ce/papers Proceedings in civil engineering

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ORIGINAL ARTICLE

eurosteel AMSTERDAM 2023

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Dynamic Increase Factors for progressive collapse anaylsis of steel structures accounting for column buckling

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Abstract

Man-made hazards, such as fire, explosions, or impacts, may induce the progressive collapse of structures, in which the localised failure spreads from the single affected structural component to other parts of the structure. A typical approach to model progressive collapse consists in performing static column removal analyses considering a Dynamic Increase Factor (DIF), whose determination becomes paramount to account for the dynamic effects related to a sudden column loss scenario. Current recommendations on the definition of such factor mainly consider a beam-type collapse in non-linear analyses, though different mechanisms, *e.g.,* column buckling, may govern progressive collapse events. This paper presents the determination of the DIFs through a numerical procedure for five steel structures with an increasing number of storeys. Both global and local imperfections are modeled to account for the geometric non-linearities of the structure and column buckling. DIF values are obtained considering two different Engineering Demand Parameters (EDPs), suited for describing beam-type and column-type mechanisms respectively. The evaluated DIFs are compared with the values recommended in the current UFC design prescriptions for progressive collapse, and considerations on the choice of the appropriate DIF values are provided.

Keywords

Dynamic Increase Factor, Progressive Collapse, Steel Structures, Column Buckling, Robustness

1 Introduction

Progressive collapse of structures may be induced by accidental events such as fires, explosions or impacts, where the local damage spreads from single elements to part or the entire structure. Disasters such as the collapse of the Ronan Point Building (London, 1968) [1], of the Murrah Federal Building (Oklahoma City, 1995) [2] and of the World Trade Center (New York, 2001) [3] showed the potential social and economic losses that the progressive collapse of a building might involve. Owing to the significant consequences such collapses may cause, many researchers worldwide investigated this research topic, often focusing on the most dangerous situations, like the sudden loss of a base column [4-10]. Since the middle of the last century an increasing understanding of the phenomenon has been achieved [4-10], and recommendations on progressive collapse are nowadays incorporated in several design codes worldwide [11-13].

A well-established procedure to investigate sudden column loss scenarios is the Alternate Path Method (APM). This consists of numerical simulations in which, after the application of gravity loads, a column loss is simulated, and the capability of the structure to redistribute the loads is assessed. This problem is often addressed by static analyses where the dynamic effects are indirectly considered by increasing the static loads through a Dynamic increase factor (DIF). The DIF provided in design codes for nonlinear static analyses is typically computed according to a ductile failure mode involving the plastic rotation of the structural elements, components and connections [11]. However, the dynamic behaviour does not depend solely on such characteristics, and different considerations may be more appropriate for other phenomena governing the collapse mechanism, *e.g.,* column buckling. Similarly, in the last few decades, several studies proposed different formulations for the DIF [14-18]; however, the literature is currently lacking extensive investigation of the actual dynamic effects when progressive collapse occurs with brittle mechanisms in steel structures, as for column buckling.

The present paper employs a procedure involving nonlinear static and dynamic analyses to evaluate the DIF of steel structures accounting for column buckling. For this purpose, both global and local imperfections are carefully considered in numerical simulation. Five steel moment resisting frames (MRF) of different heights are considered

https://doi.org/10.1002/cepa.2387 wileyonlinelibrary.com/journal/cepa

for case study purposes to evaluate the influence of the dynamic effects. The obtained DIFs are compared with the current UFC [11] recommendations, showing that the latter underestimate the dynamic effects. The paper is organised as follows: Section 2 presents the case study structures; Section 3 describes the Finite Element (FE) modelling strategy; Section 4 presents the procedure for the DIF evaluation and the results of the analysis; finally Section 5 provides some conclusive remarks.

