COMPDYN 2015
5th ECCOMAS Thematic Conference on
Computational Methods in Structural Dynamics and Earthquake Engineering
M. Papadrakakis, V. Papadopoulos, V. Plevris (eds.)
Crete Island, Greece, 25–27 May 2015

3-D DYNAMIC ANALYSIS OF BRIDGES WITH ROCKING ISOLATION UNDER EXTREME EARTHQUAKES

T.Y. Lee[1], K.J. Chung[2], and H.Y. Lee[2]

[1] National Central University
Taoyuan County, Taiwan 32001, R.O.C.
tylee@ncu.edu.tw

[2] National Central University
Taoyuan County, Taiwan 32001, R.O.C.
{100322016,102322028}@cc.ncu.edu.com.tw

Keywords: Dynamic Analysis, Ultimate State, Bridge, Spread Foundation, Rocking Mechanism, Soil Spring.

Abstract. *Rocking response of spread foundation has been found to be a unique isolation system in bridge structures. Through the extension of structural periods due to rocking mechanism, the induced seismic force can be decreased. In addition, the underlying soil may go through plastic behavior and dissipate more energy under extreme earthquakes. However, the stability of the bridges with rocking response is generally concerned by practical engineers. This study is aimed to analyze the ultimate state of bridges with rocking response subjected to extreme ground motions by using 3D Vector Form Intrinsic Finite Element (VFIFE). The VFIFE, a new computational method, is adopted in this study because it has the superior in managing the engineering problems with material nonlinearity, discontinuity, large deformation and arbitrary rigid body motions of deformable bodies. Nonlinear behavior of soil is taken account in the interaction between soil and foundation. Nonlinear soil springs are developed in the VFIFE including vertical soil springs, lateral passive soil springs and friction force soil springs. All developed soil springs are verified to be feasible and accurate through numerical simulation. Finally, a five-span continuous bridge is analyzed to investigate the effectiveness of rocking isolation of bridges with spread foundation. The simulation results show that the induced seismic force decreases as the rocking response increases. Also the unstable response of the whole bridge does not occur under the extreme earthquakes.*

1 INTRODUCTION

Spread foundations are widely used in the substructure of bridges which are located in soil conditions with high bearing capacity. From the post-earthquake investigation, it is often observed that cracks occurred along the sides of spread foundations on the ground surface after strong earthquakes. Such cracks imply that the rocking response occurred in spread foundations during earthquakes. In the past studies, rocking response can be regarded as an isolation effect on bridge response. Since the natural periods of bridges elongate due to the rocking of foundations, the induced-seismic forces decrease. However, bridges are generally designed based on design codes with the regulations which restrain the sliding, overturning and settlement of foundations. Since bridge engineers concern the issue of overturning instability, the rocking isolation of spread foundations has not been popular in practical implementation. Therefore, this research is aimed to clarify the overturning instability and the isolation effect through ultimate numerical simulation when rocking response occurs in spread foundations.

Lately, modern bridge seismic design has been developed toward the seismic performance-based design on whole bridges as well as their elements. Understanding of the performance of the components of bridges, such as bearings, unseating prevention devices, columns, and the effect of soil-structure interaction under extreme condition shall be favorable to determine the goal of performance. The Vector Form Intrinsic Finite Element method (VFIFE), a new computational method developed by Ting et al. [1], is superior in managing highly nonlinear engineering problems even with fracture and collapse. Thus the VFIFE is used to simulate the dynamic behavior of the bridges with spread foundations under extreme earthquakes in this study.

Soil-structure interaction has been investigated for several decades. The superstructures and substructures of bridges may exhibit nonlinear behavior simultaneously under extreme earthquakes. In this paper, Winkler-based models proposed by Boulanger et al. [2] are adopted to develop nonlinear soil elements in the VFIFE for the analytical study on the ultimate state of bridges with soil-structure interaction.

A practical five-span bridge with spread foundations is analyzed in this study to predict the failure process. The bridge is subjected to near-field ground motions recorded at JMA Kobe in the 1995 Japan Kobe earthquake in simulation. The ground acceleration is amplified from 100% to 300% at an increment of 10%. The simulation results clarify the seismic performance for each structural element under extreme earthquakes. Besides, the optimum design parameters of bridges with spread foundations can be obtained to increase the seismic capacity of bridges under extreme earthquakes.

