Steel and Composite Structures, *Vol. 25, No. 5* (*2017*) *617-624*
DOI: https://doi.org/10.12989/scs.2017.25.5.617

Dynamic Increase factor based on residual strength to assess progressive collapse

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(*Received May 06, 2017, Revised August 13, 2017, Accepted September 05, 2017*)

Abstract. In this study, a new empirical method is presented to obtain Dynamic Increase Factor (DIF) in nonlinear static analysis of structures against sudden removal of a gravity load-bearing element. In this method, DIF is defined as a function of minimum ratio of difference between maximum moment capacity (M_u) and moment demand (M_d) to plastic moment capacity (M_p) under unamplified gravity loads of elements. This function determines the residual strength of a damaged building before amplified gravity loads. For each column removal location, a nonlinear dynamic analysis and a step-by-step nonlinear static analysis are carried out and the modified empirical DIF formulas are derived, which correspond to the ratio min $\left[(M_u - M_d)/M_p\right]$ of beams in the bays immediately adjacent to the removed column, and at all floors above it. Therefore, the new DIF can be used with nonlinear static analysis instead of nonlinear dynamic analysis to assess the progressive collapse potential of a moment frame structure. The proposed DIF formulas can estimate the real residual strength of a structure based on critical member.

Keywords: progressive collapse; nonlinear static analysis; dynamic increase factor; alternate load path; residual strength

1. Introduction

Progressive collapse of structures starts from a local failure leading to a chain failure that collapses a whole structure or an unreasonably huge part of it. In this occurrence, the structural collapse following initial failure is an independent event. Hence, a structure must be capable of withstanding extra loads after column removal and transferring them to adjacent elements in a different path. Because of increased catastrophes in recent years, numerous studies have evaluated prevention of progressive collapse in which critical gravity load-bearing elements were eliminated, thereby design of structures was carried out with the aim of mitigating associated risks (Ruth *et al*. 2006, Izzuddin *et al*. 2008). Furthermore, Málaga-Chuquitaype *et al*. (2016) studied the influence of secondary frames on mitigating the possibility of collapse. In the present study, strength, stiffness, and ductility were evaluated as crucial parameters in steel structure design.

Many approaches have been proposed in evaluating progressive collapse potential in buildings. Numerous inquiries have evaluated efficiency and effectiveness of existing progressive collapse design guidelines. Furthermore, numerous empirical DIF formulas have been offered that can be employed in nonlinear static analyses of progressive collapse potential in structures against column removal. Stevens *et al*. (2008) executed some nonlinear analyses on steel frame models and ultimately proposed an empirical DIF formula for steel building frames. McKay *et al*. (2012) followed a similar process on a wide range of steel and concrete frame models and presented relevant empirical DIF formulas. Liu (2013) offered a novel DIF for nonlinear static alternate path analysis. This DIF is contingent on column removal locations, ratio of moment demand under original unamplified static gravity load, and factored plastic moment capacity. Mashhadi and Saffari (2017) offered some new DIF formulas by considering ductility demand, gravity loads, and post-elastic stiffness ratio of members for assessing progressive collapse potential of structures. Damping ratio as a possibly effective parameter on DIF was evaluated by Mashhadi and Saffari (2016). Consequently, a series of nonlinear analysis for steel frame structures with various damping ratios was conducted. Finally, an empirical DIF was presented.

Cassiano *et al*. (2017) conducted a parametric finite element analysis in which the investigated effective parameters (bolt diameter, end-plate thickness, number of bolt rows, type of beam profile, and orientation of column axis) on flush end-plate beam to column connections. This connection is usually considered a pinned type connection while exhibiting behavior similar to semi rigid connections.

