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A new dynamic increase factor for nonlinear static alternate path analysis of building frames against progressive collapse

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ABSTRACT

This paper presents a new empirical method for calculating the dynamic increase factor (DIF) that is used to amplify the gravity loads on the affected bays of a building frame, when the nonlinear static alternate path analysis is carried out to predict the peak dynamic responses to sudden column removal. The new method defines the DIF as a function of max(M_u/M_p), where the maximum operator is applied to all beams within the affected bays immediately adjacent to and above the removed column, and M_u and M_p are the factored moment demand under original unamplified static gravity loads and the factored plastic moment capacity, respectively, of an affected beam. Therefore, 1 – max(M_u/M_p) directly, albeit approximately, measures the percentage level of the overall residual capacity of a building frame to remain essentially elastic while withstanding the dynamic effect of gravity loads upon sudden column removal, after the static effect of gravity loads has been resisted by the damaged frame. A step-by-step nonlinear static analysis procedure using the new DIF is described. As an illustration, empirical DIF formulas are derived from curve fitting data points generated by the nonlinear static alternate path analysis of three model steel moment frames originally designed to resist different levels of earthquake. It is found from the numerical examples that the new DIF is well correlated with max(M_u/M_p) for different column removal scenarios. Hence, the new DIF can be used with nonlinear static analysis in lieu of nonlinear dynamic analysis to assess the potential of building frames for progressive collapse.

1. Introduction

Recent years have seen a surge of research activities in evaluating the potential of buildings for progressive collapse, which is typically triggered by the missing of critical gravity load-bearing elements due to abnormal loading, and in designing/upgrading structures to mitigate such catastrophic risk [1–7]. Relevant standards and guidelines have become available to address the elevated concern over progressive collapse in design of building structures, especially those with significant security requirements [8,9].

The alternate path method is perhaps one of the most widely adopted approaches to systematically assessing the progressive collapse risk of building structures. Its popularity is largely credited to the inherent straightforwardness in both concept and implementation: notionally remove a structural element from the originally intact structure, then analyze the resulting damaged structure using a selected analysis procedure, and finally evaluate structural performance against prescribed acceptance criteria beyond which a structure is considered not having adequate capacity to prevent progressive collapse [9].

Three different analysis procedures (i.e., linear static, nonlinear static, and nonlinear dynamic) are available for investigating the load redistribution behavior of a building structure upon sudden removal of critical structural elements such as columns [3,9]. The linear static analysis is the simplest option as it relies on a single factor to take into account the complicated geometric/material nonlinearity and dynamic effects; as a result, this analysis option is unable to accurately predict the actual nonlinear, dynamic structural behavior following sudden element removal. In contrast, the nonlinear dynamic analysis is the most accurate yet the most expensive option because sophisticated finite element modeling is required to account for all possible types of nonlinearities; besides, computationally intensive time history calculation is needed to directly simulate the dynamic behavior of the damaged structure.

In view of both advantages and disadvantages of the above two analysis options, the nonlinear static analysis provides an appealing tradeoff option: although material and geometrical nonlinearities are still modeled, this option does not require the calculation of dynamic response time history. Instead, there are two ways to approximately account for the dynamic effects due to sudden element removal. One way is based on a balance between strain energy and external work to find the controlling structural responses [1,4,5]. The other way is to use a prescribed dynamic increase factor (DIF) to amplify the gravity loads within

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the bays that are immediately affected by a suddenly removed element, as illustrated in Fig. 1. The simple DIF approach has been adopted in the current progressive collapse design guidelines [9]. Therefore, accurate DIF quantification is key to reliably estimating the peak dynamic structural responses using the approximate nonlinear static analysis and hence to satisfactorily assessing the building potential for progressive collapse.

In the earlier editions of progressive collapse design guidelines, DIF was set equal to 2.0 regardless of the structural type and gravity loading of a specific building [8]. This value is based on the fact that the maximum dynamic deflection is twice the static deflection when a structure behaves in a perfectly linear elastic manner [10]. However, after losing a critical structural element, a building almost always responds nonlinearly, either geometrically or materially or both. As a result, it is expected that the DIF differs from 2.0, with the actual value depending on the specific level of nonlinearity the damaged building exhibits. Previous research has shown that the value of 2.0 is very conservative, that is, the actual dynamic effect of gravity loads on progressive collapse responses is much less than what is predicted by nonlinear static analysis using a DIF of 2.0 [2]. In particular, a DIF of 1.5 was suggested for alternate path analysis of steel moment frames using the nonlinear static option [2].

Recognizing the apparent drawback of using a constant value of 2.0 for DIF, the recently released new edition of the United Facilities Criteria (UFC) Design of Buildings to Resist Progressive Collapse adopts empirically derived DIFs to account for the dynamic effect when using nonlinear static analysis to best possibly reproduce the peak dynamic responses of a structure subject to sudden element removal [9,11]. For example, the DIF of a steel moment frame is expressed in UFC as a function of min(θ_{acc}/θ_y), where θ_y and θ_{acc} are the yield rotation angle and the prescribed maximum acceptable plastic hinge rotation angle, respectively, for each of the structural components (columns excluded) that contribute to progressive collapse resistance and are within the immediately affected bays.

