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Probabilistic Robustness Assessment of Structures under Sudden Column-Loss Scenario

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***Abstract*—**This paper presents a probabilistic incremental dynamic analysis (IDA) of a full reinforced concrete building subjected to column loss scenario for the assessment of progressive collapse. The IDA is chosen to explicitly account for uncertainties in loads and system capacity. Fragility curves are developed to predict the probability of progressive collapse given the loss of one or more columns. At a broader scale, it will also provide critical information needed to support the development of a new generation of design codes that attempt to explicitly quantify structural robustness.

***Keywords*—**Incremental dynamic analysis, progressive collapse, structural engineering, pushdown analysis.

I. Introduction

PROGRESSIVE collapse or disproportionate collapse means the sequential spread of damage from an initiating event, such as fire, blast, impact or the consequences of human error, from element to element, resulting in the failure of a number of elements that is disproportionate to the initiating damage. On the other hand, robustness is the ability of a structure to withstand such events, without being damaged to an extent disproportionate to the initiating damage.

Collapse prevention and life safety of the occupants are of utmost importance in structural engineering. Accordingly, there is an increasing interest to quantify robustness for buildings that are susceptible to primary element loss, due to accidental or abnormal events. Such a measure could be used to provide the practical means of quantifying the desired system performance, which could then be tied to the economic losses or potential casualties.

There has been an increasing interest into understanding progressive collapse in the past decade, especially after the events of September-11. Furthermore, the general design methods do not consider extreme load cases, so there is a need to understand the structural behavior and to evaluate the robustness under such conditions. The initiating events are often unexpected and are not appropriately accounted for in the current design codes. Some studies argue that for slender high-rise buildings, a local failure will cause the simultaneous failure of all vertical members. Hence, the global failure probability will be very high if a local failure is initiated [1].

This paper will present different methods of analyzing structures for progressive collapse. It will also discuss the methods of quantifying structural robustness. A case study building will be subjected to sudden column removal and analyzed by using IDA. Based on the IDA curves fragility curves will be developed to give the probability of exceeding given three limit states.

The IDA is used because it is the most accurate in simulating the progressive collapse of the building. Fragility curves give the probability of progressive collapse for the loss of one or more columns.

A. Method of Assessing Progressive Collapse Potential

1. Numerical Simulations

Many studies have been done on progressive collapse in recent years. Most of them focus on modelling of the structure at the component level without consideration of the global structure. The best way of simulating progressive collapse is by using Nonlinear Dynamic Analysis with consideration of large displacements and material nonlinearity. Fascetti *et al*. [2] mention that the use of high fidelity models is computationally prohibitive and macro models are preferred in collapse simulations of reinforced concrete (RC) structures. Mainly because it has been shown that this kind of approach is reasonably accurate from an engineering standpoint and can replicate the global behavior of the structure, however careful calibration of the models is needed. Macro models refer to frame element models of the structure composed of one-dimensional beam elements with fiber-section discretization at integration points along the member length.

One of the most common methods of robustness assessment in international codes is the use of the alternate path method (APM) as discussed in [3]. APM is simple method that does not explicitly model the loading but it evaluates the collapse resistance of the system by removing critical load bearing elements. Fascetti et al. [2] argue that APM does not provide information on the proximity to failure of the system, for example a structure that passes APM could still be on the verge of collapse.

Abruzzo et al. [4] demonstrated that by using static push down analysis with considerations on energy conservation that a building that exceeds ACI integrity requirements but designed for low seismic forces can be still vulnerable to progressive collapse.

Marjanishvili and Agnew [5] compare four methods for progressive collapse analysis by analyzing a 9-storey steel moment-resistant frame building. The four methods are: linear

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static, nonlinear static, linear dynamic and nonlinear dynamic methods. They also give detailed steps for performing all analyses in SAP 2000. Contrary to the GSA Guidelines, where they discourage the use of nonlinear dynamic analysis, the authors of the papers state that using nonlinear dynamic analysis is more accurate and also easy to perform by using modern FEM software. Powell [6] also confirms that the static analysis for progressive collapse is too conservative, also stating that performing nonlinear dynamic analysis using current FEM software gives better accuracy and it is not any harder.