The energy balance method was applied to evaluate dynamic behavior and subsequently, DIF in models. In addition, the effect of end plate thickness and beam cross section was examined on DIF. Liu *et al*. (2013) studied the influence of cleat beam-column connection on structures encountered by sudden column removal scenario. For this purpose, experiments and numerical analyses were

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http://www.techno-press.org/?journal=scs&subpage=6 ISSN: 1229-9367 (Print), 1598-6233 (Online)

conducted and both force and displacement-based DIFs were attained and matched to the standards suggested by design guidelines. Results revealed that force-based DIF slowly increases from 1.1 to 2.8, increasing displacement demand while displacement-based DIF increases from 1.1 to 1.4, increasing column support force up to about 10; the DIF subsequently increases to 1.45 up until final failure.

In order to evaluate the progressive collapse potential of buildings, the state of energy balance between external work, internal strain, and dissipated energy are reflected upon (Xu and Ellingwood 2011, Szyniszewski and Krauthammer 2012). Some techniques including pushdown analysis and pull-down analysis, which exhibit the same impression are offered for predicting the peak structural reactions of structural frames upon sudden removal of a column (Liu 2015, Liu (Max) and Pirmoz 2016). In order to assess the relationship between loss-of-stability phenomena and progressive collapse, Gerasimidis *et al*. (2015) implemented a number of nonlinear static analyses on2D and 3D models of a 20-story steel frame.

Numerous studies have investigated progressive collapse resistance of buildings (Chen *et al*. 2016, Cassiano *et al*. 2016, Larijani *et al*. 2017). Mirtaheri and Abbasi Zoghi (2016) explain some retrofitting and design methods to enhance resistance to progressive collapse. Tian et al. (2017) assessed the progressive collapse resistance capacity of long-span of spatial grid structures. They proposed a novel dynamic analysis method to simulate progressive collapse of long-span spatial grid structures. Mohajeri *et al*. (2016) proposed a simplified theoretical method based on catenary capacities of the beams to predict general behavior of RC frames under the column removal scenario.

Applicable standards and design guidelines (e.g., General Services Administration (GSA 2003) and United States Department of Defense (UFC 4-023-03 2013)) are accessible for design of progressive collapse resistant structures. These standards address measureable significant safety practices that enhance resistance toward progressive collapse. One of the methods proposed in both of the mentioned guidelines is the Alternative Path Method (APM). APM is a broadly established method that is used to evaluate the potential progressive collapse of building structures via direct removal of a column (UFC 4-023-03 2013).

It is worth noting that in the DIF formula of the UFC guideline and nonlinear static analysis, the only effective parameter is the maximum ratio associated with plastic rotation and yield rotation angles for the acceptance criteria $\left(\theta_{pra}/\theta_y\right)$. θ_{pra} and θ_y only depend on material and mechanical properties of the affected structural members. It is clear that the effect of gravity load and capacity of structure which have influence on structural response are not considered to calculate DIF in accordance with UFC guidelines. It is foreseeable that DIF value is different for structures with different gravity loads and different ultimate strengths. Thus, adjustment of DIF in a way that embraces residual strength of structures based on gravity loads in order to match the NLS procedure to the NLD procedure in the acceptable way.

In this research, an empirical formula is proposed to determine DIF, which considers ductility demand, gravity loads in affected bays, and ultimate strength of member in evaluating the progressive collapse potential of a structure. For this purpose, a series of 3-D moment frames with three and ten-storey buildings (as low and mid-rise buildings) with different span lengths are provided. The mentioned structures are designed to withstand diverse seismic ground motion intensities to cover a variety of buildings with different section members. Hence, a step-by-step nonlinear static analysis is conducted on every column removal location and the function min $\left[\left(M_u - M_d\right)/M_p\right]$ is considered as residual strength of the critical member in developing the proposed DIF formula. Consequently, an empirical DIF is offered that can be applied for nonlinear static analysis of structures. The offered DIF formula is capable of predicting residual strength of critical members in structures.

