening of material, not by the spread of plastification. The yield ratio is controlled through the manufacturing process so called TMCP(Thermo-mechanical Control Process) in the factory. Because of its heat-sensitivity, welding used in the bridge construction may change its characteristics from that pre-designed. Therefore, the high possibility of the composite construction by using several structural steels with different yield/tensile strength is expected in order to improve the strength and ductility as an alternative.

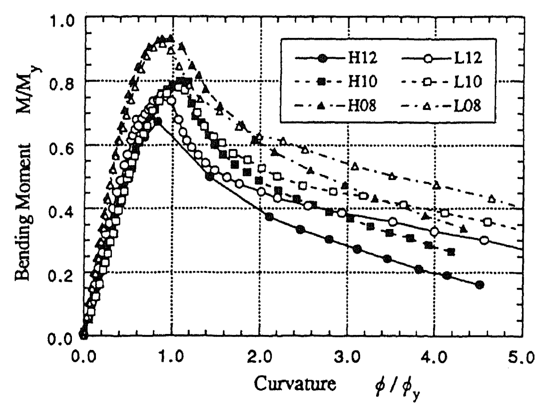

Fig. 7. Bending moment-curvature curves (H and L types)

For the effect of cross sectional shapes, M-ϕ curves of A, B and G-types are shown in Fig. 8. By using the round corners instead of sharp corners, the strength deterioration becomes insignificant. Furthermore, the strength deterioration of G-type may not be observed up to about $5\phi_y$ although the rigidity of longitudinal stiffeners used herein is only 17% of required value by new JSHB(1991). This strength and ductility of G-type is very attractive for the seismic design, compared with standard box sections with sharp corners.

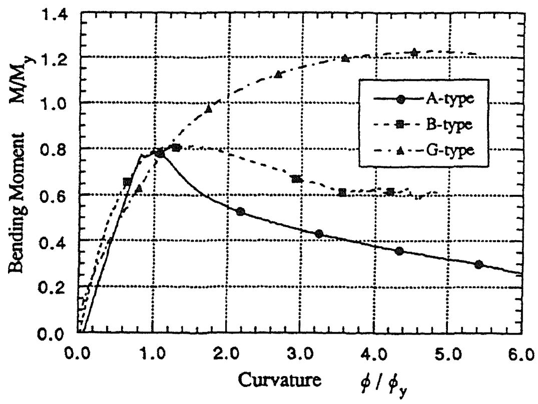

Fig. 8. Bending moment-curvature curves(A, B and G types)

4 DUCTILITY OF BRIDGE PIERS

4.1 Procedure of numerical simulation

In the previous section, the sectional load-deformation characteristics considering local instability of plate/ shell elements is evaluated experimentally. It may also be possible to obtain the whole load-deformation characteristics of bridge piers by experiments. However, it requires to install a large structural testing equipment. Therefore, in this study, the horizontal load-displacement relation is obtained through simulation taking into account the plastic hinge behavior of short thin-walled beam-column to the section at the bottom of a model of bridge pier of scale one tenth as shown in Fig.9. The height, the slenderness parameter, the yield horizontal load and the yield horizontal displacement of a model of bridge pier are listed in Table 2.

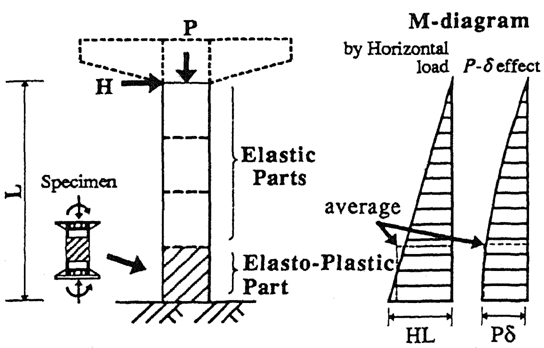

Fig. 9. Idealized model of bridge pier

Table 2. Dimensions and strength of model bridge piers

Type	Height of Column L(mm)	Slenderness Parameter* $\bar{\lambda}$	Yield Horizontal Load H_y(kN)	Yield Horizontal Displ. δ_y(mm)
H10	1163	0.4	62.4	8.31
L10	1463	0.4	50.3	9.35
A	2215	0.4	49.2	12.45
G	2112	0.4	47.4	11.33

$$^{*}\bar{\lambda}=\frac{1}{\pi}\sqrt{\frac{\sigma_y}{E}}\left(\frac{kL}{\sqrt{I/A}}\right)$$

kL: effective length of a column(k =2)

