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Investigation of progressive collapse resistance and inelastic response for an earthquake-resistant RC building subjected to column failure

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ABSTRACT

Following the linear static analysis procedure recommended by the US General Service Administration (GSA), the potential of an earthquake-resistant RC building for progressive collapse is evaluated in this study. Nonlinear static and nonlinear dynamic analyses are conducted to estimate the progressive collapse resistance of the building subjected to column failure. Under an approximate deflection demand, different collapse resistances are obtained. It indicates that different criteria for estimating the collapse resistance may be adopted for these two nonlinear analysis methods. The nonlinear static approach leads to a conservative estimation for the collapse resistance if "2.0" is used as the dynamic amplification factor (DAF). As the column-removed building is loaded into a significantly yielding phase, different assessed results are obtained by the linear static method and the nonlinear acceptance criterion suggested by the GSA guidelines. A DAF considering the inelastic dynamic effect may be needed in the GSA linear procedure. The capacity curve constructed from the nonlinear static analysis is shown to be capable of predicting the progressive collapse resistance and the DAF of a column-removed RC building.

1. Introduction

Many practicing engineers and academic researchers have been engaged in the prevention of progressive collapse since the partial collapse of the Ronan Point apartment building in 1968. Especially after the malevolent bombing of the Murrah Federal Building in 1995, several changes to the philosophy and practice of design for important buildings have been made in the last decade. Resistance of building structures to progressive collapse has been an important task for the development of structural design codes. Some study results, code approaches, and design strategies or standards have been reviewed, discussed, and/or compared in the literatures [1–7]. Generally speaking, the investigated issues include abnormal loading events, assessment of loading, analysis methods, and design philosophy. In recent years, the development of analysis methods for evaluating the progressive collapse potential of an existing or new building has been an imperative subject. Linear static, nonlinear static, linear dynamic, and nonlinear dynamic methods are four basic approaches for the progressive collapse analysis. Advantages and disadvantages of these approaches have been discussed by Marjanishvili and Agnew [8,9]. Detailed descriptions of a step-by-step, linear static procedure for progressive collapse analysis have been issued by the US General Service Administration (GSA) [10] and Department of Defense (DoD) [11]. The GSA linear static analysis approach has been applied to evaluate the potential of a steel moment frame and a RC frame for progressive collapse [8,12].

Terrorist events are quite few in the history of Taiwan. Even so, the potential hazard of terrorist attacks always exists because of the trend of globalization. Since Taiwan is located in an earthquake-hazardous region, most of the RC buildings are detailed according to the seismic design code. Some studies indicated that seismic design detailing might help to enhance the resistance of buildings against progressive collapse [13–15]. Hence, seismically designed RC buildings are expected to have low potential for progressive collapse. In this paper, the progressive collapse potential of an earthquake-resistant RC building under four threat-independent, column-removed conditions is evaluated by using the GSA linear static analysis procedure. Nonlinear static and dynamic analyses are carried out to verify the linear analysis result and estimate the progressive collapse resistance for each column-removed condition. The catenary effect is neglected and only the flexural failure mode is considered herein. Dynamic effect on the assessed results obtained from the linear or nonlinear static method is discussed. Application of the nonlinear static method to the estimation of the progressive collapse resistance and the dynamic amplification factor (DAF) of a column-removed building is proposed and demonstrated.

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Table 1
Dimensions of RC member sections (cm)

Floor	Column	Beam	Joist
1F	70 × 100	50 × 90	30 × 65, 20 × 50
2F–11F	70 × 90	50 × 75	30 × 65, 20 × 50

2. Descriptions and modeling of the RC building

2.1. Descriptions

The RC building is an 11-storey, moment-resisting frame structure with a 2-storey basement. Its first storey is an open space for the public. The center-to-center plan dimensions are 17.75 m in length and 12.25 m in width from the ground floor to the roof, as shown in Fig. 1. There are three bays with center-to-center span length arranged as 5.6 m, 6.55 m, and 5.6 m in the longitudinal (west–east) direction, and two bays with a 6.6 m and a 5.75 m span in the transverse (north–south) direction. The storey height is 3.8 m for the first storey and 3.2 m for the others. In addition to the self weight, a dead load (DL) of 0.98 kN/m^2 is applied to the roof and 0.245 kN/m^2 to other floors. The service live load (LL) is 4.91 kN/m^2 for the roof and 1.96 kN/m^2 for other floors. Conventionally, the structural design consultants in Taiwan use larger imposed DL and LL on the roof to account for the loading of special waterproof roofing and some accessory facilities (e.g. water reservoir, ventilation system, etc.), respectively. Table 1 presents the section dimensions of the RC members for the building. A compressive strength equal to 27 500 kN/m^2 is used for the concrete. The design yield strength is 412 000 kN/m^2 for the main reinforcements and 275 000 kN/m^2 for the stirrups.