2. Analysis procedure

In the alternate path method, according to General Services Administration (GSA 2003) and Department of Defense (UFC 4-023-03 2013), one of the three Linear Static (LS), Nonlinear Static (NLS), and Nonlinear Dynamic(NLD)procedures can be applied to determine the ability of a structure to bridge over a detached structural element.

The simplest solution here is the Linear Static (LS) option that is incapable of offering an exact prediction of nonlinear dynamic structural behavior. In this procedure, the projected load cases differ based on deformation-controlled or force-controlled actions.

In the Nonlinear Static (NLS) procedure, material and geometrical nonlinearities are reflected in the model with removed vertical load-bearing element. To include both of dynamic effect due to sudden column loss and nonlinearity, the loads are amplified in the bays directly adjacent to the removed element and on all floors above it (UFC 4-023-03 2013). Results of ductile member deformations and brittle member strengths are compared with anticipated deformation capacities and maximum internal member forces, respectively. Expected deformation capacities are shown in guidelines and standards.

Nonlinear Dynamic (NLD) analysis is considered the most precise and costly procedure sensitive to certain parameters including time step, gravity loads, damping ratio, plastic hinge definition and post-elastic stiffness ratio.

3. Introducing a new method for determining Dynamic Increase Factors (DIF)

3.1 Description of the proposed method

As mentioned above, many methods have been presented to calculate DIF. However, none of them exhibit adequate precision and efficiency in all probable conditions. The only effective parameter in the DIF formulation of the UFC 4-023-03 (2013) guideline and nonlinear static analysis is θ_{pra}/θ_y. This method presents a DIF that depends only on ductility whereas other effective parameters such as gravity loads in affected bays and ultimate strength of members are not considered.

Liu (2013) proposed novel DIF formulas for nonlinear static alternate path analysis as a function of $\lambda = \max(M_u/M_p)$. In these formulas, gravity loads in affected bays and ductility are considered. Nonetheless, the proposed formula is inaccurate for structures within a high level of nonlinearity.

In the current study, a new modified DIF is proposed that includes ductility demand, gravity loads in affected bays and ultimate strength of a member. In this method, the critical member and DIF are determined based on residual strength of the member. Figure1 shows member characteristics of a concentrated plastic hinge. The ultimately accepted moment (M_u) can be calculated as follows

$$M_u = M_p + \Delta M \tag{1}$$

where $M_p = K \times \theta_y$ and $\Delta M = \eta K \times \theta_u$ are the plastic moment and additional moment capacities of the member after yield, respectively. In Fig. 1, K is linear stiffness, ηK is post-elastic stiffness, θ_y is yield rotation, θ_u is post yield rotation, and $\mu = \theta_u/\theta_y$ is member ductility. Substituting the above parameters into Eq. (1), it can be written as

$$M_u = M_p \times (1 + \eta\mu) \tag{2}$$

Using Eq. (2), the ratio of residual moment to plastic moment $(\gamma = (M_u - M_d)/M_p)$ can be calculated as follows

$$\gamma = (1 + \eta\mu) - \lambda \tag{3}$$

Where

$$\lambda = M_d/M_p \tag{4}$$

M_d is moment demand of the member; therefore, it can be found that, Eq. (4) is related to the ratio of moment demand and plastic moment (λ).

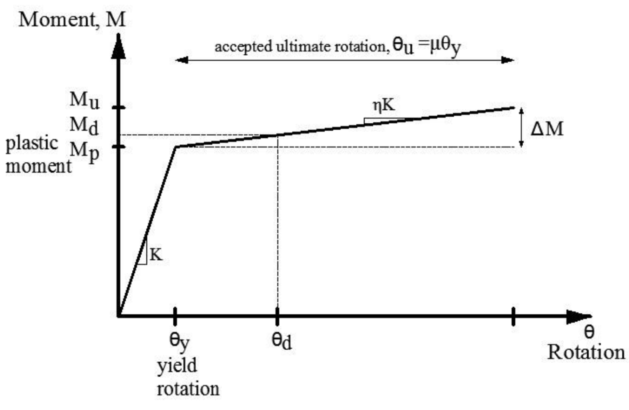

Fig. 1 Moment-Rotation curve and characteristic of member

The minimum value of γ determines the critical member in damaged structure; therefore, in the current study, the parameter γ is considered to determine DIF. To calculate the ultimate moment of members, an identical value of maximum ductility $(\mu = 9)$ and post-elastic stiffness ratio $(\eta = 3\%)$ are considered (ASCE 41 2013). Here tensile catenary action is not taken into account in calculating DIFs.

