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ORIGINAL ARTICLE

THE EFFECT OF SEISMIC DETAILS ON THE ROBUSTNESS OF A FRAME STRUCTURE DURING A PROGRESSIVE COLLAPSE

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ABSTRACT

Background: In the context of progressive collapse, robustness is broadly defined as a measure of the ability of a building system to carry most of its usual functions in the presence of local component failures. Specifically, structural robustness is a measure of the capacity of a building system to withstand loss of local load carrying capacity. **Objectives**: The objective of this study is to evaluating the robustness of building systems that have lost critical members By focusing on the residual capacity and associated collapse modes, particularly for seismically designed buildings. **Methods**: This paper presents a technique termed 'pushdown analysis' that can be used to investigate the robustness of building systems by computing residual capacity and establishing collapse modes of a damaged structure. The proposed method is inspired by the pushover method commonly used in earthquake engineering. Three variants of the technique, termed uniform pushdown, bay pushdown and incremental dynamic pushdown, are suggested and exercised using nonlinear analysis on 9-story steel moment frames designed for low and high levels of seismic risk. **Conclusions**: Simulation results show that the frame designed for high seismic risk is more robust than the corresponding one designed for moderate seismic risk. The improved performance is attributed to the influence of seismic detailing, specifically, the presence of reduced beam sections and stronger columns. It is shown that the dynamic impact factors associated with column removal are significantly lower than the commonly used value of 2.0 and are in line with lower values in the guidelines recently proposed by the US Department of Defense.
Keywords: *Progressive collapse, column loss, Beam-to–column joints, Catenary action, Dynamic increased factor*

1. INTRODUCTION

Robustness is a broad term used in a variety of contexts .This is usually used in engineering applications to measure "the resistance against breaking'' of an underlying quantity of interest. In structural control, for instance, a controller is said to be robust if it can perform satisfactorily (does not break down) for admissible perturbations in system properties and/or loading conditions. Similarly, a robust computer code is one that does not crash when unexpected computational errors are encountered, such as division by zero. In the context of progressive collapse, robustness is broadly defined as a measure of the ability of a building system to carry most of its usual functions in the presence of local component failures. Specifically, structural robustness is a measure of the capacity of a building system to withstand loss of local load carrying capacity.

There is a growing consensus in the structural engineering community that there is a need to quantify robustness for buildings that are susceptible to element loss, e.g. due to blast or impact. Such a measure could be used to provide a means for quantifying desired system performance, which could then be tied to the economy whereby important buildings could be characterized by some minimum level of robustness. Information about robustness is also necessary to decide if a structure is safe for continued occupancy after local distress or whether extensive repairs are needed before the structure can be deemed safe. Research efforts to quantify robustness, as applied to buildings vulnerable to collapse, are quite limited. In a pilot project for the study discussed herein, Khandelwal and El-Tawil (2008) proposed pushdown analysis to quantify the robustness of a structure with lost critical members [1]. Izzuddin et al., (2007) proposed three factors for measuring robustness including energy absorption capacity, ductility supply and redundancy [2]. They concluded that each of the three factors cannot be used as a standalone measure of robustness, but that system pseudo static capacity, which aggregates all the three factors, could be a suitable measure. Kim et al., (2009) investigated robustness by gradually pushing down at the location of a removed column. They showed that collapse capacity was a function of the number of stories, number of spans, and length of spans [3]. They compared their results to those from incremental nonlinear dynamic analyses and concluded that pushing down at the damaged column location could overestimate the progressive collapse capacity of a structure. In other recent studies, the authors have investigated the progressive collapse behavior of seismically designed steel building frames [4,5]. In those studies, progressive collapse of steel frames was investigated using the Alternate Path Method (APM) within a nonlinear dynamic analysis framework. The APM, which is a threat independent methodology advocated by the GSA [6] and UFC [7], is generally applied in the context of a 'missing column' scenario to assess the potential for progressive collapse. The authors concluded that though APM can be used to investigate progressive collapse behavior, it could not be used to measure "robustness" of the structural system in cases where the structure under consideration is deemed to be able to survive loss of critical members. In particular, it is unable to explicitly determine if a structure is near an incipient collapse state. In this paper, pushdown analysis is presented as a means for evaluating the robustness of building systems that have lost critical members. The study presented herein differs from previous studies that addressed pushdown response in: (1) its focus on the residual capacity and associated collapse modes, particularly for seismically designed buildings, and (2) the means by which the method is applied. The proposed method is inspired by the pushover method (both static and incremental dynamic) commonly used for assessing the seismic resistance of building structures (see, for example, FEMA-350 [8]).