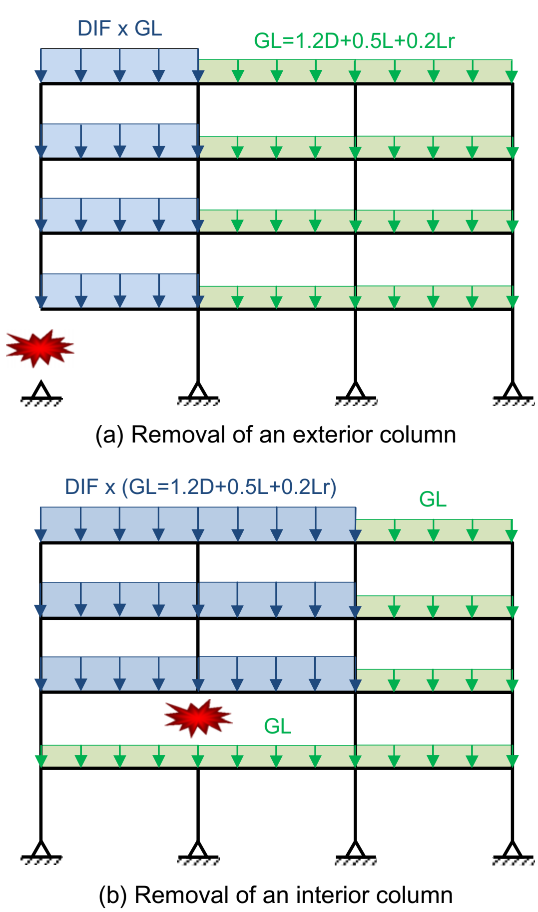

Fig. 1. Illustration of DIF-amplified gravity loads (GLs) for nonlinear static alternate path analysis.

It should be noted that the actual level of post-yield nonlinearity, as affected by both gravity loading and structural capacity, is not fully considered in the DIF formulation of the UFC guidelines. For example, UFC would give the same DIF value regardless of the specific gravity loading, since min(θ_{acc}/θ_y) depends only on the mechanical properties of the affected structural members. It is easily understandable that, however, when having a larger structural capacity and/or subject to lesser gravity loading, the damaged structure would have more residual capacity to bridge over the sudden removed element. Conversely, a damaged structure having a smaller capacity and/or subject to greater gravity loads would be more susceptible to progressive collapse. Any of the cases clearly affects the dynamic structural responses. Therefore, it is necessary to adjust the DIF accordingly for the nonlinear static analysis in order to produce static responses that can best match the affected dynamic responses.

This paper proposes a new method for empirically calculating the DIF for the nonlinear static alternate path analysis to assess the potential of a building frame for progressive collapse. The new DIF is able to directly take into account the level of overall residual capacity of a frame upon sudden column removal. Beam flexural strength is considered the primary source of structural capacity for a building frame to bridge over a removed column [6]. A step-by-step nonlinear static analysis procedure based on the new DIF is outlined. As an illustration, three nine-story, five-bay steel moment frames, which were originally designed to have different levels of seismic resistance, are used to preliminarily derive empirical DIF equations for nonlinear static alternate path analysis of framed building structures.

2. New DIF calculation method and nonlinear static analysis procedure

2.1. DIF calculation

The new DIF is defined as a function of max(M_u/M_p), where the *max* operator is applied to all affected beams that are directly adjacent to and above the removed column, and M_u and M_p are the factored moment demand under original unamplified static gravity loads and the factored plastic moment capacity, respectively, of an affected beam. Therefore, $1 - \max(M_u/M_p)$ directly, albeit approximately, measures the percentage level of the overall residual capacity of a frame to remain essentially elastic while withstanding the dynamic effect of gravity loads upon sudden column removal, after the static effect of gravity loads has been resisted by the damaged frame.

For a given column removal scenario, the DIF is obtained such that the structural responses from nonlinear static analysis using the DIF-amplified gravity loads best match those from nonlinear dynamic analysis. The process of calculating the DIF for the damaged frame under a given column removal scenario takes the following steps:

Step 1: Perform a nonlinear dynamic analysis to obtain the maximum plastic hinge rotation $\theta_{\max,\mathrm{ND}}$ among all beams within the affected bays and the maximum vertical displacement $\Delta_{\max,\mathrm{ND}}$ at the column removal location. To do so, statically apply gravity loads to the intact frame, from which end forces of the to-be-removed column are recorded; then statically apply gravity loads and end forces (in opposite directions) to the damaged frame; once static equilibrium is reached, instantly remove the applied end forces from the damaged frame to start the dynamic response time history analysis, from which the maximum dynamic responses are found.

Step 2: Using each of a range of trial DIFs, perform a nonlinear static analysis to obtain the maximum plastic hinge rotation $\theta_{\max,\mathrm{NS}}$ among all beams within the affected bays and the maximum vertical displacement $\Delta_{\max,\mathrm{NS}}$ at the column removal location.

