Advances in Computational & Experimental Engineering & Science

One Dimensional Nonlinear Analysis of Steel Tubular Columns

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Summary

One-dimensional model was proposed for static and dynamic nonlinear analysis of steel tubular bridge piers. The present model does not require the relationships between loads and displacements, which have been obtained by experimental works or shell analysis. The model dimensions such as thickness, height and radius, and material properties of tubular piers are required for static and dynamic analysis of the present model. The present analysis consists of two stages. The first stage is to obtain the stress and strain relationships in the base plates, where local bending buckling was observed. The second one is to analyze the overall behavior of tubular steel bridge piers. The validity of the present model was confirmed through comparisons between the existing experimental results and the present numerical results.

Introduction

Severe earthquake will, in most case, lead to inelastic behavior in conventional civil engineering structures. Great damages of steel bridge piers have been observed in the 1995 Kobe Earthquake. Since the great earthquake, various experimental and analytical researches have been conducted to examine the ultimate strength and ductility of steel bridge piers. A shell analysis has been used to compare the numerical results with the experimental results. Since the shell analysis is expensive, some single degree of freedom models have been proposed. The crucial point for 1-D analysis is the constitutive equations used, which account for the local buckling of steel piers. The enveloped curves, obtained by experiments or shell analysis, have often been used to establish the constitutive equations. Therefore, when no experimental results are available, the shell analysis is indispensable for establishing the constitutive equations for 1-D analysis.

In this paper, as a first step, the constitutive equations for 1-D analysis are established by analyzing the modified Shanley model. Not experiments nor shell analysis are needed for establishing the constitutive equations. The enveloped curves, which take into account the local buckling, are used only in the compression side of constitutive equations for piers. The tension side of constitutive equations is assumed to be tri-linear.

The second step is to analyze the overall behavior of steel piers. The constitutive equations, obtained in the first step, are introduced into the base plates of steel piers. The height of base plates is determined by the equations proposed in this paper. The material and geometrical nonlinearities are taken into account in the base plates. The upper parts

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of steel piers are modeled by an elastic beam, in which the geometrical nonlinearities are taken into account.

The static and pseudo-dynamic experimental results are used to confirm the validity of the present model.

One-Dimensional Model

The present model for steel piers is shown in Fig.1. The cross-sections of steel bridge piers are circular. The model for piers consists of base plates with height L_2 and elastic beam with height L_1. The base plates are fixed at the bottom of piers, while the axial and lateral loads are applied at the top of piers.

As a first step, the constitutive equations for the base plates are obtained. The model, as shown in Fig.2, is used to establish the constitutive equations. The height L_2 depends on the material and is determined by the following equations:

$$L_2 = \left\{ 0.182 \left(1 - {P_v}/{A_0 \sigma_y} \right)^{0.5} \left(\frac{D}{t^*} \right)^{0.5} \left(\frac{L}{D} \right) \left(\frac{I_0}{I} \right) + 2.94t \right\} \quad \text{for SS400} \tag{1}$$

$$L_2 = \left\{ 0.109 \left(1 - {P_v}/{A_0 \sigma_y} \right)^{0.5} \left(\frac{D}{t^*} \right)^{0.5} \left(\frac{L}{D} \right) \left(\frac{I_0}{I} \right) + 3.74t \right\} \quad \text{for SM400 and SM490} \tag{2}$$

