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Efficient progressive collapse analysis for robustness evaluation of buildings experiencing column removal

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ABSTRACT

Progressive collapse analysis is used to evaluate robustness of buildings against unexpected extreme events and is a highly nonlinear dynamic problem. Motivated by the need for realistic and yet reasonably fast analysis, a "middle-way" approach for efficient progressive collapse analysis (ePCA) is explained in this paper, accounting for column buckling, semi-rigid connection and membrane action of slab. The accuracy of ePCA is validated by comparing with experimental results available in the literature. Application of ePCA on realistic building systems shows that robustness can be improved cost-effectively by introducing minor changes to the steel connection and slab reinforcement. The computational efficiency of this method enables incorporation of robustness design in building system in the early stage of design process. It can also be useful for researchers to more efficiently evaluate the behavior of different structural systems.

1. Introduction

Structural robustness may be defined as the insensitivity of a structure to local failure [1], particularly for events which we do not anticipate or have full control. A key aspect is the evaluation of structural robustness to avoid disproportionate failure against unexpected events. Progressive collapse typically involves a chain of reactions that commence with the failure of one or several structural components leading to redistribution of internal forces and causing other components to fail in sequence. Due to the extreme loading (caused by severe earthquake or blast for example) and dynamic nature of the progressive collapse process, equilibrium may only be archived when a considerable part of the structure has been damaged or even collapsed locally. If the collapse area is substantial due to a minor triggering event, the phenomenon is known as disproportionate collapse and the structure is deemed not robust.

Most design codes have traditionally focused on inherent properties of structures such as redundancy and ductility. More recent efforts have been directed on performance based evaluation of the structural system e.g. by considering removal of critical structural member such as a column of a multi-storey building [2–4]. Accordingly, one needs to quantitatively analyze the response of a damaged structure caused by sudden removal of a column, and re-design if necessary to provide sufficient robustness such that damage is contained within a limit proportionate to its cause. It is imperative to include material damage and geometry nonlinearity for realistic performance evaluation. This often involves dynamic effects and complex interaction between different structural components. To address these challenges, the use of state-of-art commercial software is common in the literature, e.g. Alashker et al. [5], Kwasniewski [6], Yu et al. [7], Fu [8] and Sadek et al. [9] etc., among others.

Fu [8] studies the structural behavior of a three-dimensional 20-storey composite steel-framed building under sudden column removal event using finite element software ABAQUS [10]. All beams and columns are simulated using beam elements, while slabs and walls are modeled as 4-node shell elements. Steel reinforcement in slab is assumed to act as a smeared layer. For concrete components, the material properties are modeled based on concrete damage plasticity model and nine integration points are used for each shell element to capture concrete cracking behavior. Steel member connections are assumed to be fully pinned and no damage behavior is assumed for all connections. Therefore, the effects of connections are not considered in the study. Kwasniewski [6] presents a case study of progressive collapse analysis of an 8-storey building using finite element analysis software LS-DYNA [11]. The study utilizes the advantage of parallel processing on multiprocessor computers to analyze detailed three-dimensional model with 1.08 million finite elements. All beams and columns including the flush and fin plate connections are modeled by shell elements. This detailed model captures the local effects such as inelastic bending of end plates or local buckling of compressed flange. Component disintegration is represented by deletion of a finite element from further calculation. The 130-mm orthotropic composite slab is modeled using 4-

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node shell elements and is divided into two types of strips, with different overall cross-sectional properties, and positioned alternatively side by side. Each strip is modeled as a multilayer composite using a sophisticated isotropic elastic-plastic model [11]. The proposed detailed finite element analysis is not applicable in practice due to the enormous computational demand. For example, a case of column removal would require 19 days of computational time using 60 parallel processors [6].

Yu et al. [7] studies the progressive collapse behaviors of composite floor system due to push-down experiment of the perimeter column using LS-DYNA [11]. Beams and columns are modeled as beam elements whereas metal decks are modeled as shell elements. Concrete slabs are modeled as constant stress 8-node solid elements. The slip between metal deck and concrete is ignored. A simplified approach is adopted to model pin connections, semi-rigid connections and hinged connections. Semi-rigidity and partial-strength of structural connection are not considered in the study. Sadek et al. [9] studies the robustness of composite floor system with simple shear connection under internal column removal event using LS-DYNA [11]. All beams, columns and metal decks are modeled by shell elements. Concrete slabs are modeled as solid elements with a sophisticated concrete damage model in LS-DYNA. All steel components including slab reinforcements are modeled by truss elements with bilinear stress-strain relationship. Shear studs are modeled as beam elements embedded in the concrete slab. Multiple contact constraints are defined between the concrete slab and metal deck, and the metal deck and top flanges of steel beams. The detailed numerical model consists of 295,000 shell and solid finite elements. In a related study, Alashker et al. [5] uses the same numerical model proposed by Sadek et al. [9] to investigate the influence of deck thickness, slab reinforcement the connection design on global progressive collapse resistance of the floor system when subjected to uniform floor load. Such detailed numerical models are difficult to apply in practice due to the enormous computational demand.