Step 3: Determine the trial DIF that minimizes the difference between the responses from nonlinear dynamic analysis and from nonlinear static analysis. Use this trial DIF as the final DIF for the given column removal scenario. That is,

$$\min\left(\left|\frac{\theta_{\max,NS}-\theta_{\max,ND}}{\theta_{\max,ND}}\right|+\left|\frac{\Delta_{\max,NS}-\Delta_{\max,ND}}{\Delta_{\max,ND}}\right|\right)\rightarrow \text{final DIF} \quad (1)$$

Step 4: Statically apply original unamplified gravity loads to the damaged frame (note that the applied loads in this step are different from those in Step 1), from which obtain M_u for each beam within the affected bays; also calculate the plastic moment capacity for each affected beam as $M_p = \Omega_0 \phi Z F_y$ [12], where Ω_0 = over-strength factor, ϕ = strength reduction factor, Z = cross-sectional plastic modulus, and F_y = steel yield stress; then find $\max(M_u/M_p)$.

After the above four steps, a data point of DIF vs. $\max(M_u/M_p)$ is obtained. This process is then repeated for all other column removal scenarios of the current building frame and also for all other representative building frames. Finally, curve fitting of these data points is conducted to find empirical formulas for calculating the DIF. Fig. 2 illustrates the above major steps for DIF calculation.

2.2. Nonlinear static analysis

After the empirical DIF formulas are obtained, they can be used in the future nonlinear static analysis to assess the potential of similar-type building frames for progressive collapse. Under a given column removal scenario, the nonlinear static analysis procedure using the new DIF includes the following steps:

Step a: Same as Step 4 in Section 2.1 to statically apply original unamplified gravity loads to the damaged frame and find $\max(M_u/M_p)$.

Step b: Calculate the DIF using the empirical formulas derived in Section 2.1.

Step c: Continue from Step a by statically applying additional gravity loads calculated as the original gravity loads on all affected bays multiplied by a factor of (DIF-1).

Step d: Check if the resulting deformation and internal actions of the damaged frame meet the acceptance criteria set forth in relevant design guidelines [9,12].

The above steps are repeated for all other pre-selected column removal scenarios. If the frame satisfies the prescribed acceptance criteria under all column removal scenarios, the frame is considered having the capability of redistributing gravity loads to resist progressive collapse. Otherwise, if the frame violates the acceptance criteria for any column removal scenario, the frame is considered not having enough load redistribution capability to withstand progressive collapse.

Note that, compared with the existing nonlinear static analysis procedure using DIFs [9], the new nonlinear static analysis procedure does not require any extra modeling or computational effort. The only difference is that $\max(M_u/M_p)$ as opposed to $\min(\theta_{acc}/\theta_y)$ is calculated to determine the DIF. The value of this new parameter depends on the specific column removal scenario under consideration, and it can be easily obtained as a by-product of the nonlinear static analysis because the original gravity loads need be applied to the damaged frame anyway.

3. Illustrative application to steel building frames

The proposed DIF calculation method is now illustrated using steel building frames. It is worth noting that, although only steel frames are selected as a convenient example in the present study, this DIF calculation method itself is general-purpose and thus is equally applicable to any type of building frames.

3.1. Model steel moment frames

The three planar nine-story, five-bay steel moment frames with a same elevation (Fig. 3) are adapted from the seismically designed model steel frames representative of post-Northridge steel moment frames located in Los Angeles (LA), Seattle (SE), and Boston (BO), respectively [13]. The gravity loads are the same for all three buildings: dead loads of 96 psf (4.60 kPa) for each floor and 83 psf (3.97 kPa) for the roof, including steel self weight assumed as 13 psf (0.62 kPa), and reduced live loads of 20 psf (0.96 kPa) for each floor and the roof.

In the following progressive collapse analysis, the steel material has a yield strength of 50 ksi (345 MPa) and a tensile strength of 65 ksi (450 MPa). An over-strength factor Ω_0 of 1.1 is used to convert steel strength from lower-bound to expected values [12]. The ASCE 7 load combination for extraordinary events and resistance factors are used to define the factored loads and structural strength [14]. According to ASCE 7-10 Section 2.5.2.2, in order to assess the residual load-carrying capacity of a damaged frame, a gravity load combination of 1.2D + 0.5L + 0.2Lr shall be used, where D, L, and Lr are the dead load, floor live load, and roof live load, respectively. A strength reduction factor ϕ of 0.9 is used for calculating the plastic moment capacity of steel beams.

Member sizes for the three steel frames are listed in Table 1. As shown in Fig. 3, column members between any two adjacent splices have the same section size along each of the exterior or interior column lines. A single section size is used for all beams at a given floor or roof level. The naming of structural members in the frames is as follows. A column is named by the column line and story where it is located. For example, the column in the second story along the column line A is denoted as A-2, and the column in the sixth story along the column line C is designated as C-6. Similarly, a beam is named by the bay and floor/roof where it resides. For example, the middle beam on the seventh floor between column lines C and D is denoted as CD-7.