2 VECTOR FORM INTRINSIC FINITE ELEMENT

The Vector Form Intrinsic Finite Element (VFIFE) is developed based on theory of physics to mainly simulate failure responses of structural systems subjected to extreme loads. To analyze a continuous structural system by using the VFIFE, a lumped-mass idealization is first performed to construct a discrete model. All lumped masses are then connected by deformable elements which are massless and exhibit resisting forces during deformation. Applying Newton's Second Law of Motion, the equations of motion are assembled at each mass for all degrees of freedoms. Assume that a structural system consists of a finite number of particles. The equations of motion for a particle are written as

$$\mathbf{M}^{\alpha}\ddot{\mathbf{d}}^{\alpha}(t) = \mathbf{P}^{\alpha}(t) - \mathbf{f}^{\alpha}(t) \tag{1}$$

where $\mathbf{M}^{\alpha}$ is the diagonal mass matrix of the particle α and $\mathbf{d}^{\alpha}(t)$ is the displacement vector when the particle α is at time t ; $\mathbf{P}^{\alpha}$ is the vector of applied forces or equivalent forces acting on the particle; $\mathbf{f}^{\alpha}$ is the vector of the total resistance forces or internal resultant forces exerted by all the elements connecting with this particle.

It is noted that each massless element is assumed to be in static equilibrium. Observed from Eq. (1), the VFIFE analysis is exempted from the assemblage of the global stiffness matrix for structures consisting of elements with multiple degrees of freedom. Therefore, a matrix algebraic operation for the entire system is not required. Instead, each equation of motion for each particle, Eq. (1), can be individually solved. Since the failure of structures involves changes in material properties and structural configuration, it is necessary to use discrete time domain analysis to solve the equations of motion. The central difference method, an explicit time integration method, is thus selected in the VFIFE to solve the equations of motion, Eq. (1).

Compared to the traditional finite element method, the unique of the VFIFE is that the element internal forces are calculated by using the element deformations obtained through subtracting rigid body displacements from total displacements. A set of deformation coordinates is defined for each element in each time increment to calculate the element deformations. Therefore, the VFIFE is capable of dealing with structural dynamic problems with large displacements, deformations and rigid body motion simultaneously.

When subjected to extreme earthquakes, bridges may undergo highly nonlinear behavior even structural failure. In the past large earthquakes, some bridges suffered deck unseating. Generally, deck unseating follows high material nonlinearity, geometric nonlinearity as well as rigid body motion. To simulate the ultimate states of bridges, the failure mechanism of major bridge components should be taken into account.

The studied failure components are isolators, unseating prevention devices and plastic hinges of decks and columns. Firstly, isolators are idealized as a bilinear model. Once the resistance force of an isolator reaches the designated rupture strength, the isolator fractures and then no longer exerts resistance shear force. Assume that the isolator becomes a sliding-like bearing and friction force exists on the fractured surface. When the relative displacement between superstructure and column exceeds the unseating prevention length, the superstructure will lose the supporting force provided by the column and then fall down from the cap beam due to the gravity force. The failure of isolators represents a typical failure mechanism completing material linear and nonlinear behavior, fracture, and sliding of structures. Such elements in the VFIFE have been developed in the previous studies. [3]

After an isolator ruptures, the interface between the superstructure and the column turns to a sliding surface if the relative displacement between the superstructure and the column is less than the unseating prevention length. The motion on the sliding surface can be separated into stick and slip phases. When the friction force is smaller than the maximum static friction force, there is no relative motion in the interface, namely in stick phase. Once the friction force overcomes the maximum static friction force, relative movement starts in the interface and the friction force converts to dynamic friction force, namely in slip phase. In this study, assume that the maximum static friction force is equal to the dynamic friction force, and the dynamic friction coefficient remains constant during sliding.

In the calculation process of the VFIFE, the material properties and structural configuration are assumed to be unchanged in each time increment. Therefore, the interface should be in either stick phase or slip phase during each incremental time. Before solving the response at the next time step $i+1$, the condition at the interface must be determined. In this study shear-balance procedure, which was proposed by Wang et al. [4] for analyzing sliding structures by state-space approach, is used to determine the phase of the interface.

