Steel and Composite Structures, *Vol. 22, No. 3 (2016) 677-690*
DOI: http://dx.doi.org/10.12989/scs.2016.22.3.677

Effects of damping ratio on dynamic increase factor in progressive collapse

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(*Received June 16, 2016, Revised October 18, 2016, Accepted October 23, 2016*)

Abstract. In this paper, the effect of damping ratio on nonlinear dynamic analysis response and dynamic increase factor (DIF) in nonlinear static analysis of structures against column removal are investigated and a modified empirical DIF is presented. To this end, series of low and mid-rise moment frame structures with different span lengths and number of storeys are designed and the effect of damping ratio in DIF is investigated, performing several nonlinear static and dynamic analyses. For each damping ratio, a nonlinear dynamic analysis and a step by step nonlinear static analysis are carried out and the modified empirical DIF formulas are derived. The results of the analysis reveal that DIF is decreased with increasing damping ratio. Finally, an empirical formula is recommended that relates to damping ratio. Therefore, the new modified DIF can be used with nonlinear static analysis instead of nonlinear dynamic analysis to assess the progressive collapse potential of moment frame buildings with different damping ratios.

Keywords: progressive collapse; alternate load path; nonlinear static analysis; dynamic increase factor; damping ratio

1. Introduction

In the commentary of the ASCE Standard "Minimum design loads for buildings and other structures" (ASCE/SEI 7-10 2010), progressive collapse is defined as "the spread of an initial local failure from element to element, eventually resulting in the collapse of an entire structure or a disproportionately large part of it." So far, many triggering events have caused catastrophic outcomes in structures such that these events have had the most effective result in improving design guide lines. The Ronan Point apartment tower collapse on May 16, 1968 is one of the most famous progressive collapse failures (Griffiths *et al.* 1968). The mentioned 22-storey building was built of precast concrete bearing wall system. The collapse was initiated by a gas explosion in a kitchen on the 18th floor which blew out a wall panel near the corner of the apartment. Failure of the corner bay propagated up and down and at last covered almost the whole height of the building. The Alfred P. Murrah building is the other famous example of a progressive collapse in Oklahoma City in 1995. A bomb blast that destroyed three perimeter columns of the nine-storey building led

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

to the collapse of approximately 50% of the total floor area of the building (FEMA-277 1997). Collapse of the World Trade Center towers on September 11th 2001 generated interests to its highest level in progressive collapse (NIST 2005). Due to the increase of catastrophic events in recent years, a surge of research activities on the evaluation and prevention of progressive collapse have been conducted. In these researches, critical gravity load-bearing element was eliminated and then structures were designed in order to mitigate risk (Ruth *et al*. 2006, Marjanishvili and Agnew 2006, Izzuddin *et al*. 2008). To minimize the risk of progressive collapse in buildings, many approaches have been suggested. McKay (2008) conducted a series of nonlinear analyses for steel and RC frame models under various column-loss scenarios to generate regression formulae for load increase and dynamic increase factors. A number of studies have been carried out to investigate the progressive collapse resistance of buildings (Chen *et al*. 2016, Cassiano *et al*. 2016). Some retrofitting and design methods are described by Mirtaheri and Zoghi (2016) to resist progressive collapse.

Relevant standards and design guidelines such as General Services Administration (GSA) (2003) and United States Department of Defense (UFC 4-023-03 2013) are available to design structures that are resistant to progressive collapse. These standards are concerned with quantifiable and significant security methodologies to resist progressive collapse. The alternative path method (APM) is proposed in both mentioned guidelines. APM is one of the most widely accepted methodologies that are applied to assess the potential progressive collapse of building structures by direct removal of a column (UFC 4-023-03 2013). In the following sections, alternate path method, different analysis procedures, and empirical formulae in dynamic analysis are reviewed.

