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Seismic behavior of irregular reinforced-concrete structures under multiple earthquake excitations

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ABSTRACT

Reconnaissance studies on the recent Tohoku earthquake have reported collapse of structures due to multiple earthquake excitations in the earthquake-affected region. Strength and stiffness degradation is shown to be the primary reason for the observed damage. The present study aims to investigate the degrading behavior of irregularly built reinforced concrete structures subjected to the Tohoku ground motion sequences. Three-dimensional numerical models of three irregular reinforced concrete structures are developed. The structural characteristics of these buildings are then altered to achieve a regular case. The models contain appropriate damage features that can capture both the irregularity and material deterioration effects. The capacities of both cases are evaluated using the N2 and extended N2 procedures. The degrading models are then used for ground motion sequences measured at 23 selected stations. The results indicate that multiple earthquake effects are significant, and irregularity effects increase the dispersed damage under these excitation sequences.

1. Introduction

Reinforced concrete (RC) structures during the recent Tohoku and Christchurch earthquakes experienced excessive loss of stiffness and strength due to repeated shaking. Many RC buildings that were not heavily damaged immediately after the main excitations have collapsed because of aftershocks. Correspondingly, many previous in situ examinations have reported the unfavorable effects of multiple ground excitations on structural systems.

In literature, to determine the response of structures by modeling their structural behavior, single-degree-of-freedom (SDOF) systems were extensively used because of their simplicity. Degrading systems were first introduced by Aschheim and Black [1], who used a modified Takeda hysteretic model. Their model was able to capture both the pinching and strength degradation effects. Base on their conclusions, the displacement response of an initially damaged SDOF system was approximately the same as that of its undamaged counterpart after the peak displacement was reached. Amadio et al. [2] investigated the nonlinear behavior of SDOF structures under multiple excitations using three different hysteretic models: *non-degrading stiffness and strength, degrading stiffness and non-degrading strength, degrading stiffness and strength*. They concluded that elastoplastic systems can be classified as the most vulnerable SDOF systems. Hatzigeorgiou and Beskos [3] conducted an extensive parametric study to obtain an appropriate inelastic displacement ratio while examining the period of vibration, viscous damping ratio, strain-hardening ratio, force reduction factor and soil class. They revealed that the repeated earthquakes have significant effect on both the inelastic displacement ratios and maximum inelastic displacement values of SDOF systems.

In order to consider degrading behavior of moment resisting frame systems in structural analyses, component-level-based degrading models (multi-degree-of-freedom systems) have been developed and widely used in the literature. These models utilize nonlinear moment-rotation relationships at locations of possible plastic hinges (beam and column ends) that consider both stiffness and strength degradation. The idealization of assuming concentrated inelasticity at predefined plastic hinge locations lacks the consideration of localized failure modes and therefore can lead to inaccurate assessment of degrading response under earthquake sequences. Hatzigeorgiou and Liolios [4] investigated the effectiveness of component-level-based models under multiple excitations, assuming bilinear moment–rotation relationships at beam–-column connections. Moreover, beam and column elements are assumed to behave elastically. These developed models can also consider second-order effects; however, they exclude material deterioration effects. The mentioned studies highlighted the fact that residual displacements play a major role on stiffness degradation.

To the best of the authors' knowledge, Abdelnaby and Elnashai [5] are the only researchers who have studied the effects of multiple

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earthquakes using distributed plasticity models, including the material deterioration effects on two-dimensional (2-D) structures. They utilized nonlinear dynamic analyses to incorporate structural damage features in their models. They investigated the degradation behavior of RC structures under the Tohoku and Christchurch earthquake sequences. They concluded that neither system-level-based models nor component-level-based models can accurately estimate the degradation response.

This study is unique because it considers the effects of multiple earthquake excitations for irregular structures. Furthermore, a plastic-damage model of concrete developed by Lee and Fenves [6], and the modified Menegotto-Pinto steel model [7,8] that are previously utilized and implemented by Abdelnaby and Elnashai [5] to ZEUS-NL [9] are used to evaluate the seismic response of irregularly build RC structures. The reader is referred to the cited reference for more information regarding the material model. The aim of this study is to evaluate the seismic performance of irregularly designed RC buildings when subjected to strong ground motion sequences. To properly assess the seismic response of three previously selected RC buildings, 516 nonlinear dynamic analyses are executed using ZEUS-NL [9], considering the Tohoku earthquake sequence. The structures are analyzed for two cases: (1) in their original form and (2) after modifying the geometry with the objective of reducing their level of irregularity without altering the overall stiffness. The results are then discussed in terms of comparing the degrading and non-degrading models subjected to this multiple earthquake sequence. Furthermore, the plastic hinge distributions, residual displacements, and interstory drift ratios are evaluated by applying the aforementioned material level-based model.

2. Earthquake ground motion sequences

A devastating earthquake with a moment magnitude (M_w) of 9.0 occurred at 14:46 (JST GMT + 9) on March 11, 2011, in Japan. This high-M_w earthquake is categorized among the most powerful excitations in the world since the 1900s, when modern record keeping began [10]. The Japan Meteorological Agency (JMA) [11] reported that the focal mechanism of this earthquake excitation was a reverse fault with a compression axis in the east-to-west direction at a depth of 24 km. The earthquake occurred at the plate boundary between the North American and Pacific plates [12].