The bilinear model is also used to idealize reinforced concrete columns and steel columns. No matter how the elements may change properties and configuration, even fracture in each time step, they are assumed to be unchanged in each time interval $t_i \le t \le t_{i+1}$. Thus, the internal forces are calculated based on the element properties and configuration at the initial time t_i. The deformation coordinates of elements are redefined at the beginning of each time step. In other words, once an element undergoes nonlinear or discontinuous behavior, all changes are reflected only at the beginning of next time step.

3 WINKLER-BASED MODEL

Generally the soil-structure interaction behavior is idealized by using a linear elastic model with an infinite capacity. However, the linear behavior between soil and foundations may be not realistic under extreme earthquakes. This study thus adopts the concept of a Beam-on-Nonlinear-Winkler-Foundation, a nonlinear model and called Winkler-Based Model, to model the soil-structure interaction behavior under large seismic loading.

The foundation is herein idealized as a linear elastic beam supported by a number of discrete. Each beam element has six degrees of freedom at both end nodes to consider nodal external loads and deformations. All Winkler springs are independent and considered as one-dimensional zero length elements [5]. Nonlinear Winkler springs can be used to simulate the soil vertical bearing forces, horizontal passive forces to the side of foundations, and vertical shear friction on the sides of foundations in the soil-structure interaction, as described below.

3.1 q-z Model

The q-z model consisting of elastic and plastic parts simulates an unsymmetrical hysteretic response as in Fig. 1. The elastic part captures the far-structure response while the plastic part captures the near-structure permanent displacement as in Fig. 2. A gap model consisting of a drag and a closure spring in parallel is inserted in series with the plastic spring. Soil radiation damping can be evaluated by using a dashpot on the far-structure elastic part.

The equations used to represent the q-z model are similar to those used for the p-y model described in Boulanger et al. [2]. In the elastic portion, the spring stress q is calculated as:

$$q = k_{in}\ z \tag{2}$$

where k_{in} = initial elastic stiffness; z = instantaneous displacement. The range of the elastic region is by the following relation:

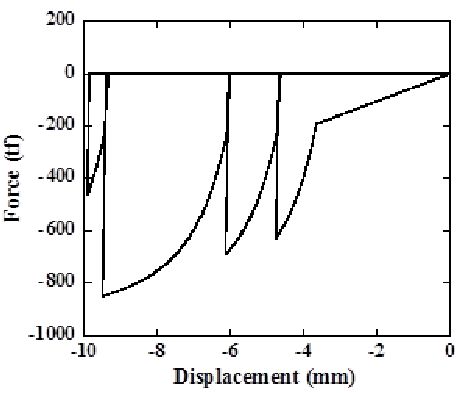

Figure 1: Hysteretic loop of q-z model

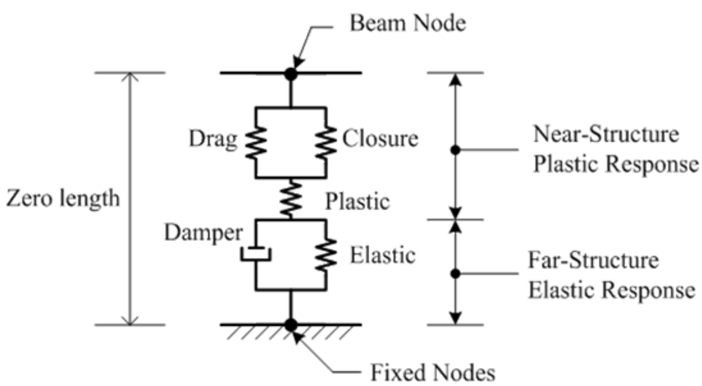

Figure 2: Conceptual construction of the springs (by Raychowdhury [5])

$$q_o = C_r q_{ult} \tag{3}$$

where q_0 = yield stress; C_r = parameter controlling the range of the elastic portion; q_{ult} = ultimate stress.

In the plastic portion, the spring q stress is calculated by

$$q = q_{ult} - (q_{ult} - q_o^p)\left[\frac{cz_{50}}{cz_{50} + \left|z^p - z_o^p\right|}\right]^n \tag{4}$$

where q_0^p = stress at the yield point; c and n = the constitutive parameters controlling the shape of the post-yield portion of the backbone curve ; z_{50} = the displacement at which 50% of the ultimate stress is mobilized ; z^p = the displacement at any point in the post-yield region ; z_0^p = the displacement at the yield point.

