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DYNAMIC BEHAVIOR OF SEISMIC-EXCITED BRIDGES IN ULTIMATE STATES

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ABSTRACT

This study is aimed to predict the ultimate situation of bridges with or without unseating prevention devices through numerical analysis. In the past extreme earthquakes, a number of bridges suffered damage with unseating of the superstructures. Therefore, it is full of curiosity that how large earthquake will cause a bridge to collapse and how the ultimate state will be. It is extremely difficult to conduct a shaking table test by using a proto-model, especially for multi-span or long-span bridges. The Vector Form Intrinsic Finite Element (VFIFE) is superior in managing the engineering problems with material nonlinearity, discontinuity, large deformation, large displacement and arbitrary rigid body motions of deformable bodies. In this study the VFIFE is thus selected to be the analysis method. Once reaching the ultimate state, a bridge undergoes progressive failure, fragmentation and collapse. Structural elements enter nonlinear material range and/or exhibit large geometry nonlinearity even rigid body motion. The analysis methods in VFIFE for sliding of structures and fracture of elements are herein introduced to predict the collapse mechanism of bridges. Three types of bridges, a six-span simply-supported bridge, a continuous-span bridge with hinge and roller bearings and a continuous-span bridge with high-damping-rubber isolators, are analyzed. The input ground motion was recorded at JR Takatori station in 1995 Japan Kobe earthquake. The ground acceleration varies from 100% to 300% at an increment of 10%. Through numerical simulation of three bridges with or without unseating prevention devices, the ultimate states are demonstrated and compared. The results show that the unseating prevention devices do not increase the safety of the studied bridges as expected. It is interesting to observe that the simply-supported bridge suffers unseating of the superstructure under much lower ground motion than the continuous-span bridge with rigid bearings. The continuous-span isolated bridge suffers unseating under lower ground motion than the simply-supported bridge. Also, the results confirm that the VFIFE is a powerful computation method to simulate the failure mechanism of devices and structural elements so as to successfully predict the ultimate states of bridges.

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Introduction

In the past extreme earthquakes, such as 1995 Japan Kobe earthquake and 1999 Taiwan Chi-Chi earthquake, a number of bridges suffered damage with unseating of superstructures. Whenever unseating failure occurs, the importance of unseating prevention strategy is emphasized repeatedly (Kawashima and shoji 2000). Lately, modern bridge seismic design has been developed toward the seismic performance design on entire bridges as well as components thereof. Unseating prevention is the main requirement for the performance of safety. It is full of curiosity that if the unseating prevention devices are effective, how large earthquake will cause a bridge to collapse and how the ultimate state will be. It is quite difficult to conduct a shaking table test by using a proto-model, especially for multi-span or long-span bridges. However, understanding the ultimate performance of entire bridges and their components, such as bearings, unseating prevention devices, columns, shall be favorable to verify the achievement of performance goal. This study is aimed to predict the ultimate situation of bridges with or without unseating prevention devices through numerical analysis. A new nonlinear structural dynamic analysis method is used to simulate the dynamic behavior of bridges under large earthquakes.

The Vector Form Intrinsic Finite Element (VFIFE), a new computational method developed by (Ting et al. 2004), is superior in managing the engineering problems with material nonlinearity, discontinuity, large deformation, large displacement and arbitrary rigid body motions of deformable bodies. The VFIFE is thus selected to be the analysis method in this study. Since the VFIFE is still in its infant stage, there are still numbers of elements and functions needed to be constructed. Once reaching the ultimate state, a bridge undergoes progressive failure, fragmentation and collapse. Structural elements enter nonlinear material range and/or exhibit large geometry nonlinearity even rigid body motion. The analysis methods for sliding of structures and fracture of elements are herein introduced to predict the collapse mechanism of bridges. Three types of bridges, a six-span simply-supported bridge, a continuous-span bridge with hinge and roller bearings and a continuous-span bridge with high-damping-rubber isolators, are analyzed. Through numerical simulation of three bridges with or without unseating prevention devices, the ultimate states are demonstrated and compared. Some interesting results are observed.