3.2 Determining Dynamic Increase Factors (DIF)

As mentioned earlier, NLD analysis is a time consuming and sophisticated analysis. Therefore, a study was conducted to investigate needed essential factors to match NLS procedures to the NLD procedure in an appropriate way. In this study, residual strength of the members is considered to assess the progressive collapse potential of structures. Structural deformation is taken into account according to ASCE 41 (2013) as the best metric for approximating structural damage. A gravity load combination of 1.2D+0.5L is used where D and L are dead and live loads, respectively (ASCE/SEI 7-10 2010). For each column removal scenario, three steps are conducted to specify DIFs as follows:

(1) Performing a nonlinear dynamic analysis including ASCE extreme load case without any enhancement to obtain the maximum ratio of θ_d/θ_y among all members of the bays affected by the column removal location where θ_d and θ_y are demand and yield rotations, respectively.

(2) Applying unamplified gravity loads statistically to the structure with a removed column to find min $[(\beta M_u - M_d)/M_p]$ in NLS analysis with the same model as Step 1. M_u is maximum moment capacity; M_p is plastic moment capacity, and M_d is moment demand of members within the affected bays adjacent to the column removal location. Plastic moment capacity is calculated as $M_p = R_y Z F_y$ where R_y is over strength factor and Z and F_y are cross-section plastic modulus and steel yield stress, respectively. M_u is calculated by considering the strain-hardening slope of the moment-rotation diagram. The modified factor $\beta = [1.27/(1 + 0.03\mu)]$ is defined to normalize the residual strength of members.

(3) Performing an NLS analysis with the same design and model as Step 1within a trial DIF is applied to the ASCE extreme event load case and the minimum ratio of θ_d/θ_y is recorded and compared to the ratio measured in Step 1. Subsequently, DIF is modified and the model is re-run until the minimum ratio of θ_d/θ_y matches the corresponding ratio in the dynamic analysis. Fig. 2 illustrates the steps for calculating DIF.

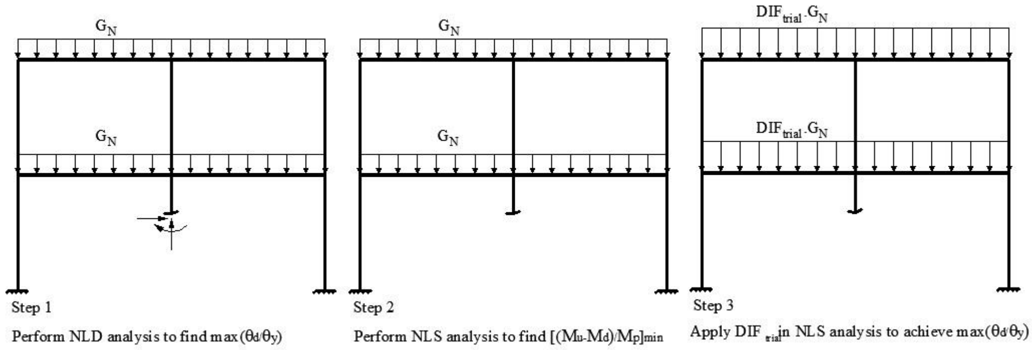

Fig. 2 Illustration of the steps to obtain the data point of DIF

4. Modeling

In order to assess progressive collapse potential of structures against column removal, a sequence of three and ten-storey buildings with diverse bay lengths were investigated and designed based on an array of different seismic ground motion intensities (Mashhadi and Saffari 2016). The associated floor plans with different bays comprising 3, 4.5, 6, and 9 meters are demonstrated in Fig.3. Each 3-D building includes five and six frames in the x and y directions, respectively. However, in all buildings, only two frames were simple frames whereas others were moment resistant frames. Table 1 summarizes the related gravity loads. Member properties of buildings with different span lengths are in accordance with the recent study (Mashhadi and Saffari 2016).