It should be noted that this earthquake is unique because of its foreshocks and aftershocks. Zhao [13] highlighted that the earthquake sequence started with a 7.3-magnitude foreshock two days before the mainshock, which triggered vigorous aftershocks. Kazama and Noda [14] reported 593 aftershocks in a 3-month period between March 11 and June 11, of which five had magnitudes of 7.0 or greater. Certainly, the Tohoku excitation, with a magnitude of 9.0, greatly increased the seismic activity in the broad regions in and around the Japanese archipelago. More than 10,000 aftershocks with magnitudes of at least 3.0 occurred in the forearc area [15]. The epicenter of the aforementioned ground excitation, the foreshock, and the aftershocks with magnitudes greater than 7.0 are marked on the map in Fig. 1.

In this study, 23 stations that recorded the strong ground motions are considered. The locations and maximum peak ground acceleration (PGA) values of these stations are illustrated in Fig. 1. Records were acquired from the National Research Institute for Earth Science and Disaster Prevention (NIED) [16] data bank. The reader is referred to the cited document for more information regarding the strong ground motion parameters. The distances from the considered stations to the epicenter, soil properties upon which the stations are built, and PGA values are the key parameters for selecting the records. Additionally, to overcome the near fault effects, only records of stations that are at least 20 km away from the epicenter have been considered. Furthermore, liquefaction effects are neglected. Hence, records having shear wave velocities for the top 30 m of the subsurface profile ($V_{s,30}$) in the range of 360–800 m/s are used in the nonlinear dynamic analyses. For brevity, the authors decided to consider the earthquake ground motion sequences having PGA values between 0.20 g and 0.80 g. Values exceeding 0.80 g are omitted owing to their destructiveness. Therefore, 43 ground motion sequences have been selected for use in the conducted nonlinear dynamic analyses. The vertical effects of the earthquake sequences have been neglected in accordance with FEMA 356 [17].

The spectral displacement (S_d), spectral acceleration (S_a), pseudo spectral velocity (PS_v), and pseudo spectral acceleration (PS_a) graphs of the sequences that are used in this study are plotted in Fig. 2. Additionally, plots of the mean, mean + standard deviation and mean – standard deviation are presented in Fig. 2. For comparison, the response spectrum for soil type B of Eurocode 8 (EC8) [18] is plotted in the same figure. It can be inferred from the figure that the mean response spectra of the selected earthquake ground motions are in accordance with that of EC8 [18] for low period.

3. Description of case-study buildings

In this study, three plan-asymmetric RC buildings are considered: Seismic Performance Assessment and Rehabilitation of Existing Buildings (SPEAR), Innovative Concepts for Seismic Design of New and Existing Structures (ICONS), and a school building in Van, Turkey. The SPEAR and ICONS frames were a part of an extensive experimental investigation research program funded by the European Union (EU). Both buildings have been designed and built to represent RC structures with no seismic detailing. The school building was designed in accordance with Turkish Earthquake Code (TEC) [19]. It should be strongly emphasized that the majority of existing RC buildings in Turkey are similar to that studied herein. Further details regarding the selected irregular buildings are given in the following sections. Moreover, to consider the seismic response of irregular RC buildings under multiple earthquake excitations, the selected structures are analyzed for two cases. In the first case, they are analyzed as designed; in the second case, the geometry is modified without altering the overall stiffness of the structures.

3.1. SPEAR building for as designed and modified cases

The SPEAR building was built for performing pseudo-dynamic tests in the European Laboratory for Structural Assessment (ELSA) in Ispra, Italy. The structure was designed by Fardis [20] according to the construction practice and materials used in Greece in the early 1970s. The building was built irregularly in plan, but it was regular in elevation. It had three stories, and the story heights were 3 m. It has two bays in the horizontal (X) and transverse (Y) directions. The structure was tested in 2004 using a full-scale pseudo-dynamic test [21]. The plan layout, elevation and the reinforcement detailing of the building are shown in Fig. 3. As highlighted in this paper, the single column (C6) with a cross section of 25 cm × 75 cm makes the structure stiffer and stronger along the Y-direction.

The compressive strength of concrete [22] and yield strength of steel [23] are selected as 25 MPa and 400 MPa, respectively. The calculated torsional characteristics according to EC8 [18] are presented in Table 1, where e_{ox} and e_{oy} are the eccentricities measured along the X- and Y-directions, respectively, r_x and r_y are the torsional radii measured along the X- and Y-directions, respectively, and l_s is the radius of gyration of a floor in plan. More details regarding the structural properties of the building can be found in the studies by Stratan and Fajfar [24] and Papanikolau et al. [23].

Owing to the large eccentricities in the Y-direction, it was decided that the structure be remodeled and the regular case be performed in this direction. In order to not alter the rigidity of the structure and retain the same stiffness value, the dimensions of the columns circled in Fig. 4(i.e. C6, C7, and C8) are altered. Consequently, the stiffness values of the structure in the Y-direction are calculated as approximately 152 kN/m and 149 kN/m, before and after remodeling the building, respectively. Furthermore, the lateral stiffness difference ratio of the