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Seismic-induced collapse simulation of bridges using simple implicit dynamic analysis

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A R T I C L E I N F O

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A B S T R A C T

A simple implicit dynamic analysis method is extended to simulate the seismic-induced collapse of bridges. Nonlinear and discontinuous behaviors, such as material yielding and cracking, member damage, separation, falling and collision with other members, are considered in the analysis. An impact model with a contact detection scheme is developed to consider the impact response between beam elements representing bridge superstructures. Multiple-support excitation is considered. The Matsurube Bridge which collapsed in the 2008 Japan Iwate-Miyagi inland earthquake due to not only strong ground excitations but also the sliding of the rock mass beneath the bridge is analyzed for verifying the effectiveness of the analysis method. By reproducing the in-situ collapse situation, the failure mechanisms of the bridge are estimated. The results demonstrate that the simple implicit dynamic analysis is robust in simulating the collapse of bridges that exhibit highly nonlinear and discontinuous behaviors under extreme earthquakes.

1. Introduction

A large number of bridges have suffered severe damage or even collapsed in past earthquakes. Accordingly, it is paramount to estimate the failure causes and mechanisms so that bridge seismic designs can be modified to avoid similar damage in future earthquakes. With significant advances in computational facilities, the number of numerical studies on structural collapse or damage has gradually increased over the last two decades [1–9]. Structural collapse is a highly nonlinear and discontinuous dynamic process that involves material yielding and cracking, member damage, separation, falling, and collision with other members, making numerical simulations complex. Since employing conventional implicit dynamic finite element (FE) analysis to simulate the failure mechanisms and collapse of structures is challenging, research on structural collapse frequently adopts explicit dynamic FE analysis, using software such as the LS-DYNA, ABAQUS-Explicit, and OpenSees [10]. In addition to the FE method, the distinct element method and applied element method have demonstrated advantages in simulating the discontinuous behavior among members during collapse [6–8]. In particular, the distinct element method in conjunction with the explicit central difference integration scheme (CDIS) has been frequently employed to analyze the discontinuous behaviors of granular materials, such as soil and rock, due to its computational efficiency [11–13]. However, explicit integration methods are conditionally stable. When analyzing a large complicated system with a high-frequency response, very small time steps are required to ensure numerical stability and obtain an accurate solution because iterations are not conducted to rigorously satisfy the equilibrium equations within explicit integrations. Additionally, damping is inherent in a dynamic system. Once stiffness-proportional damping is taken into account to simulate more realistic structural behavior, the equation decoupling and computational efficiency of explicit CDIS are lost. Furthermore, since CDIS is a multi-step integration method, strictly speaking, it cannot be employed to simulate discontinuous responses. Generally, a typical bridge consists of superstructures, substructures, and appurtenances, and compared to a building, usually comprises more types of components with various mechanical properties [14]. In order to realistically simulate the collapse process of bridges, the requirements of the detailed numerical model are very high, which makes numerical procedures complicated and time-consuming. Consequently, a simple, robust, and efficient dynamic analysis method is needed to simulate structures, especially large-scale complicated structures, with highly nonlinear and discontinuous responses under extreme earthquakes.

A simple implicit dynamic analysis method with decoupled equilibrium equations was proposed by Lee et al. based on the concept of equivalent nodal secant stiffness and damping coefficients [15,16]. This method combines the advantages of both conventional implicit and explicit integration methods while avoiding their drawbacks. In the

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present study, this simple implicit dynamic analysis method is extended to simulate the seismic-induced progressive collapse of a bridge. In the simulation of nonlinear structures subjected to multiple-support excitation (MSE), the equations of motion (EOMs) are formulated in the absolute coordinates [17–20]. In addition, the pounding between structural components is considered in the analysis. An impact model with a contact detection scheme is developed to consider the response to impact between beam elements representing bridge superstructures. Finally, the simple implicit dynamic analysis method is applied to the Matsurube Bridge, which collapsed in the 2008 Japan Iwate-Miyagi inland earthquake due to not only strong ground excitations but also the sliding of the rock underneath the bridge [21–23] to verify its effectiveness and robustness.

2. Dynamic analysis

2.1. Simple implicit dynamic analysis with decoupled equations

A simple implicit dynamic analysis with decoupled equations was used to simulate the collapse of structures under extreme earthquakes [15,16]. In dynamic analysis, basic discrete equilibrium equations based on the principle of virtual displacements for a structure can be established at time t as:

$$\mathbf{F}_I(t) + \mathbf{F}_D(t) + \mathbf{F}_S(t) = \mathbf{R}(t) \tag{1}$$

where $\mathbf{F}_I(t)$, $\mathbf{F}_D(t)$,and $\mathbf{F}_S(t)$ are the vectors of the equivalent fictitious inertial nodal forces, the equivalent damping nodal forces, and the internal element nodal forces equivalent to the element stresses, respectively, and $\mathbf{R}(t)$ is a vector of the equivalent externally applied nodal loads.