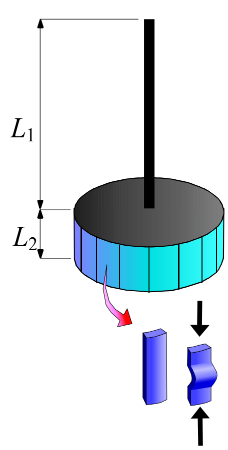

Fig.1 Steel Pier

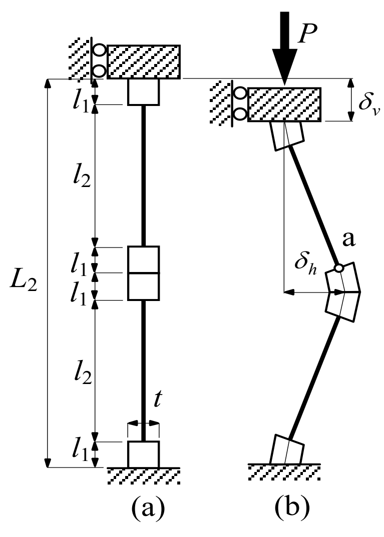

Fig.2 Base Plate Model

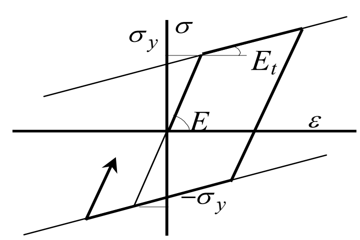

Fig.3 Bi-linear Model

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where P_v is the initial axial load, A_0 cross-sectional area, σ_y yield stress, $t*$ modified thickness defined in Ref.(1), I moment inertia with vertical stiffeners, I_0 moment inertia without vertical stiffeners, D diameter of pier, L height of pier, and t thickness of tubular pier. In Fig.2 we assume that $l_1 = t$. The four quadrates with $l_1 \times t$ consist of fiber elements, whose constitutive equation is shown in Fig.3. This fiber has only axial stiffness. The number of fiber elements used is 100. The two beams with length l_2 are assumed to be in-elastic, the constitutive equation of which is shown in Fig.3. The material and geometrical nonlinear analysis is conducted to obtain the relationships between the axial load P and the deflection δ_v. The relationships obtained are used as the constitutive equations of base plates. The ratio E / E_ξ is assumed to be 100.

The typical relationships between axial load and axial displacements are shown in Fig. 4. Due to the buckling, the softening effects are observed in the relationships. The number of fiber elements is changed from 6 to 100 in order to check the convergence of maximum loads. The convergence of maximum load is attained when 20 elements are used. In this paper, 100 fiber elements are used for keeping a numerical precision. The curves obtained are used as the constitutive equation in the compression side of steel piers. The constitutive equation of the tension side is assumed to be tri-linear.

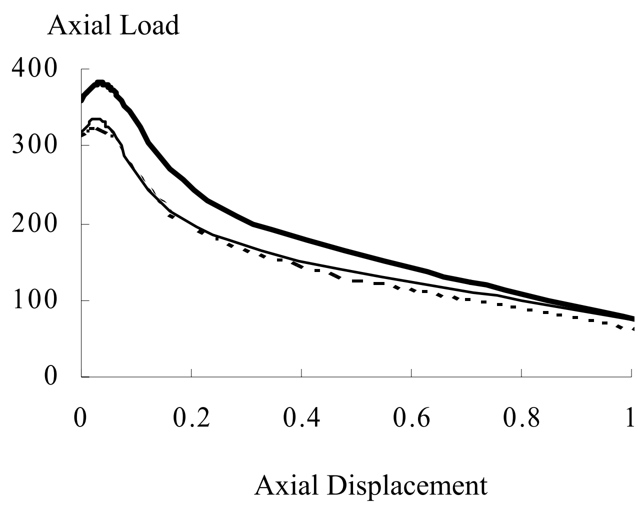

Fig.4 Relationships between P_v and δ_v

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The overall analysis of steel piers is conducted based on the model as shown in Fig.5. Figure 5(b) shows the deformed state of steel pier model. The axial load corresponds to the dead load of superstructures of bridge, and is assumed to be constant. The lateral load corresponds to the earthquake load. The base plate with height L_2 consists of fiber elements, the constitutive equation of which is shown in Fig.6. The elastic beam with height L_1 has axial and bending stiffness. The connection between elastic beam and base plate is assumed to be rigid; the Bernoulli-Euler hypothesis holds.