Table 1 Structural loading

Load	Unit (KN/m^2)	Load Type
DL	4.45	Dead load
CL	3.35	Cladding load in the perimeter
LL	3.36	Storey live load including partitions
LLR	0.96	Roof live load

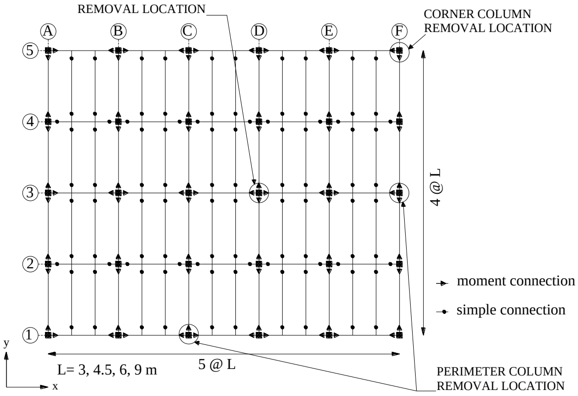

Fig. 3 Typical floor and column removal locations

In accordance with UFC 4-023-03 (2013), the gravity load combination 1.2D+ (0.5L or 0.2S) should be employed. Yield and ultimate strength values of steel members in the 3D models were considered 235.36 and 362.85 MPa, respectively. Modulus of elasticity and over strength factor utilized in the analysis were considered 200,000 MPa and 1.05, respectively. Both geometric and material nonlinearities are considered for the beams and columns via modeling of concentrated plastic hinges.

In order to investigate the load redistribution behavior in the structure after column loss, SAP2000 software (2011) was utilized for processing numerous APM analyses on a number of steel moment frame structures. Concentrated hinge properties were determined in accordance with ASCE 41 (2013). Multiple rotation yields were used based on ASCE 41 (2013) to define acceptance criteria and modeling parameters. A schematic of the employed moment-rotation diagram for beams and columns are depicted in Fig.1where θ_u is the accepted ultimate rotation of the beam or column and θ_y is the rotation at yield that is calculated from ASCE 41 (2013). In this study, the effect of simple connections is overlooked. A strain hardening slope equal to 3% of the elastic slope is considered for post-yield of steel members (ASCE 41 2013). The damping ratios for these analyses were set to 1%. Column removal time and time step are taken as 1/20 and1/200 of the vertical natural period, respectively (McKay *et al*. 2012).

4.1 Location of column removal

According to UFC 4-023-03 (2013), internal and external columns and walls should be omitted from the building plan; thus, in this study, analyses are carried out after columns at the interior, corner, and perimeter of the plan are removed. Due to symmetry in the plan, only four columns in different stories of each structure are chosen and removed, which is shown in Fig. 3. Under each column removal scenario, nonlinear dynamic and static analyses are carried out according to the procedures described in Section 3.2.