2 Case studies structures

The procedure to evaluate the DIF was performed considering five case study structures consisting of seismically designed MRFs with different heights. The design was performed for a peak ground acceleration equal to 0.16g and complying with the Eurocodes [19-21] prescriptions. The buildings are characterised by 4 bays with 5 m spans in the *x*-direction, while the bay span is equal to 7 m in the *y*-direction. They have inter-storey heights of 3 m and total heights of 9 m, 18 m, 27 m, 36 m and 45 m, respectively, for the 3, 6, 9, 12 and 15-storey structures. Only plane frames in the *x*-direction were analysed in the present study. Figure 1 shows the elevation view of the 3-storey structure and indicates the column considered for the loss scenario. The frame has sections oriented with the major axis within the frame plane, and rigid, full-strength welded beam-column joints. S235 steel grade was used, with nominal yield strength f_y = 235 MPa, Young's modulus E = 210000 MPa and Poisson ratio v = 0.3. These MRFs were selected as benchmark case studies as they were already investigated in previous research works focusing on progressive collapse, and detailed information can be found in [22]. Table 1 summarises the steel cross-sections for columns and beams.

Table 1 Case studies: columns and beam design (adapted from [21])

N. of floors	Columns	Beams
3	HE280B floors 1 to 3	IPE500 floor 1 IPE400 floor 2 IPE360 floor 3
6	HE320B floors 1 to 3 HE220B floors 4 to 6	IPE550 floor 1 IPE450 floors 2 to 3 IPE400 floors 4 to 6 IPE360 floor 6
9	HE400B floors 1 to 3 HE280B floors 4 to 6 HE220B floors 7 to 9	IPE550 floor 1 IPE500 floors 2 to 3 IPE450 floors 4 to 6 IPE400 floors 7 to 8 IPE360 floor 9
12	HE500B floors 1 to 3 HE340B floors 4 to 6 HE280B floors 7 to 9 HE200B floors 10 to 12	IPE550 floor 1 IPE500 floors 2 to 6 IPE450 floors 7 to 9 IPE360 floors 10 to 12
15	HE650B floors 1 to 3 HE450B floors 4 to 6 HE340B floors 7 to 9 HE280B floors 10 to 12 HE200B floors 12 to 15	IPE550 floors 1 to 2 IPE500 floors 3 to 9 IPE450 floors 10 to 12 IPE500 floors 13 to 14 IPE450 floor 15

The Dead Load (DL) was applied on all floors and was equal to 5.0 kN/m^2, consisting of 3.0 kN/m^2 corresponding to the self-weight of a 12 cm thick concrete slab, and 2.0 kN/m^2 to account for the non-structural permanent components. The Dead Load owing to the self-weight of beams and columns was also considered and applied directly on the structural elements. The Live Load (LL) was assumed of 2.00 kN/m^2 and applied on all floors except at the roof level. Indeed, on the roof the Snow Load (SL) was assumed equal to 0.69 kN/m^2, based on Eurocode guidelines [19] for the Greek climate region in Zone III, 200 m of altitude and standard conditions. According to the UFC [11], the progressive collapse resistance of the frame is assessed by considering the following load combination:

$$q_d = 1.2\text{DL} + 0.5\text{LL} + 0.0\text{SL} \qquad (1)$$

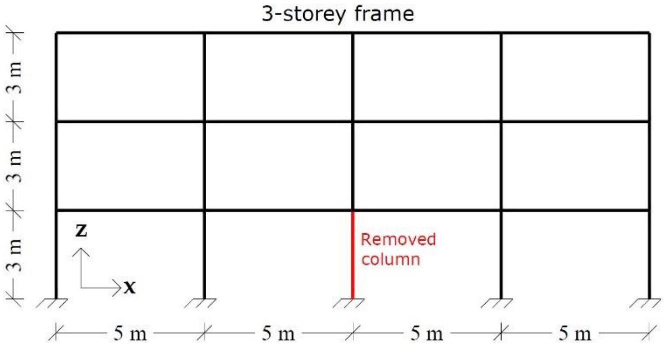

Figure 1 3-storey moment resisting frame case study

3 Numerical models

The numerical models of the 5 case studies were developed in OpenSees [23], considering both in-plane and out-of-plane local imperfections and global equivalent imperfections according to EN 1993-1-1 [20]. The modelling flexibility of OpenSees, allowed for increasing the model complexity only where needed, while simple and less computationally demanding features were employed elsewhere. In the latter case, analyses were later performed to confirm that such simplifications were not influencing the structural behaviour. A distributed plasticity approach was used for columns, including the elastic shear stiffness via the 'Section Aggregator' command. Conversely, a lumped plasticity approach was used for beams. Plastic hinges were simulated by the 'Parallel Plastic Hinge' (PPH) model proposed by Lee *et al.* [24], which accounts for the nonlinear response, including bending moment and axial force interactions. The 'Scissor Model' [25] was used to simulate the panel zone deformation at beam-column joints. To properly simulate the dynamic behaviour, preliminary analyses were performed to determine the discretisation of the masses along the beams. 33 concentrated masses at each story were considered since a finer discretisation did not produce significant variations in the results. Moreover, a Rayleigh damping with a damping ratio ξ equal to 5% was employed. Additional details on the validation of the numerical models can be found in [10]. Conservatively, the present study neglects the possible positive contribution of the slab to the progressive collapse resistance.

4 Dynamic Increase Factors (DIFs)

4.1 Numerical procedure for DIF evaluation

Two sets of analyses were performed to evaluate the dynamic effects induced by the sudden column removal. The first set consists of static gravity analyses performed on the structures with the column removed (Figure 2). The applied loads were gradually increased until the load factor λ reached the value of 1, corresponding to the situation where all the loads are applied. The load factor is defined as follows:

$$\lambda = \frac{\sum_{i=1}^{n} R_i}{Q_{tg}} \qquad (2)$$

where $\sum_{i=1}^{n} R_i$ is the sum of the vertical base reaction forces of the frame, and Q_{tg} is the load target the structure is supposed to bear in the specified situation.

The second set of analyses is subdivided into three consecutive steps (Figure 2). First, a gravity analysis is performed on the full structure, *i.e.,* with no column removal, to determine the vertical base reaction of the central column $R_c = R_3$. In the second step, the central column is removed, and the final state of the structure before removal is restored by gradually applying the base reaction recorded in the pushdown analysis R_c to the node above removal, together with the gravity loads. It was assessed that for the considered case studies neglecting the other reaction components, *i.e.,* shear force and bending moment of the removed column did not cause significant variations. In the final step, starting from the final state of step 2, a counterforce F_c is applied in a given time T_{Rem} to the node above removal to simulate the sudden column loss. The load magnitude is equal to the one of R_c, and T_{Rem} is taken as $(1/11)T_v$ according to GSA guidelines [13]. T_v is the period corresponding to the vertical vibration mode of the structure and is determined with a modal analysis of the structure with the removed column. In order to compare the two sets of analyses, the second set was exploited in an Incremental Dynamic Analysis (IDA) fashion simulating the behaviour of the structures for increasing values of the load factor λ.

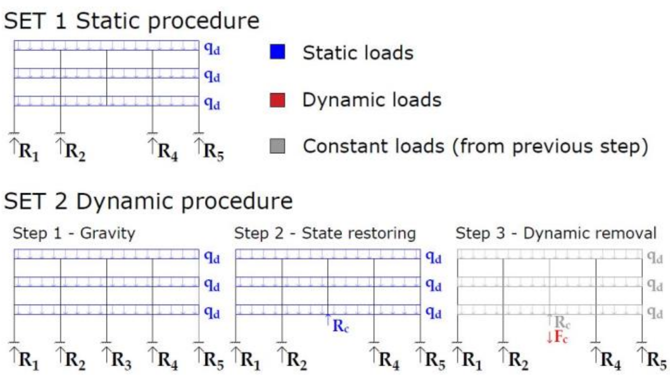

Figure 2 Static and dynamic analyses for the DIF evaluation

Since the dynamic effects may influence different aspects of the structural response, relevant Engineering Demand Parameters (EDPs) should be considered to evaluate such effects. The numerical simulations showed that exceedance of the maximum beam bending capacity and column buckling were the two main mechanisms governing the collapse of the investigated structures. Therefore, the displacement of the node above the removal δ and the axial compressive force in the columns adjacent to the removal N were identified as the most suited EDPs to evaluate the DIFs, which were defined as follows:

$$\mathrm{DIF}_{\delta}(\lambda) = \frac{\delta_D(\lambda)}{\delta_S(\lambda)} \qquad \mathrm{DIF}_{N}(\lambda) = \frac{N_D(\lambda)}{N_S(\lambda)} \qquad (3)$$

where the subscripts D and S are related to the dynamic and static procedures, *i.e.,* set 1 and set 2 in Figure 2, respectively.

4.2 Analysis results

For the sake of brevity, relevant results of the static and dynamic analyses are only shown for the 3-storey structure. Figure 3 and Figure 4 show the vertical displacement of the node above the removed column and the the axial compressive force in the column to the left of the removal for the IDA performed for the second set of analyses.

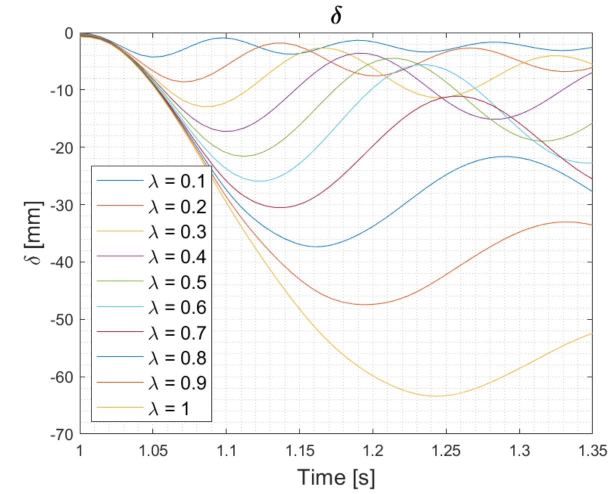

Figure 3 Dynamic removal analyses with increasing load factor for the 3-storey structure: vertical displacement of the node above the removed column

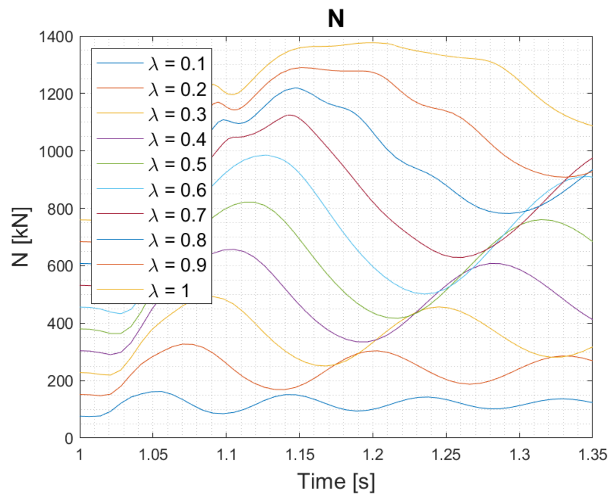

Figure 4 Dynamic removal analyses with increasing load factor for the 3-storey structure: axial force in the column on the left of the removed column

The force removal is applied dynamically, starting at time of 1 second, before which the 'State restoring' analysis, *i.e.,* step 2, was performed. Figure 3 and Figure 4 show how the λ affects both the peaks of the measured EDPs, and the vibration period in the dynamic analyses.This effect is related to the higher mass which is increased proportionally to the load. The increasing peaks obtained for increasing λ values were compared with the displacements and the axial forces obtained in the static procedure, as shown in Figure 5 and Figure 6. It can be observed that the structure exhibits a linear behaviour in the static procedure, and the displacement and the axial force increase proportionally with λ. On the contrary, the maximum displacement and axial force from the dynamic procedure increase linearly only up to λ=0.6, while a marked nonlinear behaviour can be observed afterward. This is mainly due to the nonlinear behaviour exhibited by columns and/or beams for higher λ values.