3.2. Structural modeling

The DRAIN-2DX program [15] is used as a computational tool for investigating the load redistribution behavior within the damaged steel frame after a column is removed. The structural modeling techniques for progressive collapse analysis of steel moment frames using DRAIN-2DX follow those used in [16]. Specifically, a planar analytical structural model is created to account for both geometrical nonlinearity (by updating the geometric stiffness based on truss bar element approximation) and material nonlinearity (by defining concentrated plastic hinges). Type-02 elements are used to model columns and beams for which a zero-length plastic hinge can form at any end of a member and yield in bending only. The yield moment of a plastic hinge is defined in Step 4 of Section 2.1. According to ASCE 41-06 [12], the yield rotation angle θ_y of a beam can be calculated in radians as $ZLF_y/6EI$ for a framed beam and $ZLF_y/4EI$ for a cantilevered beam, where Z, I, and L are

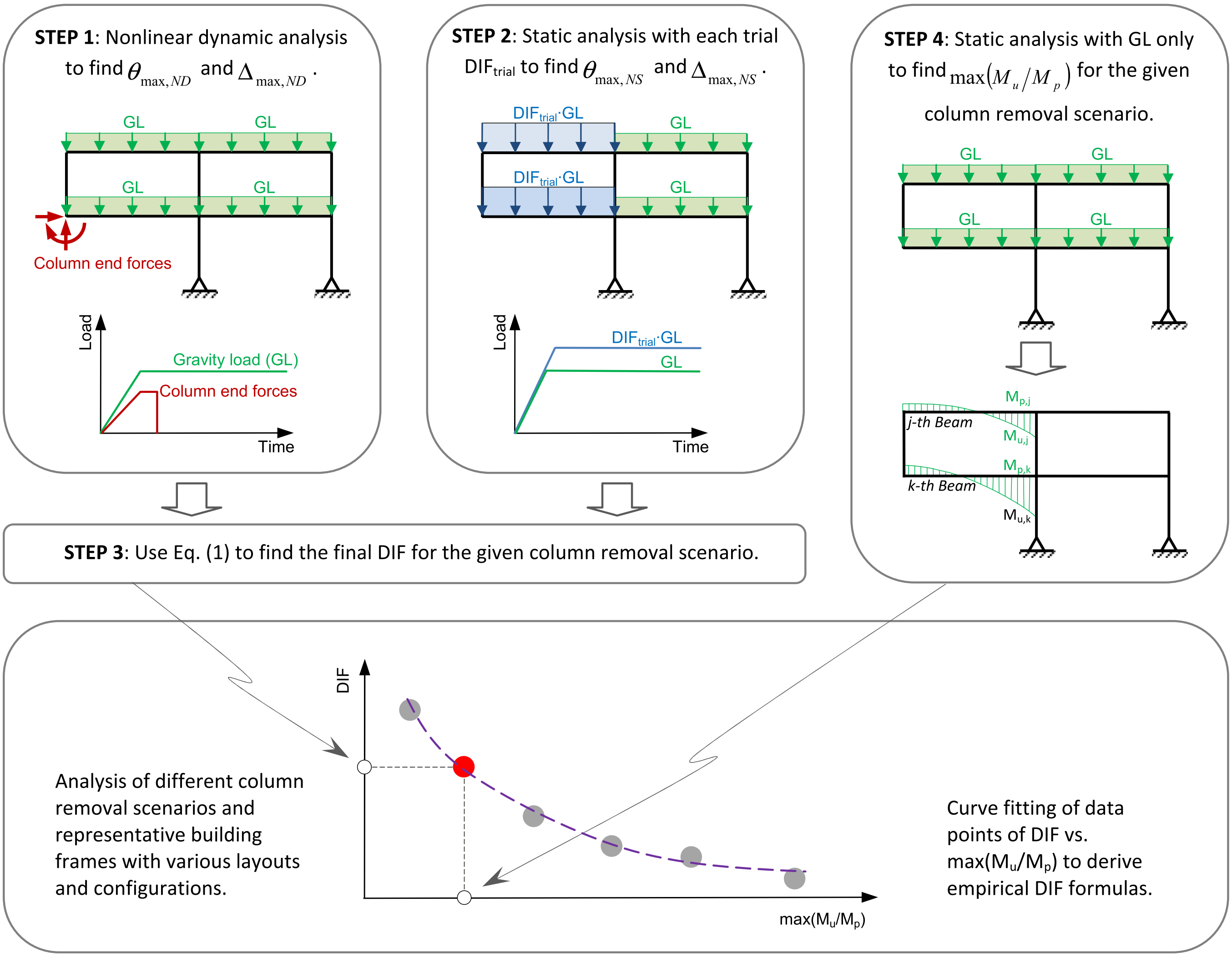

Fig. 2. Illustration of the steps to obtain the data point of DIF vs. $\max(M_u/M_p)$ for a given column removal scenario and how these data points are used to derive empirical DIF formulas.

the plastic section modulus, moment of inertia, and length, respectively, of the beam; E and F_y are the elasticity modulus and yield stress, respectively, of the beam steel material. A strain-hardening ratio of 3% is considered for the post-yield phase of steel members. Fully restrained moment connections are assumed. A critical damping ratio of 5% is used for the nonlinear dynamic analysis.

3.3. Column removal scenarios and DIFs

To completely investigate the load redistribution behavior of a steel frame, all columns within the frame are subject to removal in this study. Due to symmetry in the frame layout, only half of the columns in each story are to be removed, one at a time. Therefore, a total of $3 \times 9 = 27$ different column removal scenarios are considered for each of the three steel frames. Under each column removal scenario, firstly nonlinear dynamic analysis is carried out following the process described in Step 1 of Section 2.1 to obtain the maximum vertical displacement at the removed column location and maximum plastic hinge rotations. These dynamic responses are then used as a target in the subsequent nonlinear static analysis, which uses a range of trial DIFs varying from 1.0 to 2.0 with an increment of 0.01; the DIF that makes the nonlinear static responses best match the nonlinear dynamic responses is considered the final DIF for the column removal scenario under consideration, as described in Eq. (1).