Vector Form Intrinsic Finite Element

The Vector Form Intrinsic Finite Element has been developed based on theory of physics mainly to simulate failure response of a structural system due to applied loads. The first step in VFIFE analysis is to construct a discrete model for a continuous structure by using a lumped-mass idealization. It is noted that all lumped masses are connected by deformable elements without mass. Applying Newton's Second Law of Motion, the equations of motion are established at each mass or so-called node for all degrees of freedoms. Assume that a structural system consists of a finite number of particles. A particle designated as α has a mass $\mathbf{M}^\alpha$ and a displacement $\mathbf{d}^\alpha(t)$ at time t. The equation of motion for particle α is

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

where $\mathbf{P}^\alpha$ is the applied force or equivalent force acting on the particle; $\mathbf{f}^\alpha$ is the total resistance force exerted by all the elements connecting with the particle, or the internal resultant force. Each element without mass is in static equilibrium.

Since the failure progress of structures involves changes in material properties and structural configuration, discrete time domain analysis is essential to solve the equation of motion, Eq. 1. In calculating the internal forces, a set of deformation coordinates is defined for each element and for each time increment. Compared to the traditional finite elements, the feature of VFIFE is the calculation of rigid body motion and deformation of elements through the deformation coordinates in each time increment. By doing so, VFIFE can deal with large displacement, deformation and rigid body motion simultaneously.

For multi-degree-of-freedom systems, it is not necessary to assemble the global property matrices of structures in VFIFE analysis, i.e. matrix algebraic operation is waived. In stead, only scalar calculation is needed for each particle. The central difference method, an explicit time integration method, is adopted to solve the equation of motion, Eq. 1.

Simulation of Ultimate States

Bridges may undergo nonlinear behavior even structural failure when subjected to extreme earthquakes. In the past large earthquakes, a number of bridges suffered deck unseating, which is high nonlinearity along with rigid body motion. To simulate the collapse mechanism of bridges, the failure mechanism of major bridge components should be taken into account.

The studied failure components are hinge bearings, high-damping-rubber isolators, unseating prevention devices and plastic hinges of decks and columns. Firstly, both bearings, hinge bearings and isolators, are idealized as a linear model and a bilinear model, respectively. Assume that the bearings fracture as the deformation reaches rupture deformation. Once the bearing breaks, there is no restoring shear force between the superstructure and column. In stead, the friction force acts at the interfaces. 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 fall down from the column due to the gravity force. The failure of isolators represents a typical failure mechanism completing material linear and nonlinear hysteretic behavior, fracture, and sliding of structures. The writers (Lee et al. 2008) have developed the nonlinear elements in VFIFE in the previous study. This paper herein introduces the analytical methods for sliding structures and fracture of elements in VFIFE.

Sliding of Structures

After the bearing 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 still within 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, i.e. 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, i.e. 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 VFIFE, the material properties and structural configuration are assumed to be unchangeable 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 next

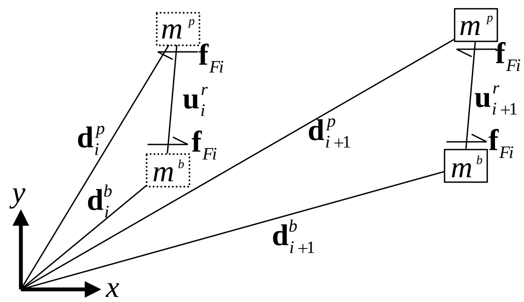


Figure 1. The motion of the superstructure and the column

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. (2001) for analyzing sliding structures by state-space approach, is used to judge which phase the interface is in.

The first step is to calculate the friction force in the interface on assumption of stick phase. It is noted that the relative displacement is null in stick phase. The interface is in stick phase if the calculated friction force is less than the dynamic friction force while it is in slip phase if the calculated friction force is equal or larger than the dynamic friction force.