According to the survey by Nakai, et al.(1982), the slenderness of bridge piers is determined to take the slenderness parameter $\bar{\lambda}$=0.4. In this numerical simulation, the column of bridge pier is divided into 4 segments, where it is assumed that only one segment at the bottom behaves elasto-plastically(plastic hinge) and the other segments behave only elastically. The flow of this calculation is shown in Fig. 10. Firstly, the length and average curvature of the plastic hinge($L/4$ and ϕ_0) is assumed. At the next step, the horizontal displacement(δ_0) and rotation(θ_0) at the top of this segment assumed as plastic hinge is calculated by the following equation:

$$\delta_0=\frac{1}{2}\phi_0\left(\frac{L}{4}\right)^2 \tag{1}$$

$$\theta_0=\phi_0\left(\frac{L}{4}\right) \tag{2}$$

In addition to the above deformation, the average bending moment can be given by the moment-curvature relation obtained by experiments. Secondly, the elastic deformation of the upper part of the column as shown in Fig. 11 can be obtained by solving the following differential equilibrium equation:

$$EI\frac{d^4v}{dx^4}+P\frac{d^2v}{dx^2}=0 \tag{3}$$

under the boundary conditions:

$$\frac{d^2v}{dx^2}=0; \quad x=0 \tag{4a}$$

$$v=0; \quad x=0 \tag{4b}$$

$$EI\frac{d^3v}{dx^3}+P\frac{dv}{dx}=-H; \quad x=\frac{3}{4}L \tag{4c}$$

$$\frac{dv}{dx}=\theta_0; \quad x=\frac{3}{4}L \tag{4d}$$

As a result, additional horizontal displacement of top column segment is given by

$$\delta_1 = v\left(x=\frac{3}{4}L\right) = \frac{H+P\theta_0}{Pk}\tan k\left(\frac{3}{4}L\right) - \frac{H}{P}\left(\frac{3}{4}L\right) \quad (5)$$

in which ,

$$k=\sqrt{\frac{P}{EI}} \quad (6)$$

Total horizontal displacement(δ) at the top of the column can be given by summing δ_0 and δ_1. This horizontal displacement causes the additional bending moment at the base of bridge pier, so called P-δ effect. As the average bending moment is pre-determined according to the given average curvature, this calculation is necessary to be repeated until the horizontal load and displacement(H and δ) are converged. By the first and second-order elastic analyses, the horizontal displacement at the top of column can be given, respectively as follows:

1st-order elastic analysis; $$\delta = \frac{HL^3}{3EI} \quad (7)$$

2nd-order elastic analysis; $$\delta = \left(\frac{1}{k}\tan kL - L\right)\frac{H}{P} \quad (8)$$

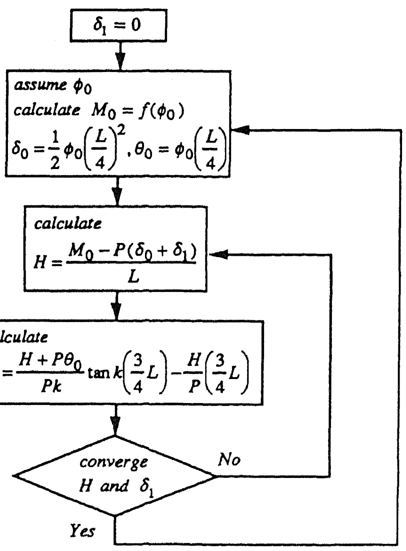

Fig. 10. Flow of numerical simulation

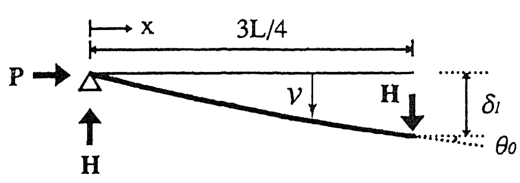

Fig. 11. Free-body diagram and boundary conditions of the upper elastic part of the column for the horizontal displacement v