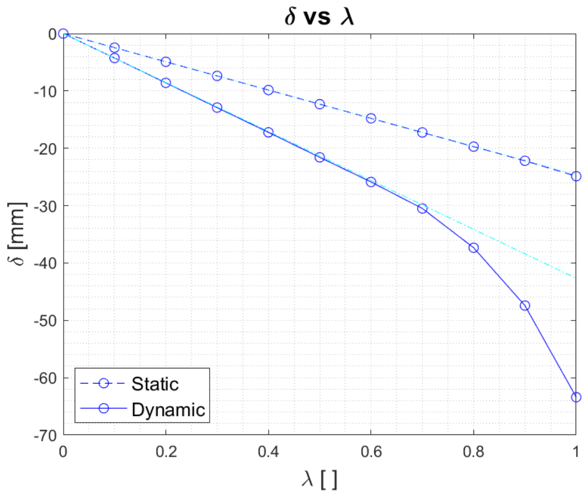

Figure 5 Maximum vertical displacement of the node above the removed column for the 3-storey structure: static vs dynamic analyses

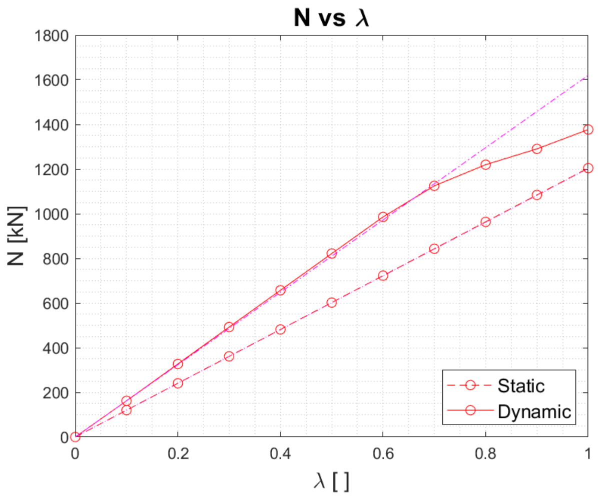

Figure 6 Maximum axial force in the column on the left of the removed column for the 3-storey structure: static vs dynamic analyses

The DIFs were calculated according to Eq.3 for all the structures, and their evolution with the load factor λ is shown in Figure 7. The DIF values are essentially constant up to λ=0.6, which can be defined as a reference threshold for the investigated structures, after which the variation in the values of DIFs becomes significant owing to a marked nonlinear behaviour of δ and N in the dynamic analyses. The DIFs obtained for the displacement is always higher than the one for the axial force and varies on a larger range, for instance, 1.32 to 1.75 vs 1.28 to 1.39 for the axial force for λ ≤0.6.

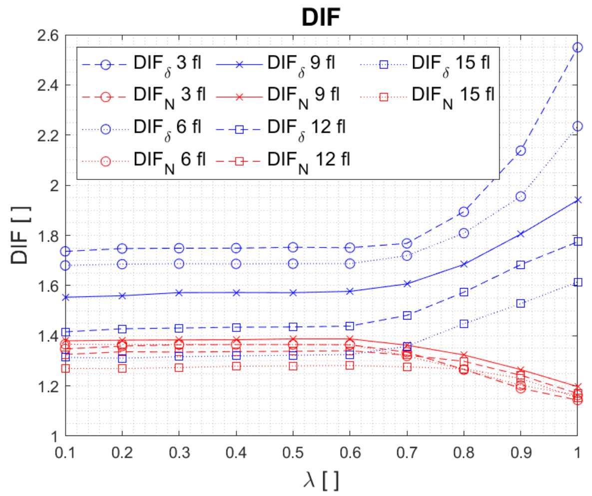

Figure 7 DIF$_\delta$ and DIF$_N$ vs load factor for all case studies

4.3 Modified DIF evaluation

In order to compare the results with the current recommendations in the UFC [11], the obtained DIF values were modified. Indeed, according to the above-described procedure, the DIF is evaluated by progressively incrementing a uniform load on all the spans. Conversely, according to the UFC [11] recommendations, the DIF amplifies only the loads on the bays above the removed column. Hence, to restore consistency between the formulations, new DIFs*, applied only to the beams above the removed column, is introduced. These DIFs* allow for reproducing the same vertical displacement above the removed column and stress state in the adjacent columns obtained by applying the DIFs values on the vertical loads on every span. The derivation of the DIFs* is based on the static schemes represented in Figure 8. A similar procedure for the DIF$_N$ was proposed in [10].