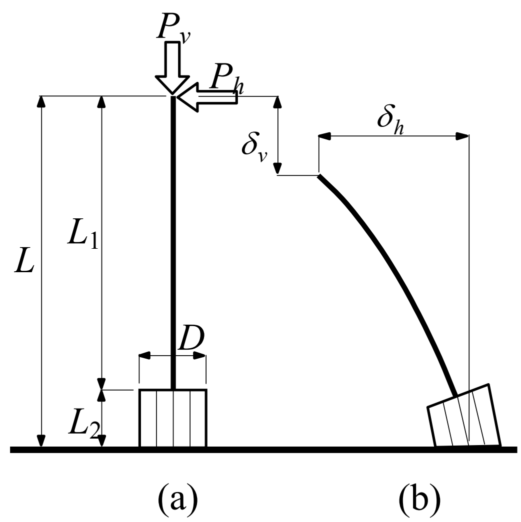

Fig.5 Overall Model

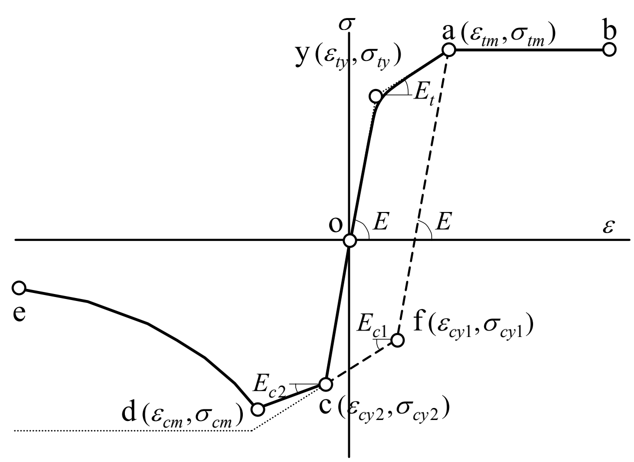

Fig.6 Constitutive Equations for Overall Model

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Numerical Results

The static numerical results are shown in Figs.7 and 8, in which the experimental results by Ishizawa and Iura [1)] are plotted by dotted linen while the numerical ones by solid lines. The model in Fig.7 is subjected to constant initial axial load and cyclic lateral load. The present numerical model detects the softening effects after peak load. A good agreement between experimental results and numerical ones are observed in Fig.7. The load history in Fig.8 was taken from The Kobe Earthquake in 1995, in which a strong earthquake came first. According to the experiment, a local buckling occurred at the base plate at first load cycle. The difference between experimental results and numerical ones are small.

