I－A76 DUCTILITY EVALUATION OF STEEL BOX BRIDGE PIERS UNDER CYCLIC LOADING

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1. Introduction

Steel bridge piers of box sections have widely been designed and constructed to support highways in the urban areas of Japan. Such cantilever type of bridge piers occupies less space and thus allows the free traffic flow in the roads below the highways. Aseismic design of such steel bridge piers is very important for the urban transportation network, which became much clearer after the 1995 Hyogoken-Nanbu Great Earthquake. The ability of such piers to survive severe earthquakes depends on both the strength and ductility of the columns. Up to now, evaluation of the strength and ductility of steel box bridge piers is mainly limited to tests. With the rapid development of computer technique, finite element method accounting for both geometric and material nonlinearities is becoming more and more popular. Its accuracy greatly depends on the exactness of material model employed.

The objective of this paper is to evaluate the strength and ductility capacity of steel box bridge piers by using the 2SM [1] for material nonlinearity. The analytical result is compared with the experimental result. As a result, empirical formulas are proposed to evaluate the strength and ductility capacity of steel box bridge piers.

2. Analytical Model

For thin-walled steel columns of uniform box sections subjected to a constant axial load and cyclic lateral loads, local buckling always occurs near the base of the columns [2]. Therefore, beam-column elements are employed for the upper part of the column, while shell elements which can consider the local buckling are used for the lower part of the column, as shown in Fig. 1.

In the analysis, only half of the column is modeled due to the symmetry of both geometry and loading. For the part of shell elements, the length between the base and the first diaphragm is divided into 12 segments while the subsequent same length is subdivided into 6 segments along the column length. Each sub-panel and longitudinal stiffener for the cross-section are cut into 6 segments and 3 segments, respectively. With respect to the diaphragm, it is also simulated with shell elements. In addition, a stiff plate with infinite bending stiffness is assumed in the interface between beam-column element and shell elements where a diaphragm is located. The modified Newton iteration technique coupled with the displacement control method is used in the analysis. Details of the solution procedure can be found in the ABAQUS theory manual [3]. The initial geometrical deflections and residual stresses are not taken into consideration.

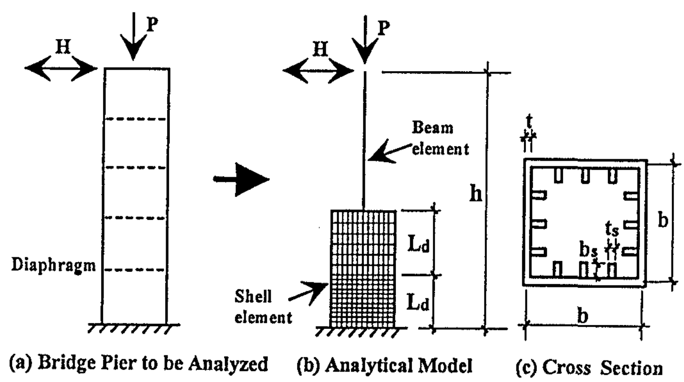

Fig. 1 Box-section Column

3. Comparison of Analytical and Experimental Results

In this section, a cyclic analysis of a tested steel box column [2] is carried out and the computed results are compared with that of the experiment. The column has a width-thickness ratio parameter of $R_f = 0.56$, column slenderness ratio parameter of $\bar{\lambda} = 0.26$, stiffener's slenderness ratio parameter of $\bar{\lambda}_s = 0.63$. Geometrical properties of the column are h = column length = 3403 mm; b = flange width = 882 mm; $L_d = \alpha \cdot b$ = distance between adjacent diaphragms = 882 mm, in which α = ratio of adjacent diaphragm distance to flange width = 1.0. In addition, the thickness of flange and web plates t is 9.0 mm, while the stiffener thickness t_s and stiffener width b_s are 6 mm and 80 mm, respectively. Material properties of the column are yield stress $\sigma_y = 379$ Mpa; elastic modulus $E = 206$ Gpa; Poisson's ratio $\nu = 0.3$; initial hardening modulus $E_{st} = E/30$; strain at the onset of strain hardening $\varepsilon_{st} = 10\,\varepsilon_y$. During the test, the column is subjected to a constant axial load of $P/P_y = 0.124$ and cyclic lateral displacement. Here, P_y denotes the squash load.

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Keywords: Steel box bridge piers, Cyclic loading, Ductility evaluation

The curves of nondimensionalized lateral load, H/H_y, versus lateral displacement, δ/δ_y, from both the experiment and 2SM analysis are shown in Fig. 2. Here, H_y and δ_y are defined as:

$$H_y = \frac{M_y}{h}(1 - \frac{P}{P_y}) \qquad \delta_y = \frac{H_y h^3}{3EI} \tag{1}$$

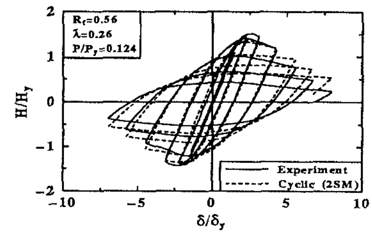

Fig. 2 Comparison of Analysis and Test

It can be observed that the shape of the hysteresis loops from present analysis is in good agreement with the experimental result. This indicates that the developed FEM formulation based on the 2SM can accurately predict the cyclic behavior of such columns due to its accurate description of material behavior and proper consideration of local buckling, .