4.2 Numerical results and discussions

The simulated horizontal load-displacement curves are shown in Figs. 12(H and L-types) and 13(A and G type), where the load and displacement are normalized by the yield values(H_y and δ_y). Fig.12 is drawn for H10 and L10 and the dashed line in figure is given by the integration of moment-curvature relation based on the exact stress-strain curves(fitted by the least square method and referred to Fig.6) neglecting the local instability. The difference may be significant when the horizontal displacement (δ/δ_y) becomes about 5, where the strain hardening of structural steel with low yield ratio(L type) occurs.

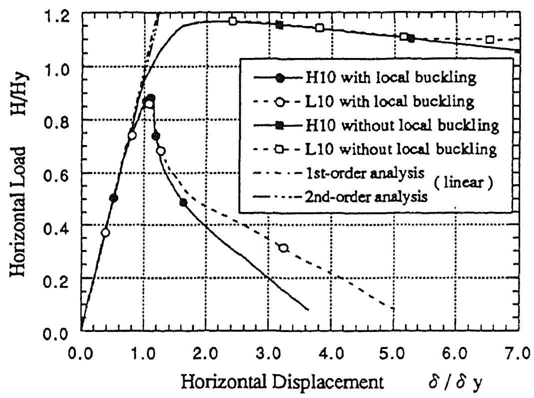

Fig. 12. Horizontal load-displacement curves(H10 and L10 types)

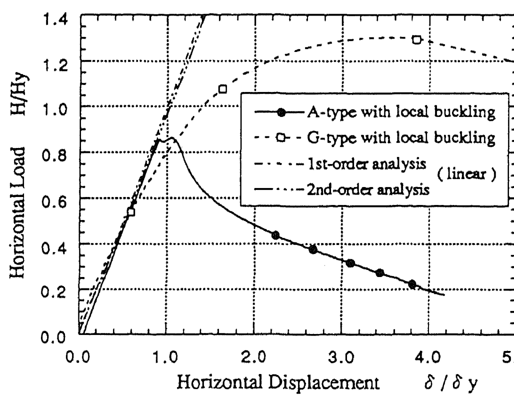

Fig. 13. Horizontal load-displacement curves(A and G types)

In addition, the numerical results considering the local instability of plate elements are shown. The slight difference of load-displacement curves can be observed even at the small horizontal displacement because the initiation of strain hardening of materials occurs earlier due to the local buckling. It is seen that the degradation of strength becomes insignificant for L-type, which is very similar to the local behavior of beam-column segments. Thus, it is concluded that the global response of bridge pier is mainly determined by the local load-deformation characteristics and that the P-δ effect accelerates the degradation. The numerical results for A, and G types are shown in Fig. 13. The significant increase of the strength and ductility of G-type is obtained. This concludes that the stiffened box section with round corners is superior to the box section with sharp corners if the cross sectional area is kept constant.

5 CONCLUSIONS

The load-deformation relationships of steel bridge piers is discussed considering the effect of the local failure characteristics. At first, the sectional inelastic behavior of thin tubular beam-column segments are investigated by two series of experiments keeping a focus on the dependency of strength and ductility on cross sectional shape and inelastic characteristics of material itself. Assuming the loading condition of the column of steel bridge piers subjected to the strong ground motions, monotonic bending tests with the constant axial compressive force are carried out, where the effect of the yield ratio and the use of round corners for box cross sections on strength and ductility are evaluated.

Furthermore, by modeling the plastic hinge behavior considering the local instability of plate elements based on experimental observations, the deformation characteristics of steel bridge piers are investigated by the numerical simulation. The results obtained from this study are summarized as follows:

1. The box section with round corners instead of sharp corners is desirable for the ductility improvement. It can avoid the cracking along the welding line at corners by smoothly allowing the propagation of buckling shapes of flange plates to web plates.
2. The use of structural steel with low yield ratio is desirable for the ductility improvement. The less strength deterioration after its peak can be expected even for the thin-walled box cross section.
3. The effect of local instability of plate elements on the strength deterioration of bridge piers is significant and the P-δ effect accelerates the degradation of strength.

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