Thin-Walled Structures 49 (2011) 544–553

Contents lists available at ScienceDirect

Thin-Walled Structures

journal homepage: www.elsevier.com/locate/tws

Behaviour of steel columns under 3-D seismic load

Shigeru Shimizu

Department of Civil Engineering, Shinshu University, Wakasato 4-Chome, Nagano 380-8553, Japan

ARTICLE INFO

Available online 28 September 2010

Keywords:
Seismic behaviour
Steel box column
Vertical component

ABSTRACT

In the current Japanese bridge design code, the seismic load of only one direction (1-dimension, 1-d) is considered. However, in fact, an actual seismic load has three components for N–S (north–south), E–W (east–west) and U–D (up–down) directions. The current paper presents a series of numerical results of steel box columns, which are used as piers of viaduct, under 3-d seismic load with 3 components. Two types of ground motion patterns of the main shock and the maximum aftershock measured in the Chuetsu Earthquake in 2004 are used as the seismic load. The results indicate that the severer deformation arises in the columns under 2-d (N–S and E–W components) or 3-d seismic load than the 1-d load by the main shock. However, under the aftershock, which contains ground motion of higher frequency, 1-d load gives larger displacement.

1. Introduction

This paper describes the results of a dynamic analysis on steel box columns used as a motorway viaduct under the seismic loading.

The ground motion caused by an earthquake has three components for N–S (north–south), E–W (east–west) and U–D (up–down) directions. Therefore, when an earthquake arises, a structure built on a seismic area is simultaneously subjected to 3-dimensional seismic load with the N–S, E–W and U–D components. However, in the current Japanese bridge design code, these three components are considered independently, and therefore a structure is assumed to be subjected to the seismic load of only one direction at one time.

The author has been made a series of numerical studies on the behaviour of partially concrete filled steel box columns under the seismic load [1–6]. In author's previous studies [4–6], conforming to the Japanese current design code, the ground motion for only one direction is considered. However, to clarify the actual behaviour of a structure caused by an earthquake, three components of the ground motion should be considered. Thus, in the current paper, a numerical analysis is made on the dynamic behaviour of concrete filled steel box columns under the seismic load having three components.

2. Numerical models

2.1. Geometry and materials of the models

A typical numerical model analyzed in this paper is a square section steel box column filled with concrete at its bottom part, which is often used as a pier of the motorway viaduct. Fig. 1 shows a typical numerical model. Within this figure, the global view of the model is presented in Fig. 1(a), and in (b) and (c) the section shapes are illustrated. This model is same to that used in author's previous studies. The model has its depth of 10 m and the 3 m $\times$ 3 m square section, and is composed of the steel plates with the thickness of 20 mm. The concrete is filled for 3 m from its bottom. In some cases, stiffeners with 200×20 mm^2 section are considered inside the box as shown in the Fig. 1(c). The plates forming the column without the stiffener have the dimension of 2960×20 mm^2.

For the elasto-plastic analysis, materials are assumed to have the tri-linear stress–strain relations. Material properties used in the current study are as follows:

For the steel plates, grade SM490 steel is assumed, which has the nominal yield stress of 315 MPa. Initial Young's modulus E of steel is $E=200$ GPa. After yielding, considering strain-hardening, the tangent modulus of the stress–strain relation is assumed to be $E/100=20$ GPa, and beyond the stress of 490 MPa, the tangent modulus is $E/200=10$ GPa.

For the concrete, initial Young's modulus is $E=30$ GPa, and tangent modulus after yielding is $E/10=3$ GPa. Beyond the stress of 240 MPa which is the design concrete strength, the tangent modulus is $E/20=1.5$ GPa.

With these width and thickness of plates and the material properties, non-dimensional width–thickness ratio R of plates composing the column and the non-dimensional slenderness ratio λ of the column can be calculated, here R and λ are defined as

$$R = \frac{b}{t}\sqrt{\frac{12(1-\nu^2)\sigma_y}{k\pi^2 E}}$$

$$\lambda = \frac{1}{\pi}\sqrt{\frac{\sigma_y}{E}}\frac{2l}{r}$$

E-mail address: shims00@shinshu-u.ac.jp

0263-8231/$ - see front matter © 2010 Elsevier Ltd. All rights reserved.
doi:10.1016/j.tws.2010.09.016

where b denotes the plate width, t the plate thickness, E, v, σ_y are Young's modulus, Poisson's ratio and yield stress of the material, k denotes the buckling coefficient of the plate, l the column length (depth) and r denotes the radius of gyration of area of the column. Substituting above plate dimensions, the material properties mentioned later and the buckling coefficient $k=4.0$ into these formulae, R and λ are calculated as $R=3.09$ and $\lambda=0.208$ for the model without the stiffener, and $R=1.02$ and $\lambda=0.204$ for the model with the stiffeners. With these values of R and λ, the column shall fail with local buckling of the column walls and not with the global column buckling.