3.2 p-x Model

The p-x model is developed to simulate passive horizontal soil resistance against footing. The p-x model is characterized by a pinched hysteretic behavior as shown in Fig. 3. The p-x springs are generally placed at the side of foundation to consider the passive horizontal soil force.

3.3 t-x Model

The t-x model is aimed to simulate the frictional resistance between the soil and footing. The t-x model is characterized by a large initial stiffness and a hysteresis loop as shown in Fig. 4. The expressions governing the p-x model and t-x model are similar to Eqs. (2)-(4) but with different constants of C_r, n and c, which control the general shape of the hysteretic curve.

In this study, the above soil spring models are developed in the VFIFE to simulate the nonlinear soil-structure interaction behavior in the failure process of bridges under extreme earthquakes. The analysis model of spread foundation is depicted in Fig.5.

4 NUMERICAL SIMULATION

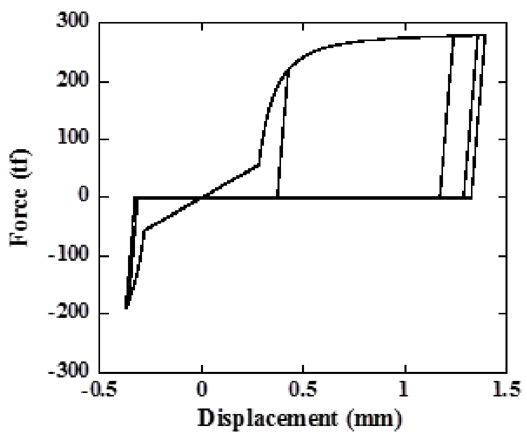

Figure 3: Hysteretic loop of p-x model

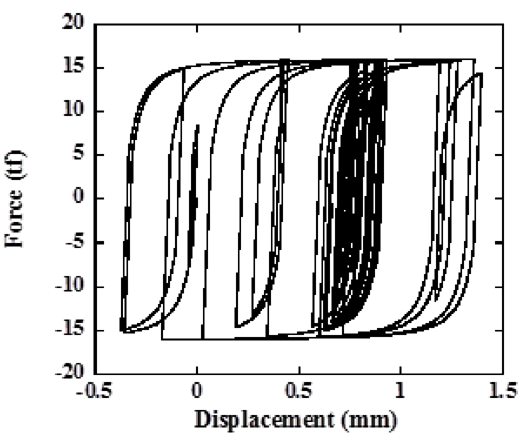

Figure 4: Hysteretic loop of t-x model

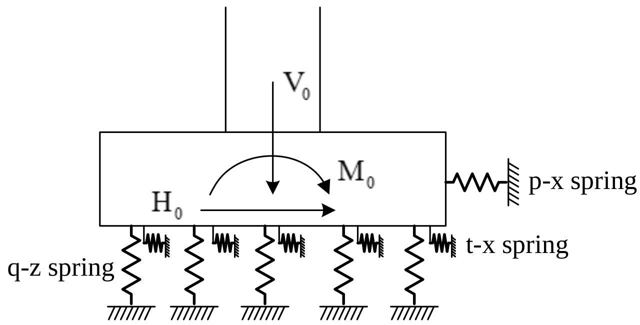

Figure 5: Analysis model of spread foundation

4.1 Target Bridge

A continuous bridge based on Japan highway design codes [6] is analyzed under near-field ground motions to predict the collapse mechanism while considering pounding effect of superstructures. Hinge and roller bearings are installed between the superstructures and the columns or abutments. This bridge consists of a five-span deck with a total length of 5@40 m = 200 m and a width of 12 m, which is supported by four reinforced concrete columns with a height of 12 m in each and two abutments as shown in Fig.6. Spread foundation is designed to consider the rocking isolation effect. The 3-D analysis model is depicted in Fig. 7. The bottoms of columns are idealized as a perfect elastoplastic model with a fracture ductility of 21.5 as shown in Fig. 8. After hinge bearings are damaged by the shearing forces, they are assumed to behave as roller bearings whose dynamic friction coefficient on the fracture interface is 0.15.