Table 2 Analysis results of building with span lengths of 4.5 meters

Case	Gravity load	θ_d	Beam $\left[\theta_y\right]$	μ	(Kgf.m) M_d	(Kgf.m) M_p	γ	DIF_{ufc}	$DIF_{analysis}$
1	1.2D+0.5L	0.043	IPE270 [0.0077]	5.60	12768	12196.8	0.22	1.16	1.21
2	1.1 (1.2D+0.5L)	$>\theta_u$	IPE270 [0.0077]	-	13041	12196.8	0.2	1.16	-
3	0.8 (1.2D+0.5L)	0.019	IPE270 [0.0077]	2.42	12292	12196.8	0.27	1.16	1.41

5. Results and discussion

Analysis results are provided to show the parameters that affect DIF. These results demonstrate that DIF depends on ultimate, plastic and yield rotations, gravity loads and damping ratio of the structure. For these analyses, the damping ratios were set to 1%. UFC 4-023-03 (2013) suggested Eq. (5) to calculate DIF for steel structures.

$$DIF = 1.08 + \frac{0.76}{0.83 + \left(\theta_p / \theta_y\right)} \tag{5}$$

According to the presented formula in UFC 4-023-03 (2013), DIF is calculated based on only the ratio of plastic and yield rotation. This means that DIF depends only on the ratio of maximum accepted rotation to yield rotation of beams, and thus, does not consider gravity loads in the bays around the removed column. To assess the effect of gravity loads on DIF, the mentioned systematic analysis conducted for a specific structure and the obtained results are summarized in Table 2. In this table, DIFs from UFC 4-023-03 (2013) and those directly concluded from analyses with different gravity loads for a specific column removal location are presented.

As shown in Table 2, DIF is obtained equal to 1.16 based on UFC guidelines for all three cases whereas the obtained DIFs from direct analyses are different for each assumed gravity load. In Case 1, gravity load is indicated by the load combination 1.2D+0.5L. In this case, the calculated DIF from analysis is 1.21 while by increasing gravity loads in Case 2, maximum plastic rotation in the structure exceeded acceptable plastic rotation and the structure does not possess adequate capacity to bridge over the removed column. In Case 3, reduction of gravity loads increases DIF to 1.41. It is understandable that by decreasing the loads, ductility demand is decreased and so DIF increases. The rate of changes of the DIF is related to ductility demand and the level of nonlinearity of the structure. It is found that due to increase in gravity loads, ductility demand is increased and residual strength of structures is decreased.

5.1 Analysis procedure

Fig. 4 shows a column removal scenario for a specific three-dimensional building the results of which are summarized in Tables 3 and 4 as an example to illustrate the mentioned procedure. In this example, a perimeter column of a three-story building with 4.5 meter bay lengths is removed from the structure. The obtained demand rotations

Table 3 First and third step analysis results of three storey building with 4.5 meter span length

Hinge number	**Beam**	θ_d		θ_y	θ_{dNLD}/θ_y
	IPE	NLD	NLS+DIF		
1	270	0.0431	0.0432	0.0077	**5.60**
2	270	0.0421	0.0429	0.0077	**5.47**
3	270	0.0427	0.0431	0.0077	**5.54**
4	270	0.0421	0.0428	0.0077	**5.47**
5	200	0.0420	0.0428	0.0105	**4.95**
6	200	0.0425	0.0429	0.0105	**4.05**
7	240	0.0341	0.0352	0.0087	**3.92**
8	220	0.0378	0.0388	0.0095	**3.98**
9	200	0.0368	0.0380	0.0105	**3.50**

of beams from nonlinear dynamic analyses and nonlinear static analyses via application of DIFs around a removed column are presented in Table 3.

Table 3 shows the maximum rotations of plastic hinges identified by NLD and NLS+DIF analyses obtained from NLD and NLS analyses after applying DIF, respectively. It is observed that a nonlinear static analysis with a proper DIF is able to predict maximum dynamic plastic hinge rotations of beams within the affected bays with satisfactory accuracy.

The value of θ_{dNLD}/θ_y is calculated to specify the critical beam. Hinge No. 1 at the first floor which has maximum rotation ratio indicates critical beam. Results of NLS analysis with unamplified gravity loads for the mentioned building are presented in Table 4.

The values of $\left(\beta M_u - M_d\right)/M_p$ which define residual strength of members are calculated to determine the critical member. As shown in Table 4, the critical member is the beam that possesses the minimum value of $\left(\beta M_u - M_d\right)/M_p$, which is 0.261.