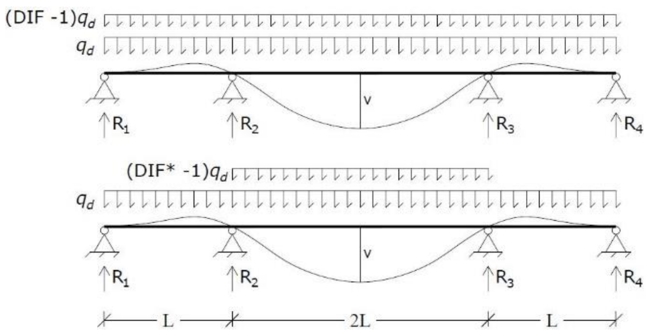

Figure 8 Static schemes for the definition of DIF* from DIF

As in the investigated case studies, 4 spans of equal length L and the same axial stiffness of the base restraints are considered in Figure 8. After removing the central base restraint, the two central spans are treated as a single span with a length 2L. The following formulations were obtained for the displacement above the removal δ and the axial force in the column adjacent to the removal N.

$$\mathrm{DIF}^*_\delta = 0.8125 \cdot \mathrm{DIF}_\delta + 0.1875 \qquad (4)$$

$$\mathrm{DIF}^*_N = 1.425 \cdot \mathrm{DIF}_N - 0.425 \qquad (5)$$

The two formulations were derived by imposing the equivalence between the two schemes in terms of vertical displacement v and reaction forces R_2 and R_3, respectively. Equal span lengths and the same base column, *i.e.,* same base restraint axial stiffness, are very common in structural design, and the contribution provided by additional spans, farther from the collapse location, is usually negligible [26]. Therefore, the provided formulation represents a good approximation in many structural applications. Nevertheless, the derivation of the formulation for the DIFs* can be based on other static schemes if necessary. The evolution of the new DIFs* values with the load factor is reported in Figure 9. The variation range is reduced for the new δ based dynamic increase factor (DIF*$_\delta$), while it is increased when axial forces are examined. For instance, at $\lambda \leq 0.6$ the range moves from 1.32-1.75 to 1.26-1.61 for the displacement and from 1.28-1.39 to 1.40-1.55 for the axial force.