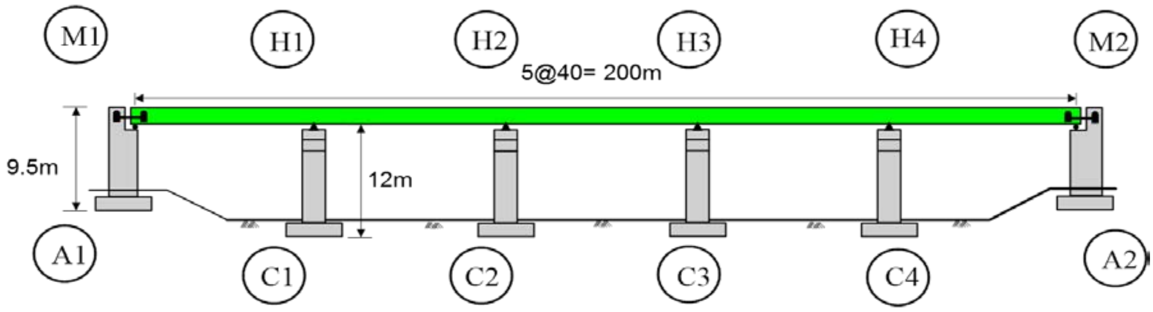

Figure 6: Target bridge

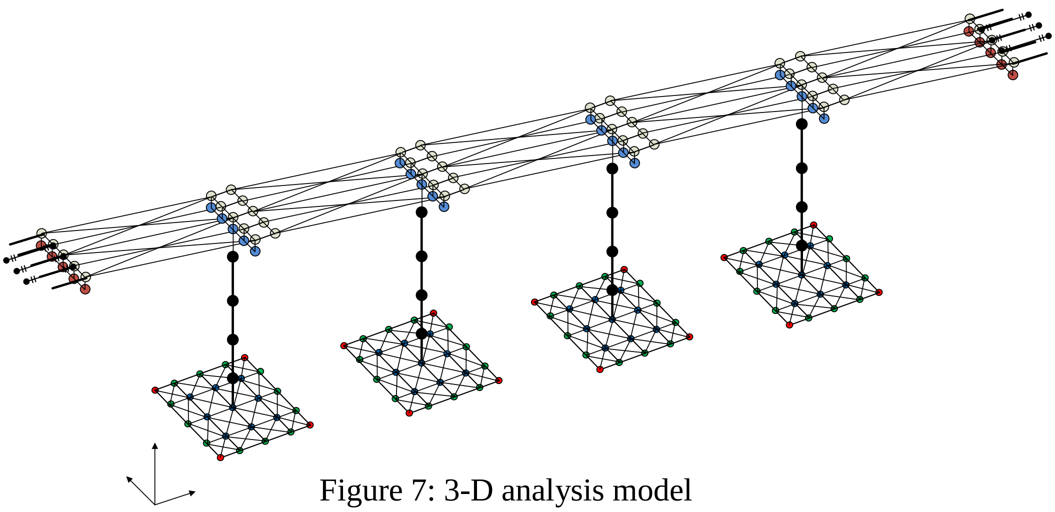

Figure 7: 3-D analysis model

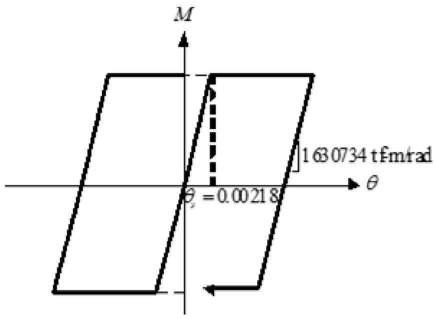

Figure 8: The hysteresis of the column

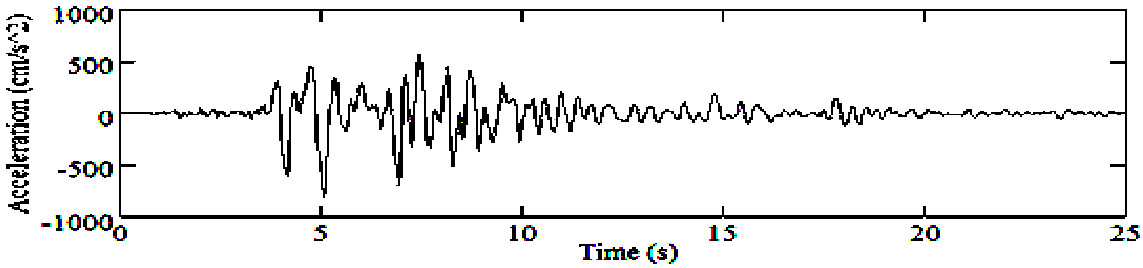

Figure 9: Ground motion recorded at Japan Meteorological agency in the 1995 Kobe earthquake