5.2 Analysis result

Obtained data points of the final DIF vs. min $\left[\left(\beta M_u - M_d\right)/M_p\right]$ for all structures against column removal are plotted in Fig. 5. As shown in Fig. 5, DIF generally increases as min $\left[\left(\beta M_u - M_d\right)/M_p\right]$ increases but when $\min\left[\left(\beta M_u - M_d\right)/M_p\right] < 0.27$, the slope is steep whereas the rate of increase is gentle when $\min\left[\left(\beta M_u - M_d\right)/M_p\right] > 0.27$ .As a result, two stages can be

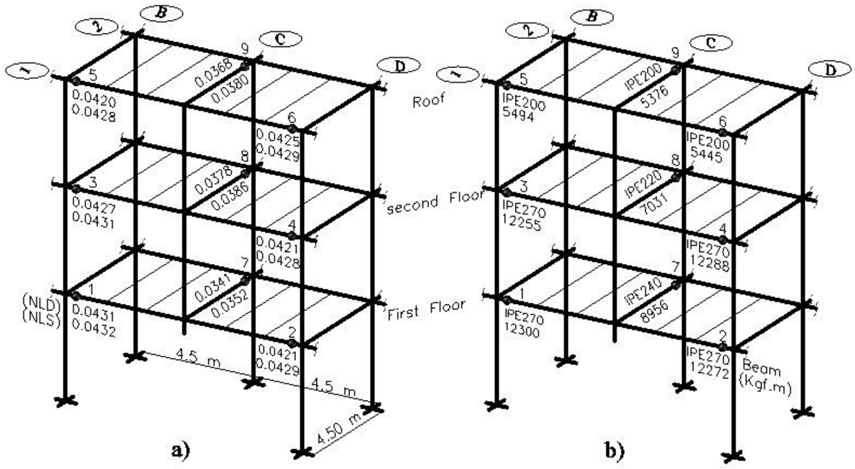

Fig. 4 plastic hinge location and rotation calculated by NLD and NLS analysis

Table 4 Second step analysis results of three storey building with 4.5 meter span length

Hinge number	**BEAM** IPE	M_d NLS	M_p	M_u	$\frac{(\beta M_u - M_d)}{M_p}$ (NLS)
1	270	12300	12196.8	15490	**0.261**
2	270	12272	12196.8	15490	**0.264**
3	270	12255	12196.8	15490	**0.265**
4	270	12288	12196.8	15490	**0.263**
5	200	5494	5569.2	7073	**0.284**
6	200	5445	5569.2	7073	**0.292**
7	240	8956	9248.4	11746	**0.302**
8	220	7031	7182	9121.1	**0.291**
9	200	5376	5569.2	7072.9	**0.305**

defined for damaged structures before applying DIF.

Detailed discussion about these observations is as follows.

For a column removal scenario, min $\left[(\beta M_u - M_d)/M_p\right] < 0.27$ indicates that the critical member in the damaged building is in inelastic stage and the moment demand resides within the post-yield stage. When $\min\left[(\beta M_u - M_d)/M_p\right] > 0.27$, this means that the critical member of the damaged structure is in elastic stage and the critical member possesses more residual strength than the structure in the inelastic stage.

Accordingly, the DIF equals 2 if the frames were to behave in a perfectly linear manner $\left(\min\left[(\beta M_u - M_d)/M_p\right] > 0.77\right)$. As a result, the actual DIF should be less than 2 and varies with the level of damping ratio and nonlinearity for a given column removal scenario.

In this study, DIF is considered 1.98 for $\min\left[(\beta M_u - M_d)/M_p\right] \geq 0.77$.

Fig. 5 shows two curve fittings, which are carried out to drive empirical equations for each stage. The curve fitting of these data points are conducted considering minimum and maximum obtained DIF values from Eq. (5).