Figure 1 illustrates the motion of the superstructure with mass $\mathbf{M}^p$ and the column with $\mathbf{M}^b$ at time step i and $i+1$. The equations of motion for the two masses in the central difference equations are

$$\hat{\mathbf{K}}^p \mathbf{d}_{i+1}^p = \hat{\mathbf{P}}_i^p - \mathbf{f}_{Fi} \tag{2}$$

$$\hat{\mathbf{K}}^b \mathbf{d}_{i+1}^b = \hat{\mathbf{P}}_i^b + \mathbf{f}_{Fi} \tag{3}$$

where $\hat{\mathbf{K}}^p, \hat{\mathbf{P}}_i^p$ are the effective stiffness and force, respectively; $\mathbf{f}_{Fi}$ is designated the friction force in the interface. If the interface is in stick phase, the relative displacement $\mathbf{u}_i = \mathbf{d}_i^p - \mathbf{d}_i^b$ between the superstructure and the column at time step i is the same as the relative displacement $\mathbf{u}_{i+1} = \mathbf{d}_{i+1}^p - \mathbf{d}_{i+1}^b$ at time step $i+1$.

$$\mathbf{d}_{i+1}^p - \mathbf{d}_{i+1}^b = \mathbf{d}_i^p - \mathbf{d}_i^b \tag{4}$$

Rearranging and substituting Eqs. 2 and 3 into Eq. 4, the calculated friction force $\tilde{\mathbf{f}}_{Fi}$ is obtained as

$$\tilde{\mathbf{f}}_{Fi} = \frac{\hat{\mathbf{K}}^b \hat{\mathbf{P}}_i^p - \hat{\mathbf{K}}^p \hat{\mathbf{P}}_i^b - \hat{\mathbf{K}}^p \hat{\mathbf{K}}^b \left(\mathbf{u}_i^p - \mathbf{u}_i^b \right)}{\hat{\mathbf{K}}^p + \hat{\mathbf{K}}^b} \tag{5}$$

If the calculated friction force $\tilde{\mathbf{f}}_{Fi}$ is less than the dynamic friction force, the assumption of stick phase is true and the calculated friction force can be used in the next time increment, i.e. $\mathbf{f}_{Fi} = \tilde{\mathbf{f}}_{Fi}$. Otherwise, the interface is in slip phase. The friction force $\mathbf{f}_{Fi}$ must be substituted by dynamic friction force μN, i.e. $\mathbf{f}_{Fi} = \mu N$. The above can be summarized as

$$\begin{cases} \mathbf{f}_{Fi} = \mu N & \text{if} \quad \tilde{\mathbf{f}}_{Fi} \geq \mu N, \text{ slip phase} \\ \mathbf{f}_{Fi} = \tilde{\mathbf{f}}_{Fi} & \text{if} \quad \tilde{\mathbf{f}}_{Fi} < \mu N, \text{ stick phase} \end{cases} \tag{6}$$

Fracture of Elements

The first design principle of isolated bridges is the appropriate utilization of isolators and dampers to shift the main periods of vibration and increase the energy-dissipation capacity of the structures (Priestley et al. 1996). High damping rubber bearings and lead-rubber bearings, which have both the functions, are commonly utilized. Both bearings can be idealized by a bilinear model. In addition, it has been shown in the past study that the columns of isolated bridges may exhibit nonlinear behavior under extreme earthquakes (Lee and Kawashima 2007). The bilinear model can also be used to idealize reinforced concrete columns and steel columns. It must be carefully managed in constructing the bilinear element of VFIFE for the loading, unloading and reloading paths. The unseating prevention devices are designed to provide the function against unseating. The unseating prevention devices are generally with non-working length before they are triggered to function. Therefore they are idealized as elements with a hook or/and a gap.

All properties and configuration of elements are assumed to be unchangeable in each time interval $t_i \leq t \leq t_{i+1}$ in VFIFE. 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. Therefore, once an element undergoes nonlinear or discontinuous behavior, all changes are reflected at the beginning of next time step.

In this study, the aforementioned elements are considered failure components. Assume that an element fractures as its deformation reaches rupture deformation. At the beginning of each time step it is checked where the element failures or not. Once the element meets the fracture condition, the element and its restoring forces are released from the system.

Numerical Simulation

Three types of bridges, a six-span simply-supported bridge, a continuous-span bridge with hinge and roller bearings and a continuous-span bridge with high-damping-rubber isolators, as shown in Fig 2, are analyzed.