The pounding effect between the deck and abutment is considered by using an element with a gap of 28 cm. The unseating prevention length at each abutment is 96 cm. In simulations, the bridge is subjected to the near-field ground motions recorded at Japan Meteorological agency, in the 1995 Kobe, Japan earthquake, as in Fig. 9. The ground acceleration is amplified from 100% to 300% at an increment of 10% to result in member failure and the collapse of whole bridge.

4.2 Simulation Results

Figure 10 compares the seismic responses obtained by numerical simulations with linear soil springs and nonlinear Winkler soil springs under JMA Kobe earthquake. Figures 10(a), 10(b) and 10(c) represent the deck displacement, column rotation and vertical displacement at the edge of the foundation, respectively. Figure 10(d) indicates the soil compression force under the edge of the foundation. Observed from the results, the analysis model with linear soil springs overestimates the seismic responses of the target bridge under large earthquakes.

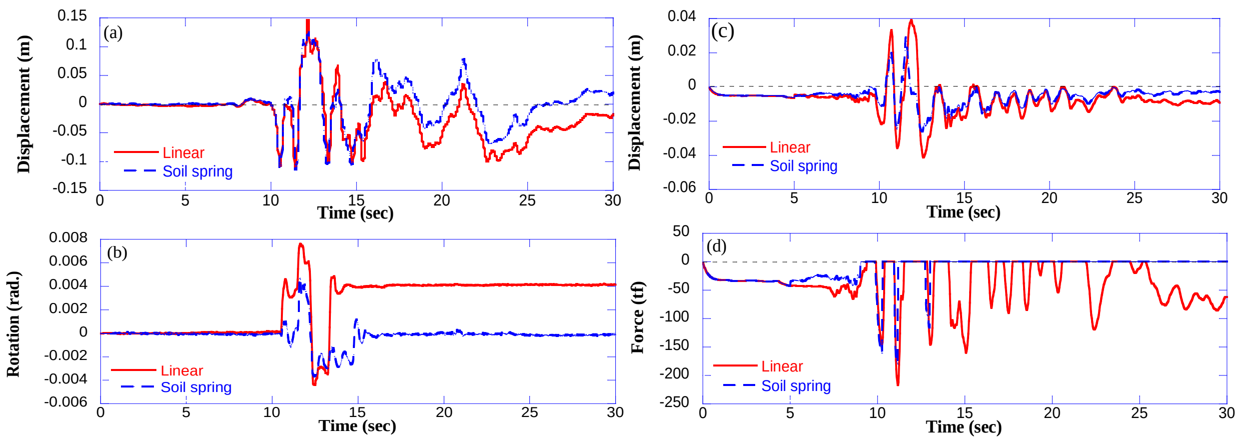

Figure 10: (a) deck displacement (b) column-rotation (c) vertical displacement at edge of foundation (d) soil compression at edge of foundation.

Figure 11 depicts the failure procedure of the target bridge under 250% of the JMA Kobe records. The first characters M, H, C, D and R of the notations in figures denote the roller bearing, hinge bearing, column, deck and tendon, respectively. In the beginning, all the hinge bearings in x-direction first fracture due to excessive shearing forces. Then all the columns gradually reach the ultimate ductility and finally collapse, which results in the falling of all the decks; meanwhile, all the tendons fracture as a result of the excessive tensile forces much larger than the ultimate strength of the tendons.

