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Assessment of progressive collapse potential in corner floor panels of reinforced concrete buildings

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ABSTRACT

The Department of Defense (DoD) document, UFC 4-023-23, which provides technical guidance for mitigation of progressive collapse, classifies buildings based on the desired level of protection. Medium and high levels of protection categories require the use of the Alternate Path (AP) method to investigate the capability of the structural system to transfer loads safely from a notionally removed column to the remaining structural elements. Certain columns and structural elements at prescribed locations must be investigated to determine the structural bridging capabilities over the removed column. Transfer of loads from a notionally removed corner column to the adjacent structural elements can impose significant stress/deformation demand on structural elements supporting the corner panel. When the panel area exceeds the floor damage limits, the panel and its structural elements must be designed to support the additional load or the loads must be transferred to adjacent columns. This paper investigates the implementation of UFC 4-023-23 to protect against progressive collapse of corner floor panels when their dimensions exceed the damage limits. A case study of a reinforced concrete building is analyzed, designed, and investigated using the AP method.

1. Introduction

Although UFC4-023-03 [2] is intended for the design of DoD facilities, it is the design document with the most detailed requirements for protection against progressive collapse and may provide a foundation for provisions that can be incorporated into general building codes such as the International Building Code (IBC). UFC limits the need for design against progressive collapse to buildings of three stories or more. The mitigation strategies discussed in this paper are for progressive collapse caused by the loss or damage of a structural element regardless of the cause. The strategies described in this paper are based on UFC and do not address strengthening of specific elements to prevent damage. A literature review on progressive collapse mitigation in codes and standard can be found is available [3].

Two levels of protection against progressive collapse are discussed in the UFC: (a) catenary action provided to the structure by tie forces in vertical and horizontal elements, and (b) flexural bridging capabilities that allow the structure to support loads transferred to adjacent elements from a notionally removed element using the AP method. The catenary action tie force requirements can generally be satisfied with limited calculations and good connection details.

Structures classified in the category of Very Low Level of Protection (VLLOP) or Low Level of Protection (LLOP) are protected against progressive collapse by providing adequate tie forces to develop catenary action in case of loss of structural element. If the specified vertical tie force cannot be provided, the AP method is used to ensure the structure is capable of bridging over a damaged element.

Structures classified in the category of High Level of Protection (HLOP) or Medium Level of Protection (MLOP) must have adequate tie forces in the vertical and horizontal directions, in addition to bridging capabilities over certain vertical elements. Predefined locations are specified for notional removal of columns to check for bridging capabilities. Fig. 1 shows the locations of external columns that are to be removed notionally to conduct a UFC AP investigation of a framed structure as well as the corner column that will be removed for the purposes of this study. UFC requires a separate AP investigation for each designated column at each floor level. Each analysis must demonstrate that the structure has the bridging capabilities after the notional removal of each selected column. If the structure cannot provide the bridging capabilities, elements must be redesigned. For large structures, this requirement can be very time-consuming.

A number of researchers has studied two-dimensional and three-dimensional modeling of structures for AP investigation of progressive collapse potential. Powel [4] concluded that static analysis is more conservative and that dynamic analysis is more accurate and no more difficult than static analysis.

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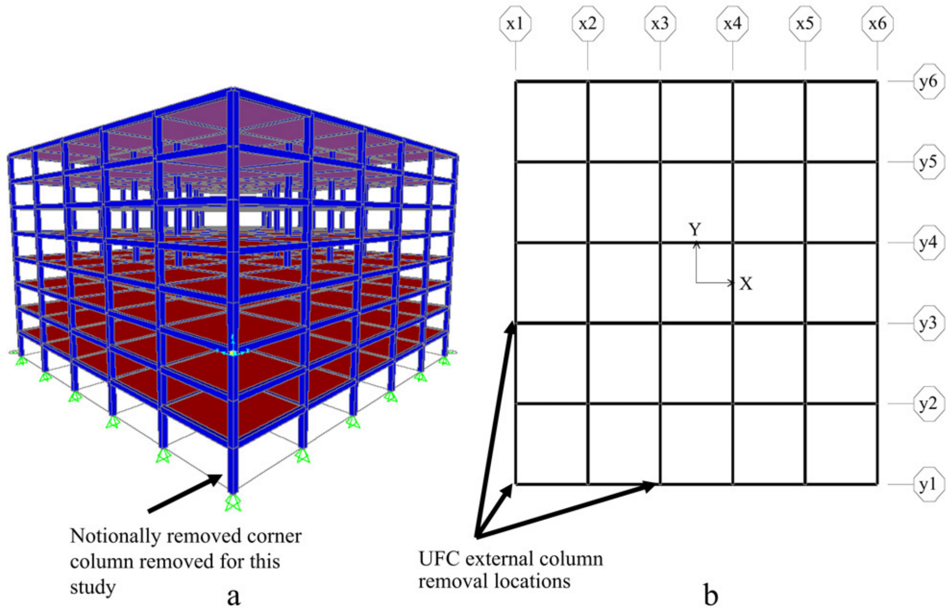