All bridges consist of a six-span deck with a total length of 6@40 m = 240 m and a width of 12 m, which is supported by five reinforced concrete columns with a height of 12 m in each and two abutments, as shown in Fig. 3. The continuous-span isolated bridge is designed based on the Japan highway bridge design codes. In order to make comparison, the substructure and superstructure of the other bridges with rigid bearings are the same as the isolated bridge except for the bearing systems. The columns are idealized as a perfect elastoplastic model with a fracture ductility of 21.5 shown in Fig. 4(a). The isolators are idealized as a bilinear elastoplastic model with a fracture shear strain of 500% while the hinge bearings of the simply-supported bridge and the continuous bridge are idealized as a linear model with a fracture shear of 255 tf and 383 tf, respectively, as shown in Figs. 4(b) and 4(c). After bearings rupture, the dynamic friction coefficient at the interface is assumed to be 0.15. The friction coefficient of the roller bearings is assumed to be 0.1.

Steel tendons are installed at each expansion joint as the unseating prevention devices. The tendons are simulated by a tension element with a yielding force of 839 kN, an ultimate

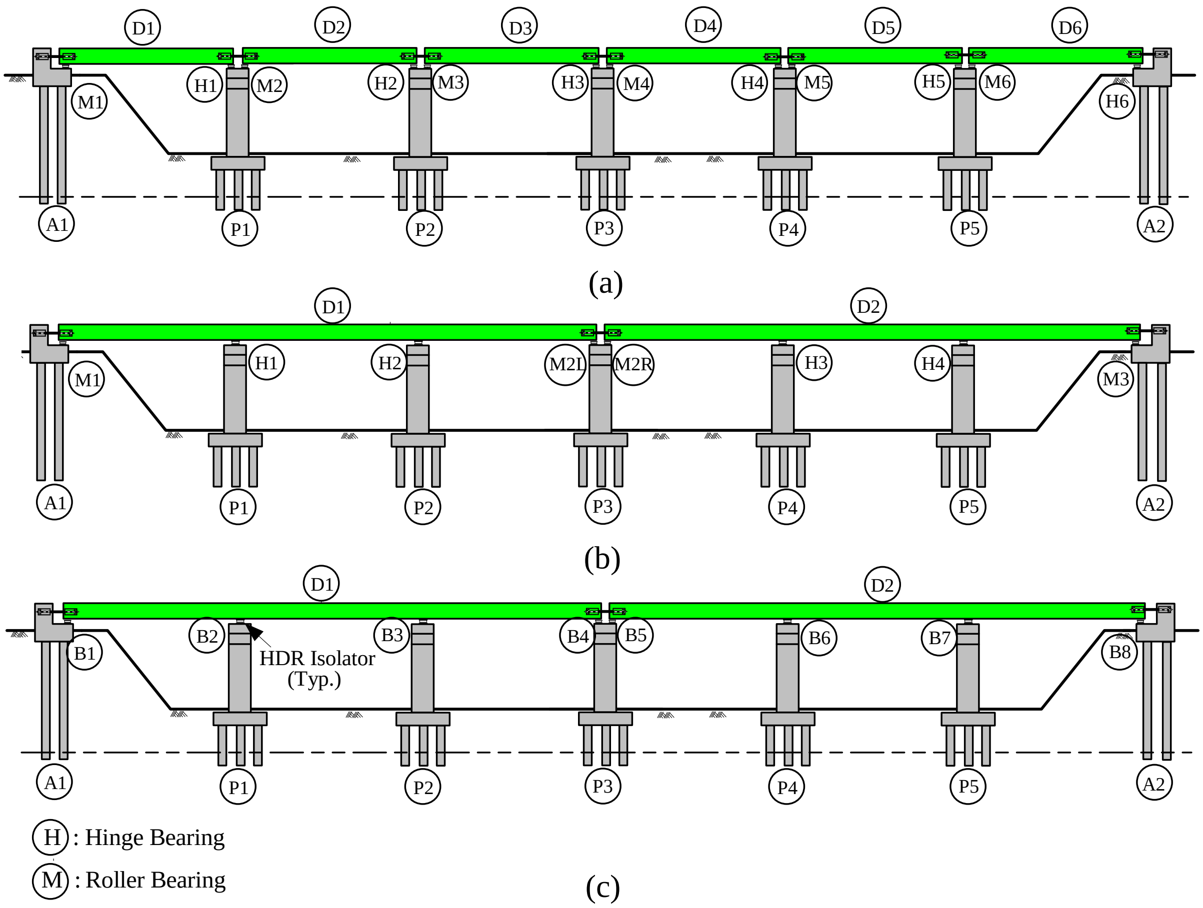


Figure 2. (a) simple-supported bridges with rigid bearings (b) continues-span bridges with rigid bearings (c) isolated bridges