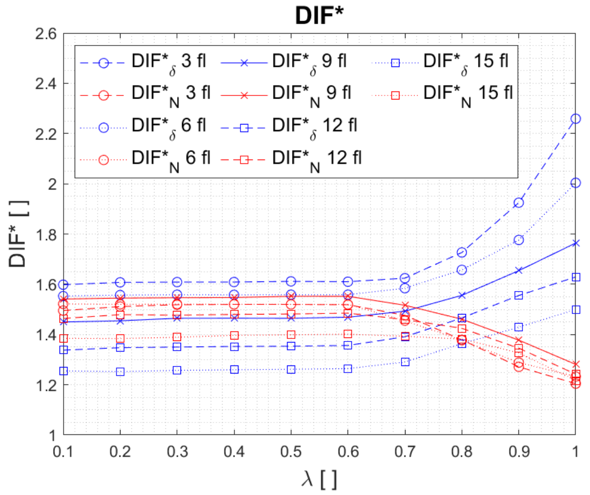

Figure 9 DIF*$_\delta$ and DIF*$_N$ vs load factor

A more concise and effective comparison is provided in Figure 10 in which the DIF and DIF* values for $\lambda = 0.6$ are compared. As mentioned, the overall variation of dynamic increase factors is reduced with the DIFs*. While for the DIF the displacement-based factors are always higher than the ones based on the axial force for each structure, the DIF*$_N$ is higher than the DIF*$_\delta$ for structures with more than 6 storeys. In particular, these preliminary analyses show that for structures in which the collapse mechanisms is governed by the behaviour of the beams, i.e. the 3 and 6 storey buildings, the DIF*$_\delta$ is higher than DIF*$_N$, whereas the DIF*$_N$ becomes higher than the DIF*$_\delta$ when column buckling is the main cause of the collapse, i.e. 9, 12 and 15 storey buildings. The DIF* values are compared with the dynamic increase factors recommended in the UFC guidelines [11], which for nonlinear static analyses is 1.24 except for the 15-storey structure, for which 1.22 is obtained. The derivation of these values is based on considerations of plastic rotations of any primary element, component, or connection in the model within the area that is loaded with the increased gravity load. The highest among the obtained factors should be considered, which in the investigated case studied was obtained by considering the welded unreinforced flanges connections. No indication is provided in the UFC guidelines regarding brittle failures for nonlinear static analyses, but a value of 2 is suggested for force-based mechanisms in linear analyses. Hence, in the absence of indications regarding nonlinear analyses, this value can be conservatively taken as a reference for brittle failures. It is noteworthy that the UFC values for nonlinear analyses underestimate the dynamic effects for the DIF*$_\delta$, while those for linear force-controlled mechanisms overestimates the DIF*$_N$.

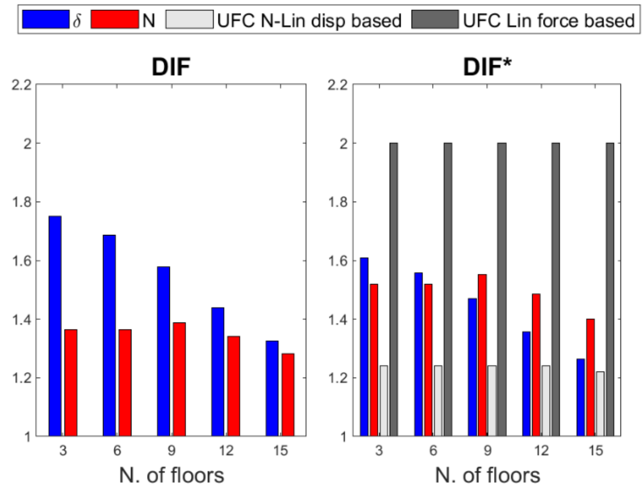

Figure 10 DIFs and DIF*s for λ=0.6

5 Conclusions

This paper examines the dynamic effects of a sudden central column removal by evaluating the Dynamic Increase Factor (DIF) for different steel structures with an increasing number of stories. A procedure involving nonlinear static and dynamic nonlinear analyses allowed for the definition of the DIF for five seismically designed case study Moment Resisting Frames (MRFs). OpenSees models incorporating mechanical and geometrical non-linearities were employed to evaluate the redistribution capacity of the structures and possible failure mechanisms, including the effects of column buckling. The Dinamic Increase Factors were evaluated considering two key Engineering Demand Parameters (EDPs) describing both the beam and the column behaviour, *i.e.,* the displacement above the removed column δ and the axial force in the column adjacent to removal N. These EDPs aim at evaluating the effects under beam-type collapse mechanisms and column buckling failure, respectively. Dynamic Increase Factors were assessed, also considering their application on all loads acting on beams (DIF) or only on those above the column removal zone (DIF*), as for the UFC recommendations. The results showed that the Dynamic Increase Factors might be considerably different from those suggested by the UFC recommendations. In particular, these preliminary results show that the UFC might underestimate the DIFs beam-type collapse mechanisms and overestimate those for force-controlled mechanisms, highlighting a significant need for more detailed and case-based evaluations. It should be observed that, though the Dynamic Increase Factors derived are meaningful for both δ and N, the most appropriate one may be determined based on the actual collapse mode of the investigated structure.

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