I－A106 SEISMIC PERFORMANCE OF INVERTED L-SHAPED STEEL BRIDGE PIERS SUBJECTED TO CYCLIC IN-PLANE LOADING

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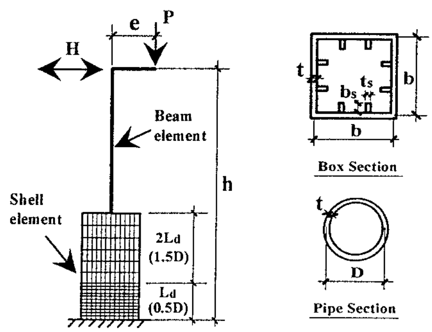
1. Introduction

A large number of inverted L-shaped steel columns have been designed and constructed to support highways in the urban of Japan. Such bridge piers provide a more broad space for the traffic flow under the expressway than that of the T-shaped piers due to its eccentric characteristic. The dead load resulting from the superstructure acts eccentrically, which in turn causes an additional bending moment. This additional moment will decrease the load-carrying capacity and ductility capacity. The piers are characterized by a relatively high plate width-thickness ratio or radius-thickness ratio, which makes them susceptible to suffer from local buckling near the base under a constant axial force and cyclic in-plane loading.

The elastoplastic large displacement finite-element formulation by using the modified 2SM [1] to model material nonlinearity has been employed to analyze the T-shaped steel box and pipe columns modeling bridge piers, and their strength and ductility prediction formulas have been proposed by authors [2, 3]. The objective of this paper aims to find out the correlations between the T-shaped and inverted L-shaped columns, and to evaluate the strength and ductility capacity of the inverted L-shaped columns.

2. Analytical Model

The analytical model is shown in Fig. 1. The load is applied at the position with an eccentric distance of e at the tip. The cross section of the cantilever beam is taken to be the same as that of the column. In the analysis, only half of the column is modeled due to the symmetry of both the geometry and loading. For the part of shell elements, in the case of steel box columns, the length between the base and the first diaphragm is divided into 12 segments while the subsequent same length is divided into 6 segments along the column length. Each subpanel and longitudinal stiffener for the cross-section are divided into 6 segments and 3 segments, respectively. The diaphragm is also simulated with shell elements. In the case of steel pipe columns, the length from the base which equals the radius of the column is divided into 15 segments, while the following length of 3 times of the radius is only divided into 10 segments along the column length. In the circumferential direction, both consist of 16 segments. For the beam elements, both the box and pipe columns are divided into 15 segments. In addition, a stiff plate with infinite bending stiffness is assumed in the interface between the beam element and shell elements. The element types employed are a two-node linear open section beam (B31OSH) with hybrid formulation and a four-node doubly curved shell element (S4R). The 2SM is used to describe the stress-strain relation of each integration point. Both these two element types account for the effect of transverse shear deformation. The initial imperfection is not taken into consideration.



(a) Analytical Model (b) Cross Section

Fig. 1 Inverted L-shaped Columns

3. Correlation between T-shaped and Inverted L-shaped Columns

To derive the horizontal load versus horizontal displacement relationship between the T-shaped and inverted L-shaped columns, 20 columns including 6 T-shaped columns are analyzed. The eccentric distance e ranges from $0.1h$ to $0.3h$. The steel types employed are JIS SS400 mild steel for pipe column and JIS SM490 mild steel for box column, respectively. For the inverted L-shaped column, with the impose of the eccentric load, an initial horizontal displacement, δ_0, will first be induced. We stipulate that the displacement of $\pm\delta_y$, $\pm 2\delta_y, \cdots$ is applied based on a reference displacement of $\delta_0/3$. Here, δ_y is the yield displacement of the T-shaped column. In the case of T-shaped columns, the displacement of $\pm\delta_y$, $\pm 2\delta_y$, $\pm 3\delta_y$, $\cdots$, is applied from the neutral position.

Fig. 2 shows the computed results of maximum strength difference between the T-shaped and inverted L-shaped columns. The specimen number is taken as the abscissa, while the maximum strength difference $|H_{c,max} - H_{e,max}|$ normalized by M_0/h is adopted as the ordinate. Here $H_{c,max}$ and $H_{e,max}$ respectively denote the maximum strengths of the T-shaped and inverted L-shaped columns in both the eccentric side and opposite side, and M_0 is the additional bending moment (i.e., $P \cdot e$). According to Fig. 2, the relationship