2. Experimental Tests

Other than numerical simulations, experimental testing is the other method for robustness assessment. However, limitations in the majority of the testing facilities combined with the cost of testing complete full-scale structures have made experimental testing rare. Previous tests have been mainly limited to either small scale structures or conducting only two-dimensional (2D) tests. In the rare cases where full-scale structures were tested, the instrumentation is often limited with fewer column removals. There are two significant studies that performed experimental testing for progressive collapse. First, Xiao et al. [7] built a half scale three story RC building and subjected it to sudden column collapse. The experiment was unique and the first of its kind, it was designed for a moderate level of seismic resistance. The observed shift in the collapse-resisting mechanism from frame action to catenary action was demonstrated in the experiment. Further, they explained the behavior using simple analytical models with yield lines and plastic hinges.

A second experiment was performed by Lew et al. [8] to study the assembly of an exterior RC beam-column frame of a full-scale model under column removal scenarios. The building represents an actual 10-story building designed for seismic design categories (SDC) C and D. Their results provided an insight into the behavior and failure modes of reinforced concrete assemblies under column removal scenarios. Thus, the results should add value into disproportionate collapse-resistant design.

Another experiment was conducted by [9], where they statically tested a reinforced concrete frame subjected to column loss. The frame is a one third scale of a 4 bay and 3 story segments out of a larger structure. The 'lost column' was simulated by unloading a mechanical jacking system. They studied the mechanical behavior, redistribution and transition of the load resisting mechanics. They concluded that the calculated capacity of the frame using the plastic limit state was approximately 70% of the tested failure capacity if catenary action is also included.

B. Pushdown Analysis

One of the common assessment methods in seismic assessment of structures is the "pushover analysis" wherein a structure is subjected to a monotonically increasing lateral load pattern till a failure mechanism is formed. This method is important in studying the formation of plastic hinges and tracking down the reserved capacity in the system beyond the required seismic demands. Similarly, for progressive collapse simulations, the method of "pushdown" analysis is utilized for structural assessment. A pushdown analysis is an incremental non-linear static procedure in which a downward load of increasing intensity is applied to the structure until the occurrence of structural collapse.

There are three methods of performing the pushdown analysis as described by Khandelwal and El-Tawil [10]. The three are: Uniform Pushdown (UP), Bay Pushdown (BP), and Incremental Dynamic Pushdown (IDP). The capacity of the structure for each of these methods is expressed in terms of the overload factor (OF), which is defined as the ratio of failure load to the nominal gravity loads. It is used to evaluate the robustness of the structural system in question.

In the UP case, gravity loads in the entire structure are increased proportionally within a nonlinear static analysis framework until the system collapses. This method might lead to a collapse state corresponding to failure of the weakest part of the damaged structure, however, failure may also occur outside the damaged bays.

The BP method is more focused on the damaged bays. In this method, the gravity load is increased proportionally only in the bays that have been damaged until the system collapses. The rest of the structure is only subjected to nominal gravity loads. Therefore, this analysis will lead to a collapse state corresponding to failure in the damaged bays. The residual capacity of the system is measured in terms of the OF.

The IDP method is inspired by the IDA method used in earthquake engineering. In IDP, successive dynamic analyses with increasing gravity loads above the lost column of interest are conducted until an OF corresponding to failure is established. In each dynamic analysis case, the system is first assumed to be undamaged while the loading is being applied. Then, the member designated as 'lost' is instantaneously deleted and the system is allowed to respond in an inelastic manner. This analysis method explicitly accounts for dynamic effects unlike the UP and BP methods. However, it is costly in terms of required computational effort since multiple nonlinear dynamic analyses must be conducted.

The paper goes on to demonstrate advantages and disadvantages of the three methods of pushdown analysis by applying them to a 10-story steel moment frames designed for moderate and high levels of seismic risk [10]. Based on their conclusions, they recommend using the proposed pushdown analysis methods to investigate the robustness of a damaged building system in terms of residual capacity. Also, IDP gives the most realistic estimate of residual capacity. In this study, the IDP method will be used to investigate the robustness of the structure.