3.4. Results and discussion

Results from two specific column removal scenarios are provided to illustrate the above calculation process. In the first example, the exterior column A-2 is removed from the BO frame. Fig. 4 plots the time history of the vertical displacement at the column removal location as obtained from nonlinear dynamic analysis. The maximum dynamic displacement is 3.91 inches (99.3 mm) that occurs at 0.70 s after the column has been suddenly removed. The final DIF for this column removal scenario turns out to be 1.63, which leads to a nonlinear static displacement of 3.90 inches (99.1 mm). In the second example, the interior column C-6 is removed from the SE frame. Fig. 5 shows the locations and maximum rotations of the plastic hinges identified by nonlinear dynamic analysis and also by nonlinear static analysis (with a final DIF of 1.40). It is observed that, except the very small plastic hinge rotations in beams BC-7 and CD-7 occurring at the end nodes that are near the upper node of the removed column, the nonlinear static analysis is able to reproduce the maximum dynamic plastic hinge rotations of beams within the affected bays with satisfactory accuracy, the largest and average percentage errors being 12.8% and 4.7%, respectively.

Fig. 6 plots the data points of the final DIF vs. $\max(M_u/M_p)$ for all column removal scenarios of each of the three steel frames. The symbol for each data point denotes the specific frame (LA, SE, or

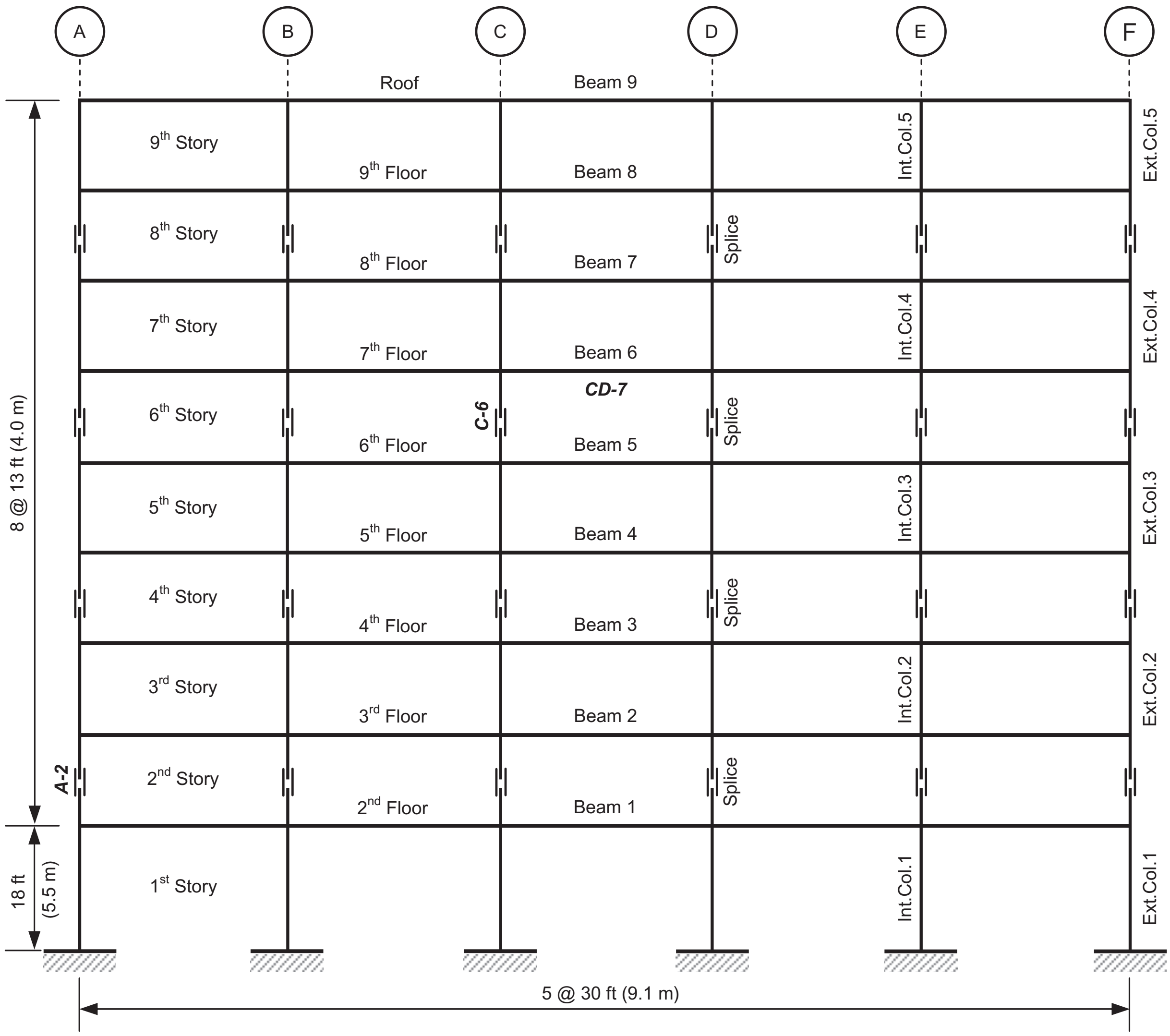

Fig. 3. Elevation of the model steel moment frames.