Assuming that the mass is time-invariant and that Rayleigh damping with constant coefficients is used for a geometrically and/or materially nonlinear structure system, Eq. (1) can be rewritten to satisfy the dynamic equilibrium equations at the time instant $t + \Delta t$ using implicit direct integration methods as:

$$\mathbf{M}\,{}^{t+\Delta t}\ddot{\mathbf{U}} + a_0\mathbf{M}\,{}^{t+\Delta t}\dot{\mathbf{U}} = {}^{t+\Delta t}\mathbf{R} - {}^{t+\Delta t}\mathbf{F}_S - {}^{t+\Delta t}\mathbf{F}_{kD} \tag{2}$$

where ${}^{t+\Delta t}\ddot{\mathbf{U}}(t)$ and ${}^{t+\Delta t}\dot{\mathbf{U}}(t)$ are the vectors of nodal accelerations and velocities, respectively; $\mathbf{M}$ is the mass matrix; $a_0\mathbf{M}$ is the mass-proportional damping coefficient; and, ${}^{t+\Delta t}\mathbf{F}_{kD}$ denotes the damping nodal forces that result from stiffness-proportional damping force $a_1\mathbf{K}_I^{t+\Delta t}\dot{\mathbf{U}}$, where $\mathbf{K}_I$ is the initial structure stiffness matrix.

When the mass matrix is constructed diagonally using lumped-mass idealization, the dynamic equilibrium equations of Eq. (2) are decoupled. Furthermore, when implementing an incremental-iterative analysis, the equation at degree of freedom (DOF) i in iteration r can be expressed as:

$$M_i\,{}^{t+\Delta t}\ddot{U}_i^{(r)} + a_0M_i\,{}^{t+\Delta t}\dot{U}_i^{(r)} + {}^{t+\Delta t}(\widetilde{C}_{\text{sec}})_i^{(r-1)}\Delta\dot{U}_i^{(r)} + {}^{t+\Delta t}(\widetilde{K}_{\text{sec}})_i^{(r-1)}\Delta U_i^{(r)} = {}^{t+\Delta t}R_i - {}^{t+\Delta t}(F_{kD})_i^{(r-1)} - {}^{t+\Delta t}(F_S)_i^{(r-1)}\ (i = 1,\ldots,n) \tag{3}$$

where $\Delta\dot{U}_i^{(r)}$ and $\Delta U_i^{(r)}$ are the velocity and displacement increments, respectively, and n is the number of DOFs in the structural system. In addition, ${}^{t+\Delta t}(\widetilde{C}_{\text{sec}})_i^{(r-1)}$ and ${}^{t+\Delta t}(\widetilde{K}_{\text{sec}})_i^{(r-1)}$ are the equivalent nodal secant damping and stiffness coefficients, respectively, at DOF i in iteration $(r-1)$; they are defined as:

$${}^{t+\Delta t}(\widetilde{C}_{\text{sec}})_i^{(r-1)}\Delta\dot{U}_i^{(r-1)} \equiv \Delta\,{}^{t+\Delta t}(F_{kD})_i^{(r-1)} \tag{4}$$

$${}^{t+\Delta t}(\widetilde{K}_{\text{sec}})_i^{(r-1)}\Delta U_i^{(r-1)} \equiv \Delta\,{}^{t+\Delta t}(F_S)_i^{(r-1)} \tag{5}$$

where $\Delta\,{}^{t+\Delta t}(F_{kD})_i^{(r-1)}$ and $\Delta\,{}^{t+\Delta t}(F_S)_i^{(r-1)}$ are the stiffness-proportional damping force increment and internal element nodal force increment, respectively.