2. MATERIALS AND METHODS

2.1 Study site

The proposed pushdown analysis method consists of analyzing the structure, which has suffered loss of one or more critical members, under increasing gravity loads. The gravity loads are incremented until collapse of the structure occurs, defined as an inability to support the applied loading. Usually this state is reached after substantial changes in the geometric configuration have occurred accompanied by member separation from the main structural system. The load corresponding to this condition is defined as the failure load.

Pushdown analysis of a damaged structure is accomplished in three different ways: Uniform Pushdown (UP); Bay Pushdown (BP); and Incremental Dynamic Pushdown (IDP). The overload factors computed from these methods, together with the corresponding collapse modes, are proposed as measures of the robustness of the structural system in question. In cases where APM shows that the structure is not capable of successfully absorbing the loss of local resistance, the structure is deemed to have no robustness. In such a situation, the proposed pushdown methods are not applicable.

In the UP case, gravity loads on the entire damaged structure are increased proportionally within a nonlinear static analysis framework until the system collapses. A UP analysis will lead to a collapse state corresponding to failure of the weakest part of the damaged structure and failure may occur outside the damaged bays. For example, a gravity bay may dominate the collapse response by failing prematurely. This method may, therefore, not adequately consider the damaged bays nor capture the propensity for collapse to propagate from damaged bays to adjacent ones. The BP method is proposed to focus attention only on the damaged bays. In this method, the gravity load is increased proportionally only in the bays that suffered damage until the system collapses. The remaining part 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 overload factor calculated as the ratio between the load leading to failure within the damaged bays and the nominal gravity load. The IDP method is inspired by the incremental dynamic analysis method used in earthquake engineering [9]. In IDP, successive dynamic analyses with increasing gravity loads in the bays of interest are conducted until an overload factor corresponding to failure in the damaged bays is established. In each dynamic analysis case, the system is first assumed to be undamaged while the loading is being applied. As soon as the dynamic effects associated with the applied loading die away, members designated as 'lost' are instantaneously deleted and the system is allowed to respond in an inelastic manner. Unlike the UP and BP methods, this analysis method explicitly accounts for dynamic effects. The disadvantage is that it is costly in terms of required computational effort because multiple nonlinear, dynamic analyses must be conducted.

2.2 Equation: The capacity of the structure at the failure load is expressed in terms of the overload factor (Eq. (1)), defined as the ratio of failure load to the nominal gravity loads.

$$\text{Overload Factor (OF)} = \frac{\text{Failure load}}{\text{Nominal gravity loads}}. \qquad \textbf{(1)}$$

The National Institute of Standards and Technology (NIST) designed prototype steel framed buildings for the purpose of studying their response to an event which may cause progressive collapse [10]. The buildings are 9-story office buildings with plan dimensions of 24×32 m and utilize moment-resisting frames as the lateral load-resisting system. The buildings are designed for: (1) Low Seismic zone (a_g=0.25g), which results in low ductility class Frames (LDCF) as defined is the AISC Seismic Provisions [11], and (2) high Seismic zone, which results in high ductility class Frames (HDCF). The two seismic design categories address low and high seismic risk and are considered to study the effect of seismic design and detailing on robustness of the steel building systems. The design loads on the buildings are determined based on the International Building Code [12]. The material design standards used in the design of members and their connections are those referenced in the AISC Load and Resistance Factor Design Specifications for Structural Steel Buildings [13] and the AISC Seismic Provisions for Structural Steel Buildings [11]. For typical floors, the dead load consists of the self-weight of the slab of 2.2 kN/m2 and a super-imposed dead load of 1.44 kN/m2; while the design live load is assumed to be 4.79 kN/m2. For the roof, the self-weight of the slab is 2.2 kN/m2, the super-imposed dead load is 0.48 kN/m2; and the design live load is 0.96 kN/m2. The reduction in live loads is based on IBC 1607.9.1 [12].

3. RESULTS

Plan views of the buildings are shown in Fig. 1, while the elevation of the East–West frames considered in this paper are shown in Fig. 2.