As observed in Fig. 5, all data points having $\min\left[(\beta M_u - M_d)/M_p\right] < 0.27$ are related to the buildings that cannot remain in elastic stage against sudden column removal; hence, these structures do not possess a large residual strength and have a small DIF compared to structures with $\min\left[(\beta M_u - M_d)/M_p\right] > 0.27$. Reduction of DIF in inelastic stage is because of the high level of nonlinearity and ductility of the structure. It is expected that DIF will decrease by decrease in the function value of $\min\left[(\beta M_u - M_d)/M_p\right]$. In inelastic stage, curve fitting is carried out considering the minimum value of the obtained DIF. The obtained equation in this stage is as follows

$$DIF = 1.04 - \frac{0.02}{\gamma - 0.32} \tag{6}$$

As observed in Fig. 5, all data points having $\min\left[(\beta M_u - M_d)/M_p\right] > 0.27$ are related to the buildings designed for high levels of seismic load. Therefore, these buildings possess large values of residual strength due to progressive collapse. Curve fitting is carried out to empirically derive the following equation

$$DIF = 1.17 - \frac{0.2}{\gamma - 1.02} \tag{7}$$

The empirical DIF formulas can be used in NLS analysis to assess the progressive collapse potential of 3-D steel moment frame buildings. Consequently, the following step is used in NLS analysis to assess steel moment frame buildings under the column removal scenario.

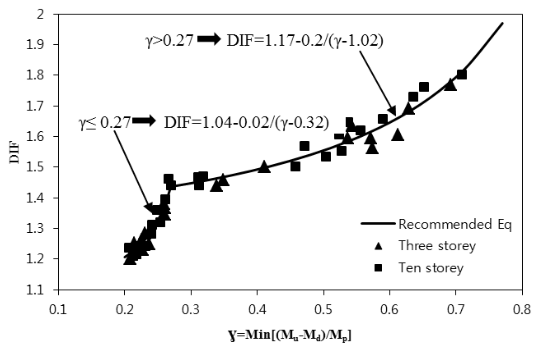

Fig. 5 Dynamic increase factor as a function of Min[(βM_u-M_d)/M_p.

Step A: Applying unamplified gravity loads (GN) statistically to the structure with a removed column to determine min $\left[\left(\beta M_u - M_d\right)/M_p\right]$ in NLS analysis.

Step B: Using the proposed empirical DIF formulas to calculate DIF.

Step C: Continue NLS analysis from Step "A" by applying additional gravity loads $\left[\left(DIF-1\right)\times G_N\right]$ on all bays affected by the removed column.

6. Conclusions

In this study, a new method for progressive collapse analysis of buildings is proposed based on the alternative load path. For this purpose, a series of moment frame structures with different span lengths and number of stories were designed to assess effective parameters on DIF. It is shown that the gravity loads and characteristics of members around the removed column are effective in calculating DIF.

The level of nonlinearity and residual strength of a member is effective in determining the critical member in a damaged structure. Residual strength depends on ductility and strain-hardening slope of the moment rotation diagram of a member. The percentage of the overall residual strength of the damaged frame under a given column removal scenario is measured as $\min\left[\left(\beta M_u - M_d\right)/M_p\right]$. Numerical results from illustrative analysis of building frames reveal that when min $\left[\left(\beta M_u - M_d\right)/M_p\right] > 0.27$, the damaged frame's response is essentially elastic. In contrast, when $\min\left[\left(\beta M_u - M_d\right)/M_p\right] < 0.27$, it means the damaged frame is located in the inelastic range. In this range, due to higher demand ductility of a member, a smaller min $\left[\left(\beta M_u - M_d\right)/M_p\right]$ leads to a smaller DIF.

It is observed that gravity loads are not considered in the suggested DIF formula in UFC design guidelines. Therefore, an empirical formula including gravity loads and ductility demand is recommended that can be used in nonlinear static analysis.

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