The implicit Newmark integration family is frequently employed in practical analyses due to its effectiveness [24]. Since the constant average acceleration scheme is unconditionally stable, it was employed in this study. The assumed variations of acceleration and velocity for DOF i within each time interval Δt are represented as [10]:

$${}^{t+\Delta t}\ddot{U}_i^{(r)} = \frac{1}{\beta(\Delta t)^2}({}^{t+\Delta t}U_i^{(r)} - {}^{t}U_i) - \frac{1}{\beta\Delta t}{}^{t}\dot{U}_i - \left(\frac{1}{2\beta}-1\right){}^{t}\ddot{U}_i \tag{6}$$

$${}^{t+\Delta t}\dot{U}_i^{(r)} = \frac{\gamma}{\beta(\Delta t)}({}^{t+\Delta t}U_i^{(r)} - {}^{t}U_i) + \left(1-\frac{\gamma}{\beta}\right){}^{t}\dot{U}_i + \Delta t\left(1-\frac{\gamma}{2\beta}\right){}^{t}\ddot{U}_i \tag{7}$$

where $\beta = 1/4$ and $\gamma = 1/2$. Substituting Eqs. (6) and (7) into Eq. (3), and then rearranging the terms yields an equivalent static equation:

$${}^{t+\Delta t}(\widehat{K}_{\text{sec}})_i^{(r)}\Delta U_i^{(r)} = \Delta\,{}^{t+\Delta t}\widehat{F}_i^{(r)}(i = 1,\ldots,n) \tag{8}$$

where ${}^{t+\Delta t}(\widehat{K}_{\text{sec}})_i^{(r)}$ is the effective secant stiffness and $\Delta\,{}^{t+\Delta t}\widehat{F}_i^{(r)}$ is the unbalanced force at DOF i, time instant $t + \Delta t$, and iteration r. These terms are expressed as:

$${}^{t+\Delta t}(\widehat{K}_{\text{sec}})_i^{(r)} = \frac{1}{\beta(\Delta t)^2}M_i + \frac{\gamma}{\beta(\Delta t)}a_0M_i + \frac{\gamma}{\beta(\Delta t)}{}^{t+\Delta t}(\widetilde{C}_{\text{sec}})_i^{(r-1)} + {}^{t+\Delta t}(\widetilde{K}_{\text{sec}})_i^{(r-1)} \tag{9}$$

$$\Delta\,{}^{t+\Delta t}\widehat{F}_i^{(r)} = {}^{t+\Delta t}R_i - M_i\,{}^{t+\Delta t}\ddot{U}_i^{(r-1)} - a_0M_i\,{}^{t+\Delta t}\dot{U}_i^{(r-1)} - {}^{t+\Delta t}(F_{kD})_i^{(r-1)} - {}^{t+\Delta t}(F_S)_i^{(r-1)} \tag{10}$$

As can be seen in Eqs. (8)–(10), assembling the structure stiffness and damping matrices is not required in the simple implicit dynamic analysis. Without the factorization of an effective matrix, the computation efficiency is greatly superior to that of conventional implicit methods. In addition, the time step used in the simple method is larger than that used in CDIS, especially for dealing with a hysteresis response. The computation efficiency and stability of the simple implicit dynamic analysis method have been previously demonstrated via the analyses of several examples [15]. Details of this method can be found elsewhere [15,16]. Since internal element nodal forces and damping forces are evaluated at the element level, any kind of FE can be easily incorporated into this dynamic analysis as long as its internal resisting and damping forces can be precisely evaluated.

A typical bridge comprises superstructures, substructures, bearing systems, and appurtenances. During progressive collapse, some critical portions in structures and appurtenances may exhibit highly nonlinear and discontinuous behaviors. Note that the objective of this study was to verify the effectiveness and robustness of the simple implicit dynamic analysis method in simulating the progressive collapse of bridges. Therefore, two-dimensional numerical models were constructed to simplify the analysis and focus on the collapse mechanism. Link and support elements were herein employed to simulate materially nonlinear and discontinuous responses, including material hysteresis, ruptures of members, Coulomb friction damping, pounding between structural components, and soil-structure interactions. Similar to a beam element, a link element connects two nodes, and a support element connects one node to the ground. Referring to the definitions of link/support elements in the well-known structural software SAP2000 [25], it was assumed that a link or a support element is composed of three independent springs, namely axial, lateral, and rotational springs.

2.2. Multiple-support excitation

Multiple-support excitation (MSE) is generally taken into account in analyzing large, spatially distributed structures, such as long-span bridges, buildings, and pipelines [17–20]. As done in the nonlinear dynamic simulation of structures with uniform base excitation, the EOMs with MSE were formulated in the absolute coordinates, including the support/ground DOFs, i.e., the bases of piers and abutments [10]. With lumped-mass idealization, the EOMs for all structural DOFs can be written as follows:

$$\mathbf{M}_{ss}\ddot{\mathbf{U}}_s^a + \mathbf{F}_D + \mathbf{F}_S = 0 \tag{11}$$