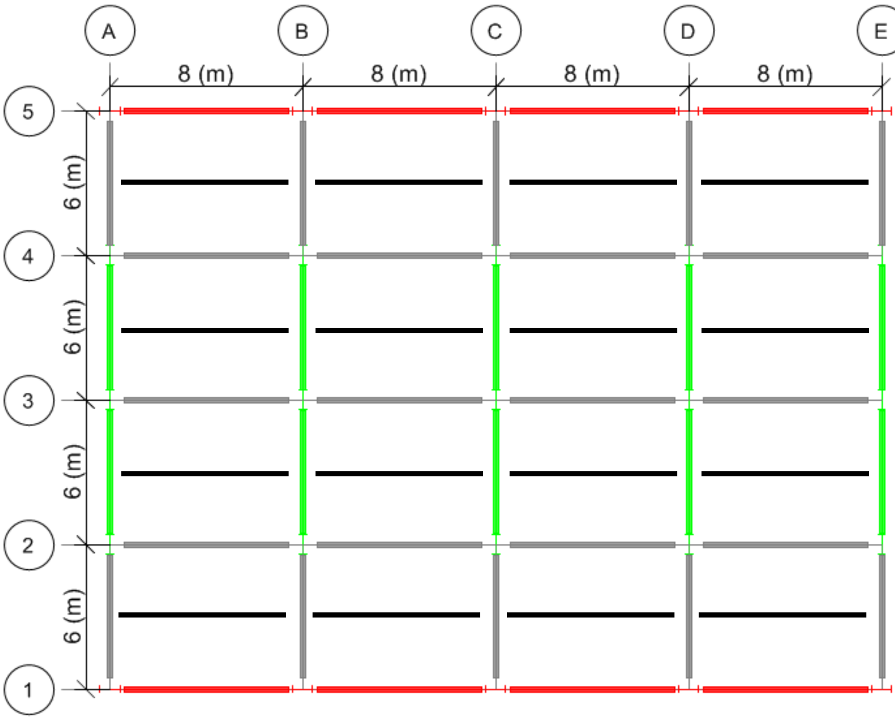

Figure 1: The figure presents Plan layout for LDCF and HDCF building systems.

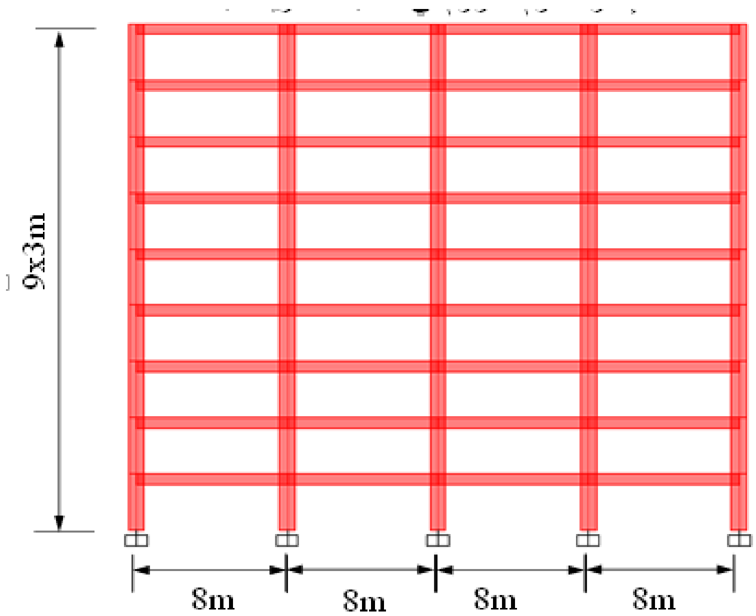

Figure 2: The figure presents Elevation of frames under consideration.

Pushdown analyses of the above-described LDCF and HDCF are carried out using the proposed analysis methods. Key results of interest, including overload factors and collapse modes for the three proposed methods, were obtained for analysis cases where APM showed that the system under consideration is able to survive member loss as outlined in [4].

Table 1 gives a summary of these cases. The corresponding pushdown analysis results for the two frames are shown in Tables 2 and 3 and the results are discussed in detail below. Both tables summarize the overload factor (computed from Eq. (1)), collapse mode (CCM or PCM), and the mode by which failure initiates. To facilitate the following discussion, the columns and beams are designated using the notation in Fig. 2. For example, column C-1 represents a first story column in column line C (Fig. 2). Similarly, beam CD-2 represents a second story beam in bay CD (Fig. 2). Analysis cases as designated by the type of analysis and an appended number that refers to the APM case in Table 1. For example, UP-1 implies a Uniform Pushdown analysis for APM Case 1 described in Table 1, while IDP-5 is an Incremental Dynamic Pushdown for APM Case 5.