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between $H_{c,max}$ and $H_{e,max}$ can be written as follows:

$$H_{c,max} - H_{e,max} = \pm\frac{M_0}{h} \qquad (1)$$

Here, symbol "+" is for the case in the eccentric side, and symbol "−" denotes the case in the opposite side. To find out the relationship between H_c and H_e at an arbitrary loading state, a figure similar to Fig. 2 is plotted in Fig. 3. It should be noted that the normalized horizontal displacement, δ_c/δ_y, is taken as the abscissa, and the value of $|H_c - H_e|/(M_0/h)$ (designated as parameter ϕ in the subsequent text) is taken as the ordinate. Here, H_c represents the horizontal load at displacement δ_c, while H_e refers to the horizontal load at displacement δ_e. Five different values of $(\delta_e - \delta_c)$, that is, 0, $\delta_0/3$, $\delta_0/2$, $2\delta_0/3$, and δ_0, are considered. Except for the case of $\delta_e - \delta_c = \delta_0/3$, the results of the other four cases are computed from the envelope curves. Comparison of the strength differences corresponding to different values of $(\delta_e - \delta_c)$ indicates that: (1) In the elastic range, the difference of the value of ϕ becomes quite noticeable. In the case of $\delta_e - \delta_c = 0$ and $\delta_e - \delta_c = \delta_0$, the value of ϕ is, respectively, about 1.5 and 0, which coincide well with the theoretical solution. (2) In the plastic range, the value of ϕ does not vary too much with the change of $\delta_e - \delta_c$. (3) It is seen that for the second case, the value of ϕ is always around unity in the whole range. In constrast, the other four cases fail to be valid in both the elastic and inelastic ranges. Accordingly, the strength and displacement correlations between the inverted L-shaped and T-shaped columns can be expressed as follows:

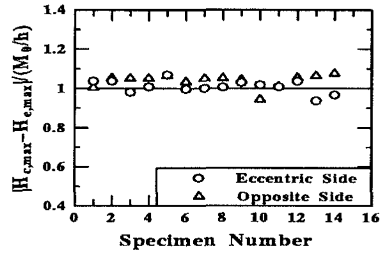

Fig. 2 Comparison of Maximum Strength Difference

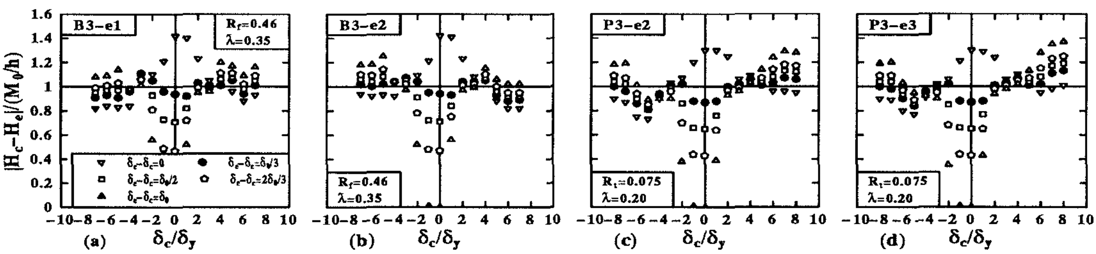

Fig. 3 Comparison of Strength Difference

$$H_c - H_e = \pm\frac{M_0}{h}; \quad \delta_e - \delta_c = \frac{\delta_0}{3} \qquad (2)$$

Comparisons of the envelope curves of the analyzed columns are shown in Fig. 4. The real line refers to the predicted result obtained from the envelope curve of the T-shaped column. It is observed that the predicted curves at each plot show a good agreement with the analytical results of the inverted L-shaped columns in both the elastic and inelastic ranges, which implies that the proposed correlation is quite convincing. In addition, to compare the hysteretic curves of the inverted L-shaped columns with those of the T-shaped columns, we choose the displacement difference of $\delta_0/3$ as the reference displacement. Otherwise, some displacement lag phenomenon will take place when we plot the normalized hysteretic curves of both the T-shaped and inverted L-shaped columns together.

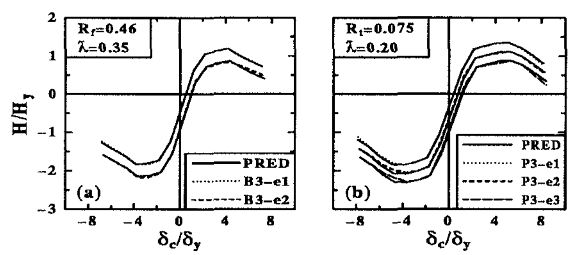

Fig. 4 Comparison of Envelope Curves

According to the proposed Eq. (2), the ultimate strength and ductility of the inverted L-shaped columns can be conveniently obtained from those of the T-shaped steel box and pipe columns modeling bridge piers.

4. Conclusions

Based on the analytical results, the following conclusions can be obtained: 1) With the increase in eccentric distance, the load-carrying capacity of the inverted L-shaped columns in the eccentric side is greatly decreased, while which in the opposite side is the reverse. 2) A definite horizontal load - horizontal displacement relationship between the T-shaped and inverted L-shaped columns has been proposed.

5. References

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