Table 1
Member sizes of three steel building frames.

Member ID[a]	LA	SE	BO
Beam 9	W24X62	W21X62	W12X53
Beam 8	W27X94	W21X62	W16X67
Beam 7	W27X102	W21X62	W18X97
Beam 6	W33X130	W24X76	W21X101
Beam 5	W33X141	W27X94	W21X101
Beam 4	W33X141	W27X94	W21X101
Beam 3	W33X141	W27X94	W21X101
Beam 2	W36X150	W27X114	W33X141
Beam 1	W36X150	W27X114	W33X141
Ext. Col. 5	W14X233	W24X131	W14X109
Ext. Col. 4	W14X257	W24X162	W14X159
Ext. Col. 3	W14X283	W24X207	W14X211
Ext. Col. 2	W14X370	W24X229	W14X257
Ext. Col. 1	W14X370	W24X229	W14X283
Int. Col. 5	W14X257	W24X131	W14X193
Int. Col. 4	W14X283	W24X162	W14X311
Int. Col. 3	W14X370	W24X207	W14X398
Int. Col. 2	W14X455	W24X229	W14X455
Int. Col. 1	W14X500	W24X229	W14X500

[a] Refer to Fig. 3 for the definition of member IDs.

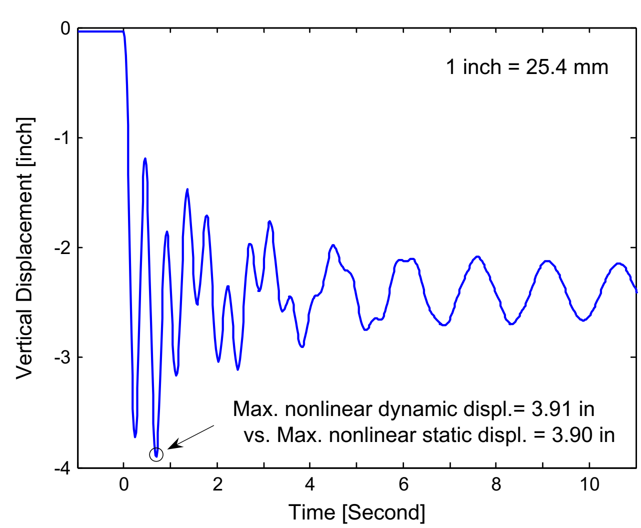

Fig. 4. Time history of the vertical displacement at the upper node of the suddenly removed column A-2 from the BO frame.

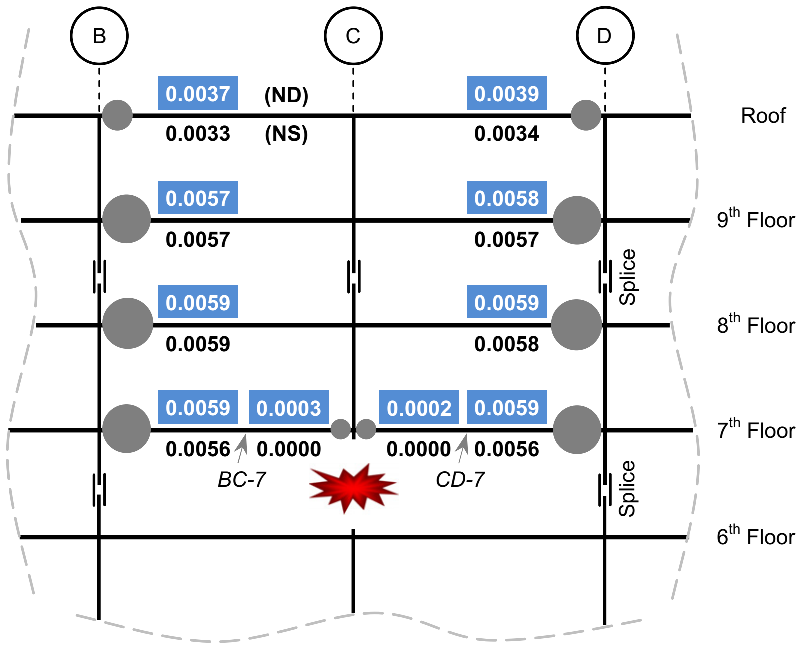

Fig. 5. Plastic hinge locations and rotations (unit: radian) calculated by nonlinear dynamic (ND) analysis and nonlinear static (NS) analysis, respectively, after column C-6 is removed from the SE frame.