Table 1: Pushdown analysis results—LDCF building.

pushdown analysis type	loading type	member removed	Overload factor	failure initiation
UP-1	Uniform	column C-1	5.65	Buckling of column B1
UP-2	Uniform	column B-1	5.9	Buckling of column C1
UP-3	Uniform	column A-1	5.3	Buckling of column B1
UP-4	Uniform	column A-1 & B-1	3.13	Buckling of column C1
UP-5	Uniform	column B-1 & C-1	3.8	Failure of connection D
BP-1	Bay BC & CD	column C-1	6	Failure of connection D
BP-2	Bay AB &BC	column B-1	6.1	Failure of connection C
BP-3	Bay AB	column A-1	5.9	Failure of connection B
BP-4	Bay AB &BC	column A-1&B1	3.1	Failure of connection C
BP-5	Bay AB &BC&CD	column B-1&C1	3.8	Failure of connection D
IDP-1	Bay BC & CD	column C-1	4.7	Failure of connection D
IDP-2	Bay AB &BC	column B-1	4.8	Failure of connection C
IDP-3	Bay AB	column A-1	4.8	Failure of connection B
IDP-4	Bay AB &BC	column A-1&B1	2.7	Failure of connection C
IDP-5	**Bay AB &BC&CD**	**column B-1&C1**	**3**	**Failure of connection D**

Table 2: Pushdown analysis results—HDCF building.

pushdown analysis type	loading type	member removed	Overload factor	failure initiation
UP-1	Uniform	column C-1	7.3	BEAM YIELD
UP-2	Uniform	column B-1	6.9	BEAM YIELD
UP-3	Uniform	column A-1	6.6	BEAM YIELD
UP-4	Uniform	column A-1 & B-1	3.73	BEAM YIELD
UP-5	Uniform	column B-1 & C-1	4.23	CONNECTION FAILURE
BP-1	Bay BC & CD	column C-1	7.1	CONNECTION FAILURE
BP-2	Bay AB &BC	column B-1	7.2	CONNECTION FAILURE
BP-3	Bay AB	column A-1	7.5	BEAM YIELD
BP-4	Bay AB &BC	column A-1&B1	4.5	BEAM YIELD
BP-5	Bay AB &BC&CD	column B-1&C1	4.9	CONNECTION FAILURE
IDP-1	Bay BC & CD	column C-1	5.6	CONNECTION FAILURE
IDP-2	Bay AB &BC	column B-1	5.4	CONNECTION FAILURE
IDP-3	Bay AB	column A-1	5.9	BEAM YIELD
IDP-4	Bay AB &BC	column A-1&B1	3	BEAM YIELD
IDP-5	Bay AB &BC&CD	column B-1&C1	3.2	CONNECTION FAILURE

Several observations can be made from the analysis results in Table 1. First, it is clear that by any of the measures employed, the LDCF has significant robustness, i.e. resistance to collapse. The lowest factor in the table is 2.7 and belongs to IDP-1. Second, UP cases have equal or lower overload factor when compared to the corresponding BP cases. This is expected because the structure is under higher overall loads in the former compared to the latter, and will overload weaker parts of the structure causing premature failure. Third, loading cases involving only moment bays (e.g. BP-2) have higher overload factors compared to cases involving gravity bays (e.g. BP-1), primarily because they are comprised of stronger members.

Table 1 show that the collapse initiation mode is similar for UP, BP and IDP cases. Collapse is typically initiated by out of plane buckling of the ground story columns. The corresponding failure modes for these analysis cases are shown in Fig. 3. After an overloaded column buckles, the loads are transferred to adjacent bays leading to additional column buckling, i.e. failure propagates to adjacent bays. The corresponding collapse modes are therefore designated as propagating collapse modes (PCMs) since failure extends to adjacent bays compromising the rest of the system. When failure modes are similar, which they are for the cases considered herein, the overload factor for IDP cases is lower than that in the corresponding BP cases because of dynamic effects. The following discussion focuses only on analysis results for BP cases since the collapse modes are similar for all three load types. Analysis results for the HDCF building are reported in Table 2, and some of the corresponding collapse modes are shown in Fig 4. The general observations made for the LDCF are also seen here, i.e. the HDCF has substantial overload capacity, UP cases have equal or lower overload factors than corresponding BP cases, and loading involving moment bays only leads to higher overload factors compared to cases involving gravity bays.