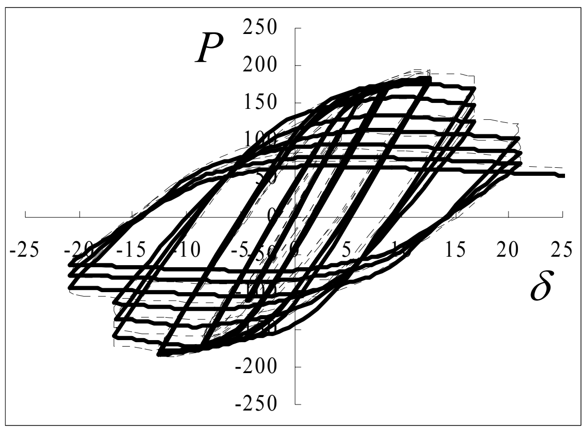

Fig. 7 Load-Displacement Curves

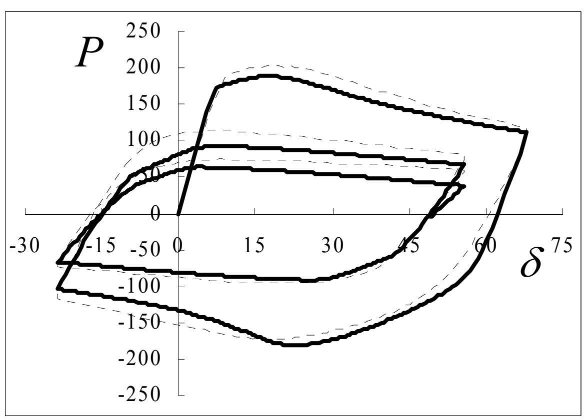

Fig.8 Load-Displacement Curves

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The pseudo-dynamic tests were conducted in Japan[2)]. The earthquake wave used in the test was recorded by Japan Meteorological Agency in 1995. The comparisons between numerical results and experimental ones are shown in Fig.9. The model in Fig.9(a) is the same as that of Fig.9(b). The height of pier is 1945mm, diameter 400mm and thickness 4.5mm. The model in Fig.9(a) was subjected to the actual recorded waves, while the model in Fig.9(b) was subjected to 1.5 times of the recorded waves. Therefore, the remarkable residual displacement is observed in Fig.9(b). The model in Fig.9(c) has the same sizes as that of Figs. 9(a) and 9(b), except the thickness of steel pier. The model of Fig. 9(c) has 6mm thickness. The actual recorded wave was used. In contrast with the model in Fig.9(a), the residual displacement was observed in Fig.9(c). A good agreement between pseudo-dynamic tests and numerical results are obtained. These comparisons confirm the capability and the validity of present numerical model.

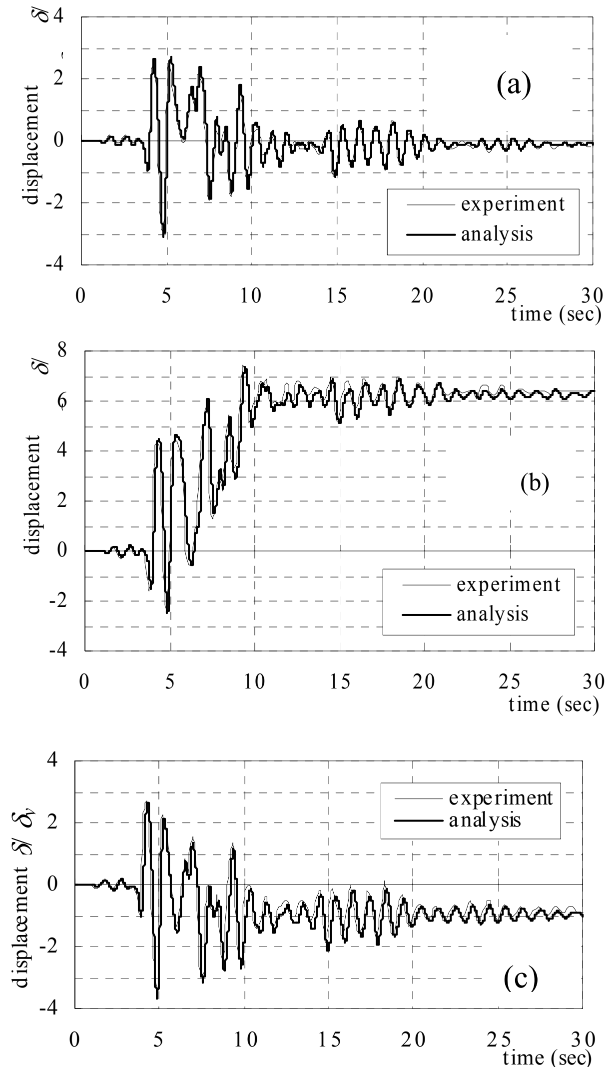

Fig.9 Time History of Displacements

References

1 Ishizawa, T. and Iura, M. (1998): ``Experimental study on ultimate strength and softening effects of steel tubular bridge piers'', *Journal of nonlinear analysis of Steel Structures*, Vol. 2, pp. 115-120.

2 Joint Research Report on Limit State Seismic Design of Highway Bridge Piers (VII) , Public Works Research Institute, 1997.