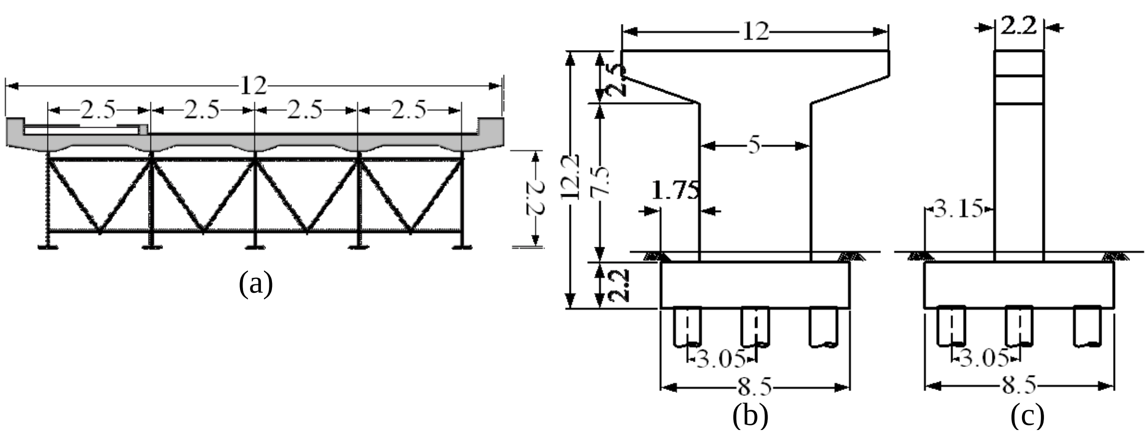


Figure 3. (a) lateral view of superstructure (b) lateral view of column and (c) side view of column

force of 932 kN and a hook of 40 cm shown in Fig. 4(d). The pounding effect of two adjacent decks is also considered by using an element with a gap of 28 cm. The unseating prevention length at each column and abutment is 96 cm. Two installation types of the unseating prevention devices located at the columns are studied. One type is the connection between two adjacent decks. The other is the connection between the deck and the cap beam. In simulation, the bridges are subjected to near-field ground motions recorded at JR Takatori station, in the 1995 Kobe, Japan earthquake. The ground acceleration is amplified from 100% to 300% at an increment of 10%.

Through numerical simulation of three bridges with or without unseating prevention devices, the ultimate states are demonstrated and compared. It is interesting to observe that the simply-supported bridge suffers unseating of the superstructure as the ground motion is amplified equal to and larger than 160% while the continuous-span bridge with rigid bearings do not unseat even under 260% ground motion. However the continuous-span isolated bridge suffers unseating when the ground motion is amplified equal and larger than 140%. Figures 5 through 8 depict the failure states of the isolated bridge under 140% ground motion and simply-supported bridge under 170% ground motion with or without unseating prevention devices at 5.6 sec and final condition, where the first characters B, C, D, R of the notions denote the bearing, column, deck and tendon, respectively. For the isolated bridge, the collapse is attributed to column failure because the fracture shear of isolators is larger than fracture lateral force of the columns. The collapse of the simply-supported bridge is due to insufficient unseating prevention length. The continuous-span bridge with rigid bearings performs better than the other bridges because a continuous deck has less possible unseating points and the hinge bearings fail earlier than the columns. The results show that the unseating prevention devices do not increase the safety of the studied bridges as expected.