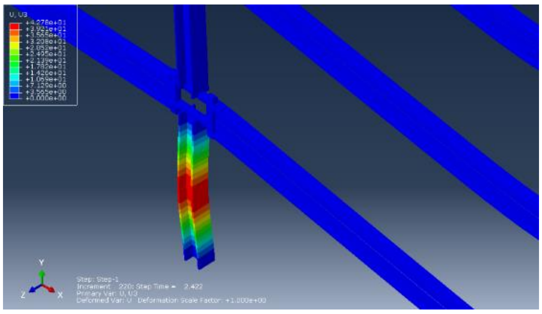

Figure 3: The figure presents failure mode for LDCF (UP-1).

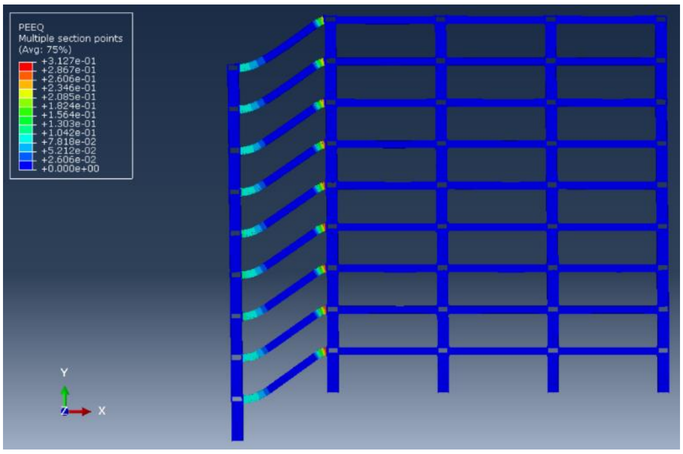

Figure 4: The figure presents failure mode for HDCF (UP-3).

4. DISCUSSION

The simulation results offer a means for computing the dynamic increase factor (DIF), which embodies the effect of dynamic loading on system response. The DIF can be computed as the ratio of overload factors for BP (quasi-static loading) and IDP (dynamic loading) cases. The DIF for the LDCF building ranges from 1.14 to 1.27, whereas it ranges from 1.06 to 1.45 for the HDCF building. Lower DIFs are observed when one of the bays involved in the computation is a gravity bay. This suggests that dynamic effects vary depending on the type of structural system, and that in both systems, dynamic effects are not as high as 2.0, which is specified in GSA [6]. This observation is in accord with the growing consensus that using DIF=2.0 is too conservative. The most recent version of the UFC [7] has specified lower values that are tied to the type of structure being analyzed. Following the UFC [7] guidelines, the DIF for the moment bays of the LDCF is 1.40 and the corresponding number for the HDCF is 1.22, numbers that are more in line with the values computed herein. Another key observation is that while significant differences in collapse modes occurred between UP and BP cases, especially for HDCF building, there is good correlation between IDP and BP cases, suggesting that BP analysis is a reasonable way to measure pushdown resistance for the types of frames discussed herein.

5. CONCLUSION

This paper proposed new analysis techniques that could be used for investigating the robustness of building systems. Three pushdown methods were proposed—uniform pushdown (UP), bay pushdown (BP) and incremental dynamic pushdown (IDP).The proposed methods were then exercised to investigate the robustness of two dimensional, 9-story seismically designed frames, one of which was an intermediate moment frame and the other a special moment frame. Based on the limited simulation studies conducted, and within the assumptions and limitations described in the paper, the following conclusions can be drawn.

1. The proposed pushdown analysis methods can be used to investigate the robustness of a damaged building system in terms of residual capacity and associated collapse modes.

2. Incremental dynamic pushdown gives the most realistic estimate of residual capacity and collapse modes. However, collapse modes associated with bay pushdown analysis cases agree well with IDP for the building systems considered in this study suggesting that static BP analysis is a simpler, more economical, but still reasonable substitute for IDP.

3. While less conservative and definitely more reasonable than GSA [6], the UFC guidelines produce DIFs that are more in line with the values computed in this study.

4. The development of tensile catenary action in some components of the damaged system necessitates development of compressive forces in other parts of the system. These force patterns develop as a result of frame action within the structural system.

5. The simulation results suggest that the HDCF building designed for high seismic risk is generally more resistant to progressive collapse and hence more robust than the LDCF building designed for moderate seismic risk. This is evident from the overload factors, which for the HDCF building range from 1.7 to 3.6, while the LDCF building has overload factors in the range of 1.4–2.9. The better performance of the HDCF building as compared to the LDCF one is attributed to better layout and the use of seismic detailing, specifically the use of RBS connections and stronger columns.

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