C. Nonlinear Beam Behavior Following Column Loss

When the beam is subjected to large displacements, it initially responds elastically until it reaches the yielding point that's when the nonlinear plastic behavior starts. Generally, there are three main stages that happen, namely, compressive arching, transient tensile stage and the final catenary action stage as described by Stylianidis et al. [11], as shown in Fig. 1.

When the beam starts yielding, it causes some softening in the pushdown force. This is the compressive arching stage, where the compressive axial forces begin to develop within the beam until it reaches the maximum axial force. As the vertical displacement continues to increase, the axial force within the beam starts decreasing until it reaches to zero and then it turns to tensile, resulting in the transient tensile stage. Finally, given the end connections have sufficient ductility, the beam enters into the last stage which is the tensile catenary stage. During this growth of vertical beam deflections, important changes occur in the beam resisting mechanism. Initially the beam resisting mechanism is just flexure but as vertical deflections increase, axial compression develops because of the resistance to the outward movement. When the last stage of catenary action phase starts, the resisting mechanism in the beam changes from flexure to mainly tensile forces, in other words, the beam acts as a cable in tension. In [11], they propose a method based on simplified beam model to explain all of this behavior for steel beams. In summary, it is important to understand when each stage occurs and the conditions that are necessary for it to occur. This will help to reach an understanding of the mechanics of progressive collapse and the best ways to limit the likelihood of its occurrence. This beam behavior is not only limited to steel beams; the same stages are also observed in reinforced concrete beams. Livingston et al. [12] discussed such behavior; they also validated the numerical results using an experimental model. In order to study and characterize the full range of response, they performed a pushdown analysis. They evaluated the effects of the axial stiffness of the beams, steel yield stresses and the span length of the beams for two types of frames, which are ordinary and special. They concluded that while special frames are more ductile than ordinary frames, the results show that the ordinary beam demonstrates a higher resistance to progressive collapse following loss of a column.

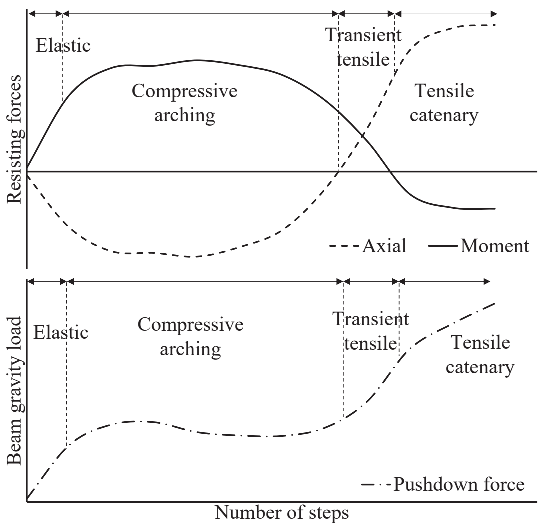

Fig. 1 The three stages of nonlinear beam behavior

D. Robustness Assessment Methods

Many of the mentioned papers deal with progressive collapse but only few attempts to quantify structural robustness. One of those is [10], which suggest the use of the OF as a measure of the robustness of a system. They use the OF to quantify the reserved capacity in the damaged bays.

Fascetti et al. [2] conducted some numerical simulations regarding sudden column removal. They suggest a method for robustness evaluation of buildings by sequentially removing the worst columns that produce the maximum displacement upon removal. Then, the next worst column is removed and that process continues until collapse in the structure is initiated.

II. Methodology

A. Sudden Column Removal

The IDP analysis of the full building subjected to column loss scenario will be conducted, under increasing gravity loads in the bays of interest. The OF computed from these methods is presented as a measure of the robustness of the structural system. In IDP, successive dynamic analyses with increasing gravity loads in the bays of interest are conducted until an OF corresponding to failure in the damaged bays is established. In this method, gravity loads are initially applied then the members designated as ‘lost’ are instantaneously deleted and the system is allowed to respond in an inelastic manner; therefore, it explicitly accounts for dynamic effects.

First, after applying the nominal gravity loads to the structure, the “lost” column is removed. At this point, the mass of the structure has not been applied yet. Before the transient analysis begins, the entire structural mass is lumped on top of the lost column only. Then, an eigenvalue analysis is performed to calculate the period that corresponds to vertical vibration. The period is used for defining a Rayleigh mass proportional damping with damping ratio of 10%. Next, the analysis continues with the beginning of the transient analysis and the floor above the lost column vibrates until all the vibrations are dampened and it settles on the final displacement.