Detailed FEA models aim to accurately capture nonlinear dynamic behavior but require high computational demand including intensive pre/post-processing efforts. Therefore, this approach is not suitable for application in design offices. At the other end of the spectrum, simplified FEA involving macro-models are easy to use but do not simulate progressive collapse well. Marjanishvili and Agnew [12] compare the effectiveness of linear static, nonlinear static, linear dynamic and nonlinear dynamic analyses for sudden column removal events. In their study, material damage is considered using simplified plastic hinge approach while geometry nonlinearity is ignored. The simplification prevents the development of catenary action which usually prevails when the floor system is loaded to large deformation. In addition, the influence of beam-to-column connections is also ignored in the study. Kaewkulchai and Williamson [13] developed beam-column elements considering multi-linear, lumped plasticity hinge model with axial-bending interaction. The proposed model considers only planar frame structures and is unable to capture the effects of floor slabs and connections as well as three-dimensional frame action. Khandelwal and El-Tawil [14] study the vulnerability of moment resisting frames to sudden removal of internal column using macro-model numerical analysis. In their study, mechanical models are used to represent the semi-rigid partial-strength behavior of beam-to-column steel connections, but the influence of floor slabs is ignored. These simplified methods typically ignore the catenary action of floor slabs on progressive collapse, thereby not providing realistic evaluation of structural system robustness. Furthermore, buckling behavior and connection behavior are often not modeled realistically in these simplified methods.

The need for fast and reasonably accurate analysis has led to the proposed efficient progressive collapse analysis (ePCA) of buildings in this paper. The main purpose of ePCA is to provide practicing engineers with a practical method for robustness evaluation, involving realistic and efficient modeling of damage behavior of main structural components of a steel-concrete composite building. One of the main benefits of ePCA is that the failure behaviors of main structural components (steel frame members and connections and concrete floor slabs) are modeled consistently using the same plastic zone method. Even though these structural components may have different properties (i.e. stiffness and resistance under axial and flexural forces), these properties can be represented consistently by using the appropriate fiber sections as will be shown in the following sections. The consistency not only makes it easier for users to use only a single failure model but also avoids the use of sophisticated constitutive model to account for material failure.

2. Modeling of frame members

Frames are likely the most common forms of man-made engineering structures. To account for the influence of buckling in compression members, both member buckling and global buckling should be included in the progressive collapse analysis. A nonlinear frame model capable of capturing buckling and post-buckling behaviors of steel member is shown in Fig. 1. The model is based on the plastic zone method that requires discretization of a section into fibers and a member into nonlinear frame elements. The sectional stiffness is obtained by assuming plane section remains plane and summing the stiffness of those fibers across the section. In this way, progressive spreading of plasticity across section and along member can be accurately simulated. Residual stress is included into stress-strain relationship of individual fiber according to measured profile or simplified profile recommended by European Convention for Constructional Steelwork (ECCS) [15]. For geometry imperfection, explicit modeling of initial out-of-straightness e_0 is adopted according to the principal buckling mode shape. The main assumptions of the proposed method are: (1) plane sections remain plane after deformation, (2) lateral torsional buckling is prevented, (3) local buckling of cross-section is prevented, (4) effect of shear on yielding of the material and deformation is negligible, and (5) residual stress is uniformly distributed over the entire length of a member.

The nonlinear frame model is implemented using an established finite element analysis software SAP2000 [16]. For illustration, a steel member shown in Fig. 1 is discretized into 14 elements, of which eight are inelastic (hatched region) and the remaining six are elastic elements. For the inelastic element, distributed plasticity is simulated by using a fiber hinge located at the center of the element (the monitored location shown in Fig. 1). In each inelastic element, distribution of plastic moment and curvature are assumed to be constant. Therefore, a sufficiently small element length is required within the inelastic region for accurate simulation of spread of plasticity along the steel member. Inelastic element requires significantly greater computational resources than elastic elements. Therefore, inelastic elements are used only at critical locations where inelasticity is possible. For a general nonlinear frame member of length L subjected to axial force and bending moment, the critical locations are most likely at the mid-span and member ends. The hatched region shown in Fig. 1 is defined as the plastic zone length (L_p), representing the region where inelasticity can take place. Based on a parametric study, $L_p \geq L/2$ can be used while element length within L_p of $L/24$ or less for a good compromise between efficiency and accuracy. In the elastic zone, a larger element length (say $L/12$) can be used. At the monitored location, the member cross-section is discretized into fibers as shown in Fig. 1. The uniaxial response of the fibers is assumed to be elasto-plastic with strain-hardening behavior. The sectional constitutive relations are obtained by summing uniaxial stresses of all fibers across the section by assuming that plane section remains plane. The use of fiber with uniaxial stress-strain relation would capture the axial-bending interaction in a simple and efficient way without the need of 2D or 3D constitutive model.

2.1. Post-buckling of frame member

Energy-absorbing capacity is one of the key factors that govern the robustness performance of a structural system. For slender members, the energy-absorbing capacity determines the post-buckling response.