At the top of the column, a mass of 700 t is considered which refers to the superstructure of the viaduct.

The model is discretized with the 20-nodes solid elements for both the steel and the concrete. The FEM mesh pattern used in this analysis is illustrated in Fig. 2. In this figure, the thick line indicates that the concrete is filled up to this level. The analysis is made for the 4 cases; (1) the model with stiffeners and filled with concrete, (2) the model with stiffeners and not filled with concrete, (3) the model without stiffener and filled with concrete, (4) the model without stiffener and not filled with concrete.

Poisson's ratio v and density μ of steel are 0.3 and 7.85×10^{-6} kg/mm^3, respectively, and those of concrete are $v=0.167$ and $\mu=2.35\times10^{-6}$ kg/mm^3. The elasto-plastic large deflection analysis is performed with the combined hardening rule considering the Bauschinger effect.

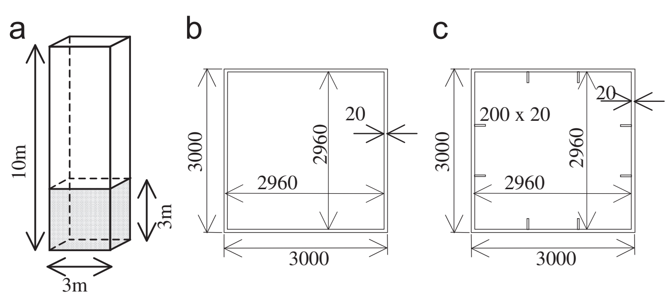

Fig. 1. Typical numerical model. (a) Column outline, (b) column section without stiffener and (c) column section with stiffeners.

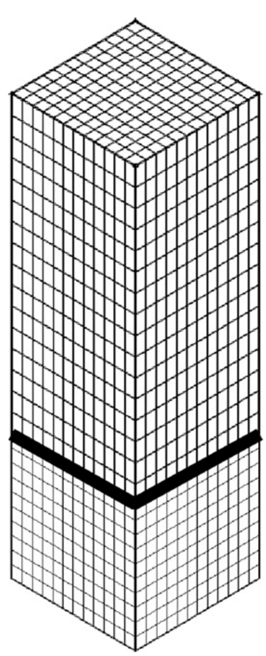

Fig. 2. FEM mesh pattern.

2.2. Natural periods and damping factors

In Table 1, natural period and natural frequency of the numerical models are summarized. Table 1 is obtained from the FEM analysis as an eigenvalue problem.

With the first mode, the model columns show the overall-bending mode, and with the second mode, plates consisting the columns deform for their out-of-plane directions without column overall bending, as illustrated in Fig. 3. With the first mode, the column shall sway with the frequency of from 2.4 to 3.2 Hz, and the stiffeners or the filled-in concrete seems to have small effect. On the other hand, in the second mode, the models without stiffener have the frequency of 6.1–6.9 Hz and with stiffeners 9.9–11.2 Hz.

Table 1
Natural period and natural frequency.

	No stiffener		With stiffeners	
Mode	No concrete	With concrete	No concrete	With concrete
First	0.415 (2.410)	0.316 (3.165)	0.394 (2.538)	0.310 (3.226)
Second	0.165 (6.061)	0.145 (6.897)	0.101 (9.901)	0.089 (11.24)

Natural period (S) and natural frequency (Hz, in parentheses).

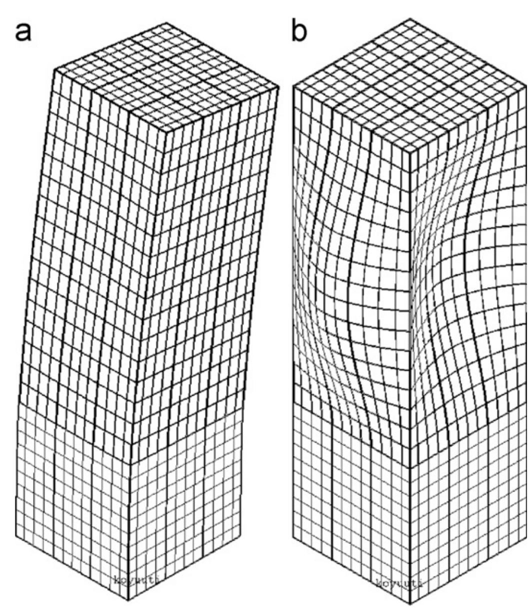

Fig. 3. Examples of natural mode of columns with stiffeners and filled with concrete. (a) 1st mode and (b) 2nd mode.

Table 2
Factors α and β.

	No stiffener		With stiffeners	
Coefficients	No concrete	With concrete	No concrete	With concrete
α	0.434	0.545	0.507	0.631
β	7.516×10^{-4}	6.334×10^{-4}	5.126×10^{-4}	4.399×10^{-4}