4. Strength and Ductility Evaluations

In order to evaluate the strength and ductility of box section columns, a series of columns are analyzed using the 2SM to account for material nonlinearity. The parameters considered are plate width-thickness ratio, column slenderness ratio, stiffener's equivalent slenderness ratio as well as magnitude of axial load. The plate width-thickness ratio parameter, R_f, ranges from 0.25 to 0.50 and column slenderness ratio parameter, $\bar{\lambda}$, lies between 0.20 and 0.50. The axial load applied, P, is in the range of $0.10P_y$ to $0.30P_y$. The thickness of both plate and stiffeners are assumed as 20 mm. The columns are made of SM490 steel with properties of $\sigma_y = 314$ Mpa; $E = 206$ Gpa; $\nu = 0.3$; $E/E_{st} = 30$; $\varepsilon_{st}/\varepsilon_y = 7$. All the columns are loaded with one cycle of lateral displacement reversal at each displacement level (i.e., $\pm\delta_y$, $\pm 2\delta_y$, $\cdots$).

Ductility capacity is an important consideration in aseismic design. Two ductility factors, δ_m/δ_y and δ_{95}/δ_y, are employed to evaluate the deformation capacity. Here, δ_m is the displacement corresponding to the maximum lateral load, and δ_{95} is the displacement corresponding to 95% of maximum load after peak load.

Computed maximum load H_{max}/H_y, ductility factors δ_m/δ_y and δ_{95}/δ_y of the box columns are plotted against the multification of parameter R_f, $\bar{\lambda}$, $\bar{\lambda}_s/\sqrt{\alpha}$ in Fig. 3. The equations that provide satisfactory predictions to the computed results are fitted as follows:

$$\frac{H_{max}}{H_y} = \frac{0.10}{(R_f\ \bar{\lambda}\ \bar{\lambda}_s/\sqrt{\alpha})^{0.5}} + 1.07 \qquad (S = 0.08) \tag{2}$$

$$\frac{\delta_m}{\delta_y} = \frac{0.22}{R_f\ \sqrt{\bar{\lambda}}\ \bar{\lambda}_s/\sqrt{\alpha}} + 1.31 \qquad (S = 0.50) \tag{3}$$

$$\frac{\delta_{95}}{\delta_y} = \frac{0.25}{R_f\ \sqrt{\bar{\lambda}}\ \bar{\lambda}_s/\sqrt{\alpha}} + 1.90 \qquad (S = 0.58) \tag{4}$$

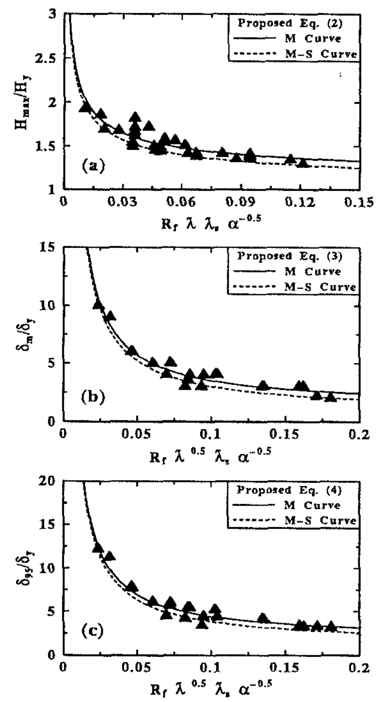

Fig. 3 Proposed Formulas

It should be noted that the real line in each figure denotes the fitted curve with average values, while the dashed line represents the curve which is lower than the real line with a constant distance of S. Here, S is the standard deviation. It is seen that the curves well represent the tendency with the variety of main parameters. The proposed equations can be used to determine the ductility capacity of box section steel columns in aseismic design.

5. Conclusions

The results of cyclic elastoplastic large displacement analysis of steel box columns modeling bridge piers were presented. Based on the analytical results, the following conclusions can be drawn: 1) The predicted hysteretic curve of the tested column using the 2SM is in good agreement with the experimental result. 2) The formulas are proposed for determining the strength and ductility capacity of steel box bridge piers.

6. References

[1] Shen, C. et al.: Cyclic behavior of structural steels. II: Theory. *J. Engrg. Mech.*, ASCE, 121(11), 1165-1172, 1995.

[2] Nishikawa, K. et al.: An experimental study on improvement of seismic performance of existing steel bridge piers. *J. of Struct. Engrg.*, JSCE, 42A, 975-986, 1996.

[3] ABAQUS/Standard User's Manual. Ver. 5.5, 1995.