Fig. 1. (a) 8-story reinforced concrete column case study with corner column to be removed in AP investigation shown, (b) External columns removed for AP analysis to investigate potential of a framed structure for progressive collapse based on UFC [1].

Hansen [5] studied the performance of three-dimensional models of external columns in reinforced concrete buildings. The authors produced response histories for edge beams using nonlinear dynamic analysis to simulate the loss of exterior columns. The authors also demonstrated that nonlinear dynamic analysis is important for progressive collapse investigations to capture a realistic structural response.

Marjanishvili and Agnew [6] compared linear-static, linear-dynamic, nonlinear-static, and nonlinear-dynamic analysis methods for progressive collapse analysis based on the General Services Administration (GSA) provisions. The authors argued that GSA performance limits for linear analysis methods are not conservative and that linear-static and linear-dynamic analysis methods produce comparable maximum deformations.

Marchand [7] presented an extensive coverage of analysis and design methods for mitigation of the effects of progressive collapse and blast. Performance expectations in GSA and DoD guides are were also discussed in this paper.

Mohamed [8] pointed out the existence of torsion stresses at corner columns that are notionally removed during AP progressive collapse analyses. Torsion shear stresses may cause brittle beam failure and should therefore be removed from the model during AP analysis.

Kaewkulchai and Williamson [9] demonstrated that when dynamic analysis is used to assess the potential for progressive collapse of frames, the use of either the initial configuration or the deformed configuration does not significantly affect the structural response.

Iwankiew and Griffis [10] studied the collapse characteristics of various buildings in recent history. The authors argued that similar distress mechanisms are exhibited by steel and concrete buildings when collapse is caused by fire in combination with impact. They also pointed out that the architectural layout of the building could play significant in role in reducing or exacerbating the number of causalities caused by progressive collapse.

Grierson et al. [11] presented a method for conducting linear static progressive collapse analysis based on the provisions of the United States General Services Administration (GSA) [12]. The authors modeled the reduced stiffness during progressive collapse using and an equivalent-spring method.

2. The modeling process

In order to demonstrate the implementation of UFC, the stress and deformation demand resulting from the application of the AP method, a case study is modeled, analyzed, and designed. The following sections describe the modeling process.

2.1. Column heights

According to section UFC 2-2.1 and for all levels of protection, the laterally unsupported length of all columns used in the analysis and design should be equal to the height of two stories. This requirement is to ensure that each column is able to support vertical loads after loss of lateral support at any floor level. Column length to be removed for AP analysis is the clear length between lateral restraints. Engineers have interpreted section 2-2.1 to mean that the column unbraced length used in design should be equal to the height of two stories. However it is conservative to interpret the reference to *analysis* in section 2-2.1 to mean that columns equal to the height of two stories can be used to obtain column design forces and to design the columns.

2.2. Over-strength and strength reduction factors

A factor $\Omega = 1.25$ is applied to the concrete compressive strength, f_c', and to the yield strength of reinforcing steel, f_y, in the process of calculating the nominal flexural resistance, M_n, and shear resistance, V_n.

Strength reduction factors: The strength reduction factor, ϕ, for flexure, shear, axial, and torsion as specified in the ACI 318 code.

2.3. Strength and deformation limits

Structural elements and connections must meet the strength limits in Table 1 and deformation limits in UFC, Table 4-4 [2]. The deformation limits are listed in terms of maximum rotations for beams and slabs and in terms of ductility for columns. For HLOP and MLOP analyses, the maximum permissible rotation for reinforced concrete doubly reinforced and shear-reinforced beams when tension membrane is not included in the analysis is $\theta = 4°$.