During the analysis, the displacement is recorded at each stage. The maximum displacement value is taken and it is used to construct the envelope of the OF. Successive dynamic analyses with increasing gravity loads are conducted until an OF corresponding to failure in the damaged bays is established. Gravity loads are only increased above the lost column and not over the entire structure. Then, the process is repeated for all other columns since the different sections used for the columns and beams will produce different results. The resultant curves for each of those columns is plotted to represent the IDP curves.

Ideally, each column in the building should undergo sudden column removal to quantify the reserved capacity of all the columns in the structure. However, in some cases due to the symmetry of the building a smaller number of columns is considered.

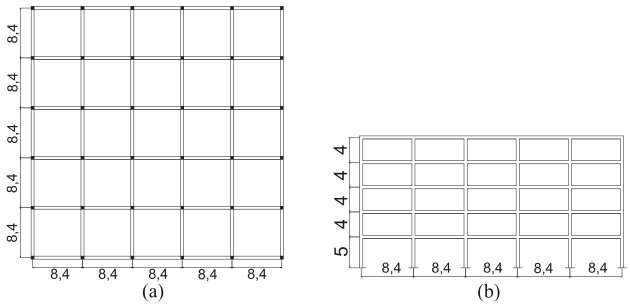

Fig. 2 (a) Floor plan and (b) elevation of the building considered in this study (dimensions in m)

B. Probabilistic Analysis Framework

Fragility curves are basically the probability that the demand (D) applied to the structure is greater than the capacity (C) of the structure. The probability is conditioned on a given Intensity Measure (IM), which in this study is taken as the OF calculated earlier. The generic representation of the conditional probability is:

$$Fragility \ = P(D > C \mid LF) \tag{1}$$

The fragility analysis is similar to the one used in seismic evaluation. One way to evaluate the fragility function given in (1) is by developing a probability distribution for D conditioned on the OF. This is also known as a probabilistic seismic demand model (PSDM), and convolving it with a distribution for the capacity. Cornell et al. [13] suggest that the estimate for the median demand ($\widehat{D}$) can be represented by a power model, however for this study, an exponential model is a better fit to the data in our case, given here

$$\hat{D} = ae^{b.IM} \tag{2}$$

where a and b are regression coefficients. A fragility curve is a function that defines the probability of a structure exceeding a given limit state for a certain OF. The fragility curve can be simplified into a cumulative distribution function of the OF capacity when the capacity and demand are expressed in terms of OF [13]. In this study, the OF α is chosen as the IM, $\alpha * [a1 * DL + a2 * LL]$ (DL and LL are the dead and live loads on the structure, respectively, and a1 and a2 are the design load combination factors). The fragility curves can be expressed as lognormal distributions:

$$P(D > C \mid LF) = 1 - \Phi\left(\frac{\ln(\hat{C}) - \ln(\hat{D})}{\beta}\right) \tag{3}$$

where Φ(.) is the standard Gaussian cumulative distribution function, $\widehat{C}$ is the capacity which is represented by the damage limit states as the displacement, β is the lognormal dispersion of the fragility curves.

The three limit states that are used are the immediate occupancy (IO), life safety (LS), and collapse prevention (CP). The IO level is acquired based on the serviceability limits for deflections. The CP is based on the last points before collapse which is acquired from the IDP curves. The LS level is estimated nearly half between IO and CP.

Three fragility curves will be developed based on the three limit states which represent the probability of exceeding the given limit state.

III. Case Study

A 3D frame model with 6 degrees of freedom is used. OpenSEES [14] is used for modeling the building and running the analysis. A 3D model rather than 2D is used to account for the member action behavior of the lateral beams since they provide extra strength for resisting collapse. This was demonstrated by [2], where they compared the results of sudden column removal in 2D and 3D and concluded that a 3D model is more accurate. Hence, a 3D model is used in this study.