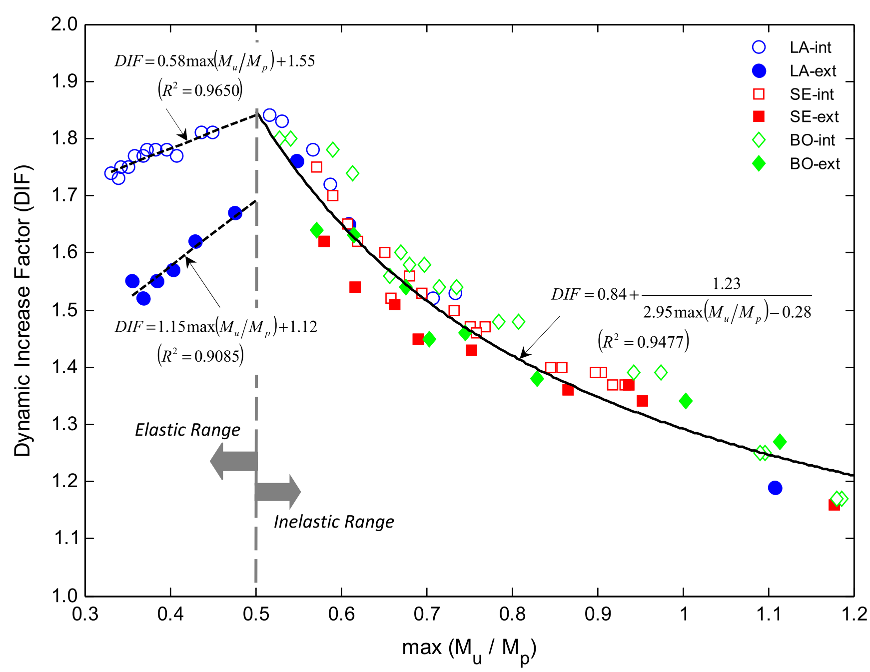

Fig. 6. Dynamic increase factor as a function of max(M_u/M_p).

BO) the data point belongs to and the location of column (exterior or interior) whose removal generates the data point. Recall that $1 - \max(M_u/M_p)$ is calculated as an indicator for the percentage level of residual capacity of a damaged frame under original unamplified gravity loads. Two distinct trends are clearly observed in Fig. 6: firstly, when $\max(M_u/M_p) \leqslant 0.5$, DIF generally increases as $\max(M_u/M_p)$ increases; secondly, when $\max(M_u/M_p) > 0.5$, DIF generally decreases as $\max(M_u/M_p)$ increases. Detailed discussion of these observations is as follows.

3.4.1. Cases of adequate residual capacity available in the damaged frame

As is well known, for an undamped at-rest linear elastic structure subject to a suddenly applied load, the dynamic amplification factor on the statically calculated displacement is 2. Or alternatively, the statically applied load shall be doubled in order to recover the peak dynamic displacement when the load is actually applied promptly. For a column removal scenario, $\max(M_u/M_p) \leqslant 0.5$ indicates that the damaged frame has adequate residual capacity to remain essentially elastic upon sudden removal of the target column. Accordingly, the DIF would be exactly equal to 2 if the frame were to behave in a perfectly linear manner. However, the damaged frame inevitably undergoes geometric nonlinearity even if it responds elastically. As a result, the actual DIF should be less than 2 and varies with the level of geometric nonlinearity for a given column removal scenario. Meanwhile, some of the vibrational energy is dissipated through various damping mechanisms that may be considered in nonlinear dynamic analysis, further reducing the dynamic responses and hence the value of DIF.

As observed in Fig. 6, all data points having $\max(M_u/M_p) \leqslant 0.5$ are associated with the LA frame (i.e., the hollow/solid circle symbols in Fig. 6). This is because the LA frame is designed to resist the highest level of seismic loads among all three frames and hence expectedly has a large residual capacity to remain elastic while withstanding gravity-load-induced progressive collapse [17,18]. As aforementioned, the general trend is that DIF increases as $\max(M_u/M_p)$ increases (while being less than 0.5).

Besides, it is noticed that, for a given $\max(M_u/M_p)$ that is less than 0.5, an exterior column removal scenario (represented by a solid symbol) has a smaller DIF than an interior column removal scenario (represented by a hollow symbol). This may possibly be attributed to the relatively higher level of geometrical nonlinearity due to the asymmetry exhibited in the frame after an exterior column is removed, mobilizing structural members within only fewer bays to redistribute the gravity loads.

Curve fitting is carried out to empirically derive the following linear equation for exterior column removal scenarios with $\max(M_u/M_p) \leqslant 0.5$:

$$\mathrm{DIF} = 1.15\max(M_u/M_p) + 1.12 \tag{2}$$

Similarly the linear equation for interior column removal scenarios with $\max(M_u/M_p) \leqslant 0.5$ is empirically obtained as:

$$\mathrm{DIF} = 0.58\max(M_u/M_p) + 1.55 \tag{3}$$

For the exterior column removal scenarios with $\max(M_u/M_p) \leqslant 0.5$, the smaller $\max(M_u/M_p)$ turns out to be associated with removal of an exterior column in a lower story level. This could be attributed to the situation that removal of a column from an upper story causes a low level of overall geometrical nonlinearity compared with removal of a column from a lower story, thereby leading to a smaller DIF for the lower-story column removal scenario. A similar trend is also observed for the interior column removal scenarios with $\max(M_u/M_p) \leqslant 0.5$, although in this case the trend is less pronounced, as evidenced by the slope of the fitted line dropping from 1.15 to 0.58.