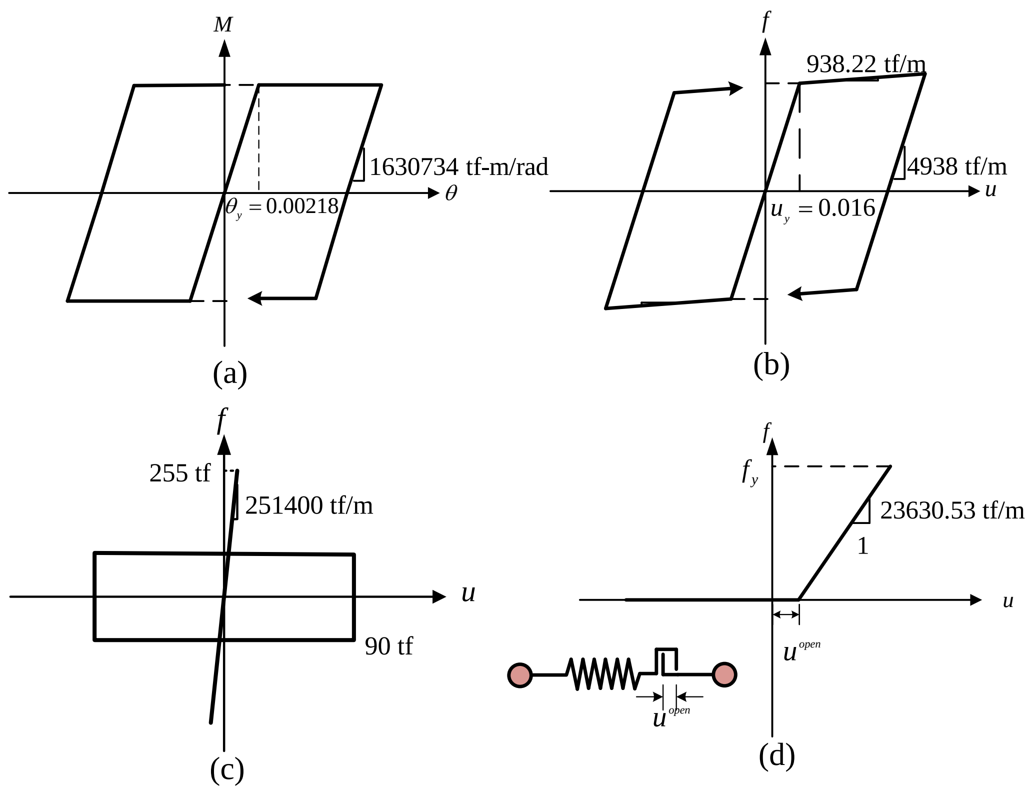
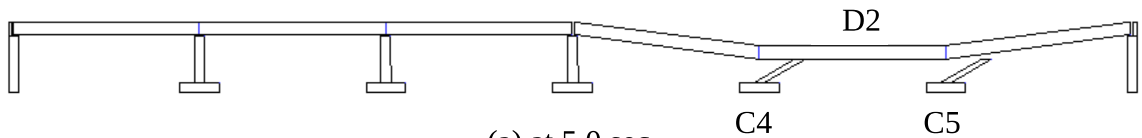
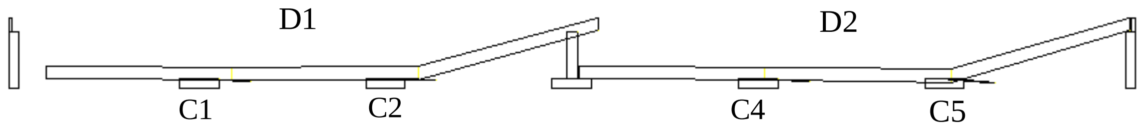


Figure 4. Material property (a) column (b) isolator (c) hinge bearing (d) steel tendon