The building is an RC structure composed of 5 stories and 5 x 5 symmetrical bays spanning 8.4 m in both directions. The first story is 5 m and the rest of the stories are 4 m each. Diagrams of the plan and elevation are shown in Fig. 2. The nodes at the ground are fixed for all degrees of freedom. The structure has a 200-mm thick slab of reinforced concrete. A superimposed design dead load of 1 kPa is acting on the slab to account for flooring finishes, ceiling services and partitions, in addition to the self-weight of the slab and all other structural members. The design live load is 3 kPa which is typical for office buildings. The same loads are applied on the roof for simplicity.

Two sections are used for columns: exterior and interior. The interior columns are 700 mm × 700 mm and the exterior columns are 500 mm × 500 mm. Similarly, the beams also have two sections for interior and exterior. All beams are 600 mm × 600 mm, however their detailing varies over the length of the beam. The use of OpenSEES allows fiber-section discretization and the usage of integration points along the length of the member to specify plastic hinges, which is important for accurate modelling of nonlinear and dynamic analysis.

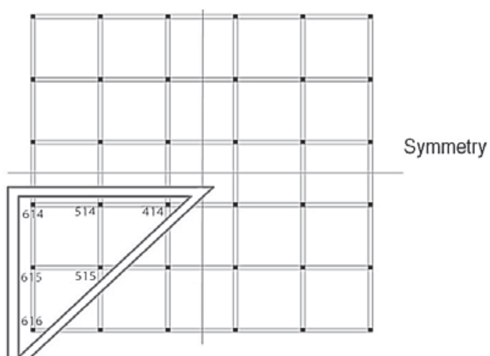

Fig. 3 Structure's symmetry and the considered columns

Due to the structural symmetry only a quarter of the columns will be considered. Further, out of that quarter, there are only six unique columns, which will undergo IDP, as shown in Fig. 3. Hence, the total number of columns is reduced from 36 to only 6 columns.

IV. Results and Discussion

Following the gravity loads application, the column is suddenly removed and a dynamic analysis is performed. The vibration of the floor above the lost column is shown in Fig. 4. These results are following the removal of the corner column (616) for nominal gravity loads which is an OF of 1. As mentioned earlier, the floor starts oscillating until all the vibrations are dampened and it settles on a final displacement. That process is repeated with increasing OF for each column to generate all IDP curves shown in Fig. 5. The three collapse resisting capacities for IO, LS, and CP are 33.6 mm, 75 mm, and 150 mm, respectively. They are used for computing the fragility curves and they are marked on the IDP curves as illustrated in Fig. 5. Based on the IDP curves, the median values and the dispersions of the fragility curves are computed.

The median is fitted according to (2) and the resulting plot is shown in Fig. 6 (a). Likewise, the fragility curves are plotted according to (3) for all three limit states and they are illustrated in Fig. 6 (b).

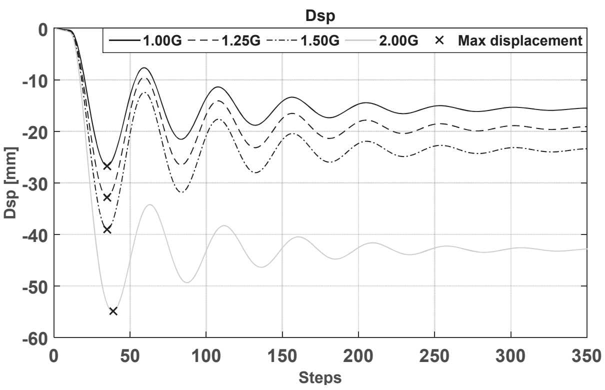

Fig. 4 Displacement history following the removal of the corner column (616)