5 CONCLUSIONS

- The nonlinear soil springs, Winkler-based springs, are developed in the 3-D VFIFE to simulate the nonlinear soil-structure interaction behavior. Therefore, the VFIFE is capable of simulating the failure process of bridges with rocking isolation under extreme earthquakes. The dynamic behavior of all structural members in the extreme situation can be realized through numerical analysis.
- The simulation results confirm the effectiveness of rocking isolation on mitigation of seismic induced forces of bridges with spread foundations. Since the soil may undergo hysteretic behavior under strong earthquakes, such hysteretic behavior can dissipate much more energy to decrease the structural dynamic responses. Observed from the simulation results under large earthquakes, the analysis results with linear soil springs overestimate the seismic responses as compared with those with nonlinear soil springs.

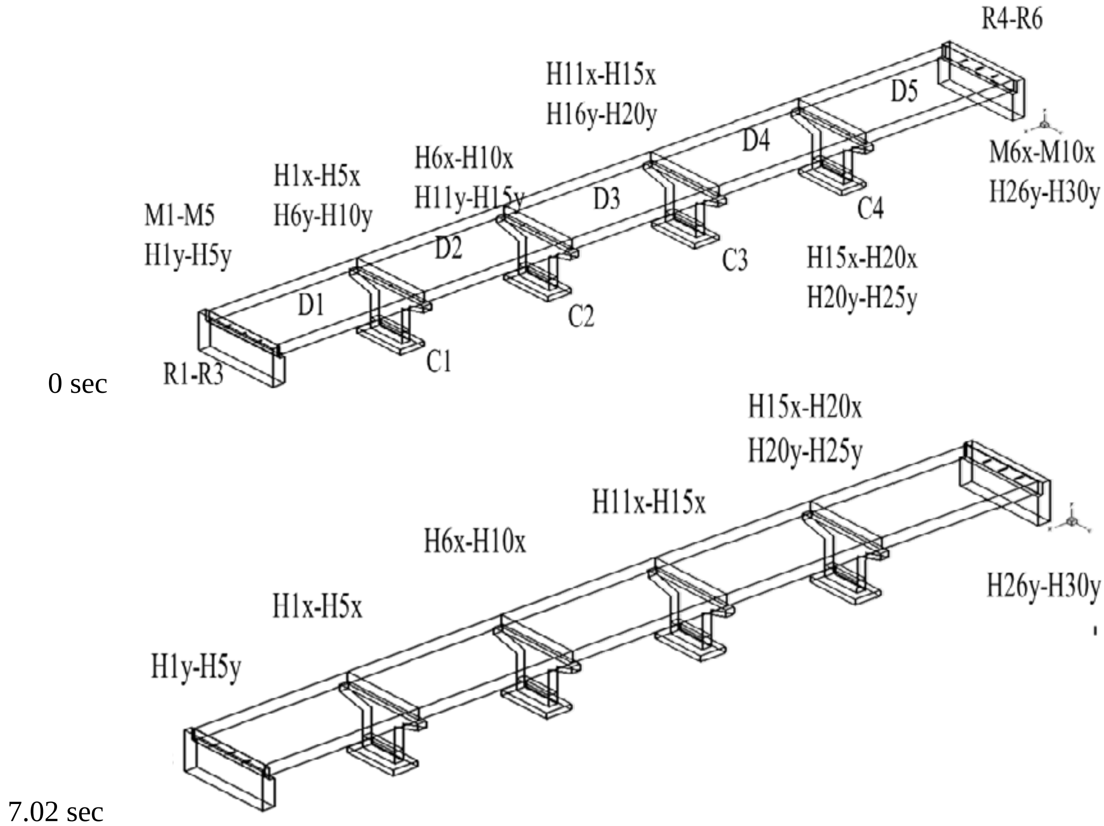

Figure 11: Failure process of the bridge with spread foundations under 250% of JMA Kobe records.