The building is located at a soft soil site and its design spectral response acceleration, S_{aD}, is equal to 0.47g estimated at the fundamental period. All the beams and columns are designed and detailed according to seismic code requirements. The beam members have at least three continuous #10 steel bars for the top and bottom reinforcement. As required by the seismic demand, more #10 top and bottom bars are provided and continuous through the column lines at the beam–column joints. The positive moment strength at each joint face is larger than one half of the negative moment strength at that face of the joint. Also, sum of the nominal flexural strengths of the columns framing into a joint is at least 1.2 times larger than that of the beams framing into the joint. Hence, a strong column-weak beam mechanism may be ensured. If any interior column on the ground floor is removed, the two-span beam will redistribute loads to adjacent columns. Flexural hinges may form at the two ends of the beams when they cannot resist the instantaneous loading in an elastic manner. If the plastic hinge strength is insufficient to sustain the loading, the beam deflection will further increase to mobilize catenary tensile action, which is the final protection against collapse.

2.2. Structural modeling

A beam–column frame model is constructed for the RC building using the SAP2000 commercial program [16]. The model is assumed to be fixed on the ground. Self weight of the exterior walls is distributed to the spandrel beams. Also, self weight of the interior walls and partitions is estimated and applied to the floor slab as a distributed load. Thereafter, according to the tributary area, self weight of the slab and all the dead loads and live load on it are distributed to the beam elements for each floor. The fundamental period of the building model is equal to 1.35 and 1.34 s in the longitudinal and transverse direction, respectively.

The reinforcement disposition of each member section is simulated based on the design drawings of the RC building. There are twenty types of reinforcement disposition and nine different spacing of shear stirrup for all beam sections. The nominal moment and shear strength vary from 620 kN m to 1460 kN m and 730 kN to 920 kN, respectively, for the beam members. Flexural plastic hinges are assigned to both ends of beam elements. Default moment-hinge properties based on the FEMA-273 guidelines [17] are adopted for the hinge model. Different performance levels are represented by circular symbols with different shadows, as shown in Fig. 2. Although, as recommended by the GSA guidelines, strength increase factors for material properties may be used in the analysis, they are not considered in this study. Preliminary studies [18] indicated that collapse of the RC building under column-removed conditions is governed by the flexural failure mode of beam elements. Also, the column members remain elastic even when the ultimate moment capacities of the connected beam sections have been developed. Hence, shear failure is not considered and the column members are assumed to be elastic in this study.

3. Progressive collapse potential

3.1. Loading and criterion

A downward loading combination

$$P_{st} = 2(\mathrm{DL} + 0.25\mathrm{LL}) \tag{1}$$

recommended by the GSA guidelines is adopted for the linear static analysis. DL includes the structural weight and additional dead loads. P_{st} is defined as the GSA loading herein. In the linear static analysis, the GSA loading is applied to the RC building subjected to column failure, and the demand-to-capacity ratio (DCR) of flexural moment is calculated to assess the progressive collapse potential. Since the building has a typical structural configuration, the acceptance criterion for the primary structural components is $DCR \leq 2.0$. When the DCR value is larger than 2.0, a hinge has to be inserted to the member end for releasing the moment. For dynamic analysis, the DAF, "2", in Eq. (1) is omitted and the downward loading is changed to

$$P_{dy} = (\mathrm{DL} + 0.25\mathrm{LL}). \tag{2}$$

3.2. Collapse potential

Four threat-independent, column-removed conditions, designated as Case 1B, Case 2A, Case 1A, and Case 2B, are considered for the building. According to the bay line numbers shown in Fig. 1, the removed column of the first storey is 1B, 2A, 1A, and 2B for Case 1B, 2A, 1A, and 2B, respectively. Linear static, nonlinear static and nonlinear dynamic analyses are carried out to investigate the column failure responses of the building. 5% inherent damping ratio is assumed for the dynamic analysis. Similar to the results observed by Sucuoglu et al. [19], most of the downward loading originally sustained by the failed column is transferred to the plane frames intersecting at the line of the failed column. Therefore, the DCR values, plastic hinge distribution, and deflection of those intersected frames are the major concerns in this paper.

Fig. 3a shows the DCR values and the plastic hinges obtained from the GSA linear static and nonlinear static analysis for Case 1B, respectively. Force-controlled method with force magnitude equal to the GSA loading (P_{st}) is used for the nonlinear static analyses. Moment distribution of the linear static analysis is presented in Fig. 3b. Because of different local axis definitions, the column moments are not displayed for the B–B frame. It is seen that there is no DCR value larger than 2. The column-removed building has low potential for progressive collapse and no moment-released