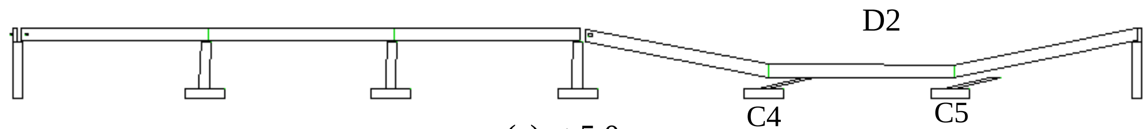


(a) at 5.0 sec

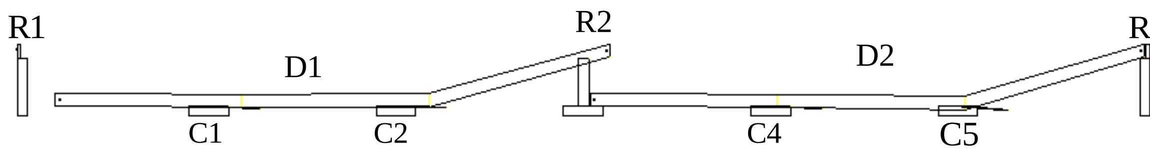


(b) at 20.0 sec

Figure5. Failure states of the isolated bridge without unseating prevention devices under 140% ground motion

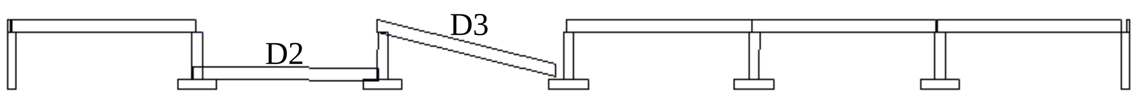


(a) at 5.0 sec

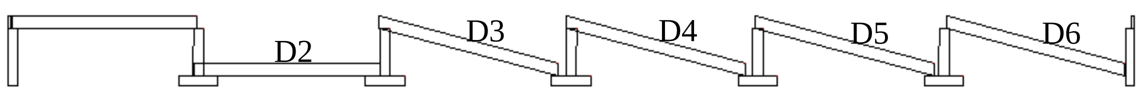


(b) at 20.0 sec

Figure6. Failure states of the isolated bridge with unseating prevention devices under 140% ground motion

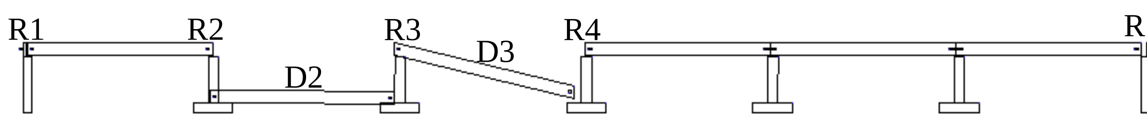


(a) at 5.0 sec

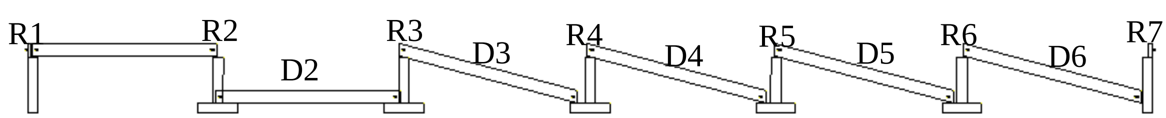


(b) at 20.0 sec

Figure 7. Failure states of the simple-supported bridge without unseating prevention devices under 170% ground motion



(a) at 5.0 sec



(b) at 20.0 sec

Figure 8. Failure states of the simple-supported bridge with unseating prevention devices under 170% ground motion

Conclusions

Since the VFIFE has the advantages in managing the engineering problems with material nonlinearity, discontinuity, large deformation, large displacement, arbitrary rigid body motions of deformable bodies and even fracture and collapse, it is adopted in this study to predict the ultimate states of bridges with and without unseating prevention devices under large earthquakes. Three types of bridges are analyzed under JR Takatori ground motion amplified from 100% to 300%. The numerical simulation successfully predicts the failure process of the bridges under extreme earthquakes. The results show that the unseating prevention devices do not increase the safety of the studied bridges as expected. It is interesting to observe that the simply-supported bridge suffers unseating of the superstructure under much lower ground motion than the continuous-span bridge with rigid bearings. The continuous-span isolated bridge suffers unseating under lower ground motion than the simply-supported bridge. Also, the results confirm that the VFIFE is a powerful computation method to simulate the failure mechanism of devices and structural elements

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