3.4.2. Cases of inadequate or no residual capacity available in the damaged frame

As $\max(M_u/M_p)$ exceeds 0.5, it is more and more likely that a damaged frame does not have enough residual capacity to remain elastic when subject to sudden column removal. It is expected that the damaged frame exhibits an eleavated level of material nonlinearity, as the dynamic moment demand may well exceed the plastic moment capacity for certain beams within the affected bays. The material and geometrical nonlinearities interact with each other, driving the damaged frame far into a post-yield stage. Accordingly, plastic hinges begin to occur in structural members and vibrational energy is further dissipated as plastic hinges rotate. As a result, the DIF decreases as the level of nonlinearity increases, or roughly equivalently, as $\max(M_u/M_p)$ increases, i.e., as the residual capacity decreases. This general trend is clearly observed in Fig. 6. The following single empirical equation is derived by curve fitting the data points associated with both exterior and interior column removal scenarios where $\max(M_u/M_p) > 0.5$:

$$\mathrm{DIF} = 0.84 + \frac{1.23}{2.95\max(M_u/M_p) - 0.28} \tag{4}$$

Similar to what has been observed for the data points with $\max(M_u/M_p) \leqslant 0.5$, for data points with $\max(M_u/M_p) > 0.5$, the scenario of removing an exterior column (represented by a solid symbol) generally has a small DIF compared with the scenario of removing an interior column (represented by a hollow symbol) with practically identical $\max(M_u/M_p)$. As stated before, this may be caused by a higher level of geometrical and/or material nonlinearity exhibited in exterior column removal scenarios. However, the DIF difference between the exterior and interior column removal scenarios with $\max(M_u/M_p) > 0.5$ seems much less significant than the scenarios with $\max(M_u/M_p) \leqslant 0.5$, indicating that material nonlinearity likely plays a more and more important role than geometrical nonlinearity in affecting the DIF as $\max(M_u/M_p)$ goes beyond 0.5. This is exactly the reason why it is decided that only a single curve is fitted for the data points associated with both exterior and interior scenarios of the example building frames when $\max(M_u/M_p) > 0.5$.

It is worth pointing out that Eq. (4) is in general agreement with the conclusion from a previous study [2], which conservatively suggests use of 1.5 for the DIF in nonlinear static alternate path analysis, assuming that the steel frame is loaded significantly into the inelastic range. Plugging the DIF of 1.5 into Eq. (4) back calculates $\max(M_u/M_p)$ as 0.73. This value confirms that the steel frame under this particular column removal scenario does not have much residual capacity to remain elastic while withstanding the dynamic effect of gravity loads and, therefore, the damaged frame is indeed loaded well into a yielding phase.

3.4.3. Further remarks

It should be noted that $\max(M_u/M_p) = 0.5$ is only conveniently used to demarcate the ranges of whether or not a damaged frame is expected to behave elastically or inelastically. It is possible that the actual threshold of $\max(M_u/M_p)$ beyond which a damaged frame begins to respond inelastically upon sudden column removal may be somehow different from 0.5, depending on the specific gravity load patterns and structural configuration. However, it is reasonable to expect that such a difference is small and may well be neglected from a practical point of view. It should also be emphasized that, in consistent with the current practice of nonlinear static alternate path analysis, the new DIF proposed in this paper is applied uniformly to all gravity loads within the directly affected bays only. It is possible that such a load amplification pattern may not best represent the actual dynamic effect of the gravity loads upon sudden column removal. Consequently, a discrepancy may inevitably exist between load redistribution calculated by nonlinear static analysis and that by nonlinear dynamic analysis. However, reducing such a discrepancy is out of the scope of the present study.

4. Conclusions

The new dynamic increase factor (DIF) proposed for use in nonlinear static alternate path analysis is directly related to the specific level of overall residual capacity that a damaged building frame can provide to remain elastic while redistributing the gravity loads after a critical column has been suddenly removed. The percentage level of the overall residual capacity of the damaged frame under a given column removal scenario is measured as $1 - \max(M_u/M_p)$, where M_u is the factored moment demand calculated using original unamplified gravity loads and M_p is the plastic moment capacity of a contributing beam within the bays directly affected by the removed column. Therefore, unlike DIFs in the existing design guidelines, the new DIF takes into account not only structural strength but also the specific level of gravity loads. This feature is particularly useful because it makes the new DIF applicable to a much wider range of analysis situations, for example, whether the damaged frame responds elastically or it undergoes significant plastic deformations.

Numerical results from illustrative analysis of three steel building frames reveal that, when $\max(M_u/M_p) \leqslant 0.5$, the damaged frame responds essentially elastically. In this elastic range, DIF is primarily affected by the level of geometrical nonlinearity a damaged frame exhibits. In contrast, when $\max(M_u/M_p) > 0.5$, most likely a damaged frame behaves inelastically. In this inelastic range, the larger the $\max(M_u/M_p)$ is, the smaller the DIF becomes, indicating that material nonlinearity plays a more important role than geometrical nonlinearity in affecting the DIF. In general, the DIF associated with removal of an exterior column is smaller than the DIF associated with removal of an interior one. This may be a result of a higher level of geometrical and/or material nonlinearity due to fewer bays being mobilized to redistribute the gravity loads after an exterior column has been removed.

Once again it is emphasized that, although the proposed DIF calculation method is expectedly applicable to any type of building frames, the conclusions drawn and the empirical formulas derived in this paper are solely based on illustrative analysis of three specific steel moment frames. Therefore, generalization of the conclusions and formulas to other layouts and/or structural types may not be immediately valid and hence further study is required. It is also noted that, although the proposed DIF is tested only for planar frame models in the present study, extending its application to three-dimensional building frame models should be straightforward.

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