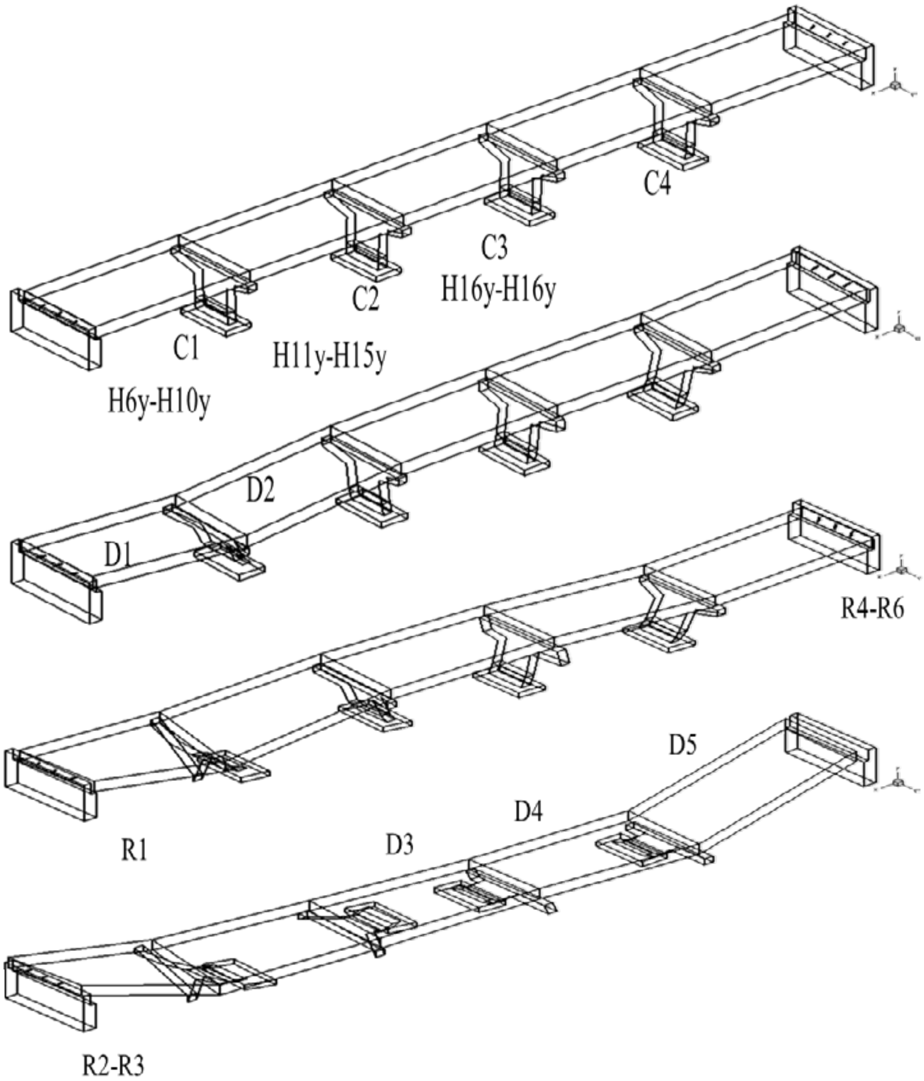

Figure 11(cont.) : Failure process of the bridge with spread foundations under 250% of JMA Kobe records.

- The simulation results show that the induced seismic force decreases as the rocking response increases. The collapse of the bridge is attributed to the failure of the columns, namely the unstable response of the whole bridge due to overturning of the footings does not occur under the extreme earthquakes.

REFERENCES

[1] E.C. Ting, C. Shih, Y.K. Wang, Fundamentals of a vector form intrinsic finite element: Part I. basic procedure and a plane frame element. *Journal of Mechanics*, **20**, 113-122, 2004.

[2] R.W. Boulanger, C.J. Curras, B.L. Kutter, D.W. Wilson, A. Abghari, Seismic soil-pile-structure interaction experiments and analysis. *Journal of Geotechnical and Geoenvironmental Engineering*, **125**, 750-759, 1999.

[3] T.Y. Lee, P.H. Chen, R.Z. Wang, Dynamic analysis of bridges in the ultimate state under earthquakes. *Sixth International Conference on Urban Earthquake Engineering,* Tokyo, Japan, March 3-4, 2009.

[4] Y.P. Wang, W.H. Liao, C.L. Lee, A state-space approach for dynamic analysis of sliding structures. *Engineering Structures*, **23**, 790-801, 2001.

[5] P. Raychowdhury, T.C. Hutchinson, Performance evaluation of a nonlinear Winkler-based shallow foundation model using centrifuge test results. *Earthquake Engng Struct Dyn*, **38**, 679-698, 2009.

[6] Japan Road Association 2002. Specifications for Highway Bridges, Part IV Substructures, Tokyo, Japan.