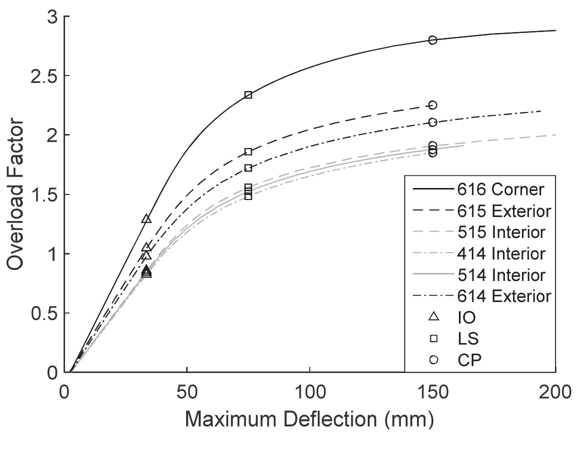

Fig. 5 IDP curves for all six columns with the limit states

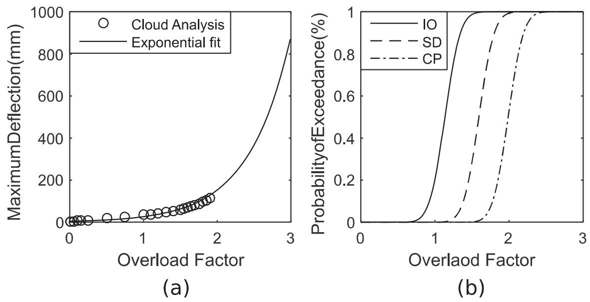

Fig. 6 (a) The median function and (b) the fragility curves for the three limit states

V. Conclusion

This paper presented the methods used in progressive collapse assessment of structural robustness. The sudden column removal of a RC structure was accurately simulated by using nonlinear IDP analysis. Fragility curves were developed to give the probability of exceeding three limit states corresponding to the level of damage due to the loss of one or more columns. This paper is important for quantifying the reserved capacity of the structure. It can help in developing robustness criteria for the design of buildings.

References

[1] S. Kokot and G. Solomos, "Progressive collapse risk analysis: Literature survey, relevant construction standards and guidelines". 2012, Luxembourg: Publications Office.

[2] A. Fascetti, S. K. Kunnath, and N. Nisticò, "Robustness evaluation of rc frame buildings to progressive collapse". *Engineering Structures*, 2015. 86: p. 242-249.

[3] GSA, "Alternate path analysis and design guidelines for progressive collapse resistance". 2013.

[4] J. Abruzzo, A. Matta, and G. Panariello, "Study of mitigation strategies for progressive collapse of a reinforced concrete commercial building". *Journal of Performance of Constructed Facilities*, 2006. 20(4): p. 384-390.

[5] S. Marjanishvili and E. Agnew, "Comparison of various procedures for progressive collapse analysis". *Journal of Performance of Constructed Facilities*, 2006. 20(4): p. 365-374.

[6] P. Graham Powell, "Progressive collapse: Case studies using nonlinear analysis". 2005.

[7] Y. Xiao, et al., "Collapse test of three-story half-scale reinforced concrete frame building". *ACI Structural Journal*, 2015. 112(4): p. 429.

[8] H. Lew, Y. Bao, S. Pujol, and M. A. Sozen, "Experimental study of reinforced concrete assemblies under column removal scenario". *ACI Structural Journal*, 2014. 111(4): p. 881.

[9] W.-J. Yi, Q.-F. He, Y. Xiao, and S. K. Kunnath, "Experimental study on progressive collapse-resistant behavior of reinforced concrete frame structures". *ACI Structural Journal*, 2008. 105(4): p. 433.

[10] K. Khandelwal and S. El-Tawil, "Pushdown resistance as a measure of robustness in progressive collapse analysis". *Engineering Structures*, 2011. 33(9): p. 2653-2661.

[11] P. Stylianidis, D. Nethercot, B. Izzuddin, and A. Elghazouli, "Study of the mechanics of progressive collapse with simplified beam models". *Engineering Structures*, 2016. 117: p. 287-304.

[12] E. Livingston, M. Sasani, M. Bazan, and S. Sagiroglu, "Progressive collapse resistance of rc beams". *Engineering Structures*, 2015. 95: p. 61-70.

[13] C. A. Cornell, F. Jalayer, R. O. Hamburger, and D. A. Foutch, "Probabilistic basis for 2000 sac federal emergency management agency steel moment frame guidelines. (abstract)". *Journal of Structural Engineering*, 2002. 128(4): p. 526.

[14] F. McKenna, G.L. Fenves, B. Jeremic, and M.H. Scott, “Open System for Earthquake Engineering Simulation”. 2007. http://opensees.berkely.edu.