It should be noted that the damping ratio of the structure is one of the significant parameters in dynamic analysis that can change the response of structure against column removal. The only effective parameter in the DIF formulation of the UFC 4-023-03 (2013) guideline and in nonlinear static analysis is θ_{pra}/θ_y. θ_{pra} and θ_y are the plastic rotation angle in acceptance criteria and yield rotation, respectively, that depends only on the material and mechanical properties of the affected structural members. It is easily predictable that having a higher damping ratio in structure reduces vertical displacement and dynamic increase factor (DIF) of a structure against progressive collapse. Therefore, it is necessary to adjust the DIF in a manner that includes damping ratio.

In this paper, in order to investigate the effect of damping ratio on dynamic response and DIF, series of three-dimensional moment frames with three and ten-storey buildings as low and mid-rise buildings with different span lengths are provided. These structures are designed for different seismic ground motion intensities to cover a wide range of structures with varied section members. Towards this aim, a new empirical DIF as a function of damping ratio is presented which can be used for nonlinear static analysis of structures.

2. Alternate path method (APM)

In this method one of the vertical load-bearing elements at the specific location of plan and elevation is removed and the capability of structures to bridge across a removed element is evaluated. In the updated UFC 4-023-03 (2013) three analysis procedures are employed for APM analysis including Linear Static (LS), Nonlinear Static (NLS) and Nonlinear Dynamic (NLD). These procedures adhere the general approach in ASCE 41-13 (2013) with modifications to accommodate the particular issues associated with progressive collapse.

3. Empirical formulas for DIF

Several empirical expressions for structures DIFs with sudden column removal have been proposed in recent years. A series of nonlinear analysis for steel frame models was conducted by Stevens *et al*. (2008) and an empirical DIF formula for steel building frames was expressed as Eq. (1)

$$DIF = 1.44\,\mathrm{m}^{-0.12} \tag{1}$$

Where m is the plastic rotation divided by yield rotation that represents the critical structural performance level of component or connection in the area which is loaded with the amplified gravity load. Plastic and yield rotations are defined in ASCE 41-13 (2013).

McKay *et al*. (2012) presented empirical DIF formulae for steel and concrete moment frames, separately. The empirical DIFs were obtained using a similar procedure for a wide range of steel and concrete frame models under various column removals. Eqs. (2) and (3) represent the empirical DIF formulae for steel and concrete frames, respectively

$$DIF = 1.08 + 0.76/((\theta_{dl}/\theta_{yield}) + 0.83) \tag{2}$$

$$DIF = 1.04 + 0.45/((\theta_{dl}/\theta_{yield}) + 0.48) \tag{3}$$

Where θ_{all} and θ_{yield} are the minimum nonlinear acceptance criteria and yield rotation of members respectively (according to ASCE 41-13 in the region affected by the column removal).

Liu (2013) proposed a new DIF for nonlinear static alternate path analysis. The new DIF was a function of maximum (M_u/M_p) in which M_u and M_p are the factored moment demand under original unamplified static gravity load and the factored plastic moment capacity, respectively. Based on Liu (2013) three DIF formulae were presented for different locations column loss and maximum (M_u/M_p) ranges.

For exterior column removal scenarios with max (M_u/M_p) ≤ 0.5

$$DIF = 1.15\max(M_u/M_p) + 0.12 \tag{4}$$

For interior column removal scenarios with max(M_u/M_p) ≤ 0.5

$$DIF = 0.58\max(M_u/M_p) + 1.55 \tag{5}$$

For both exterior and interior column removal scenarios where max(M_u/M_p) ≥ 0.5

$$DIF = 0.84 + \frac{1.23}{2.95\max(M_u/M_p) - 0.28} \tag{6}$$

4. Analysis procedure

According to General Services Administration (2003) and Department of Defense (2013) in the alternate path method one of the three procedures consist of LS, NLS, or NLD may be implemented in order to find the capability of building to bridge over a removed structural element.

Linear Static (LS): it is the simplest option to apply and is unable to present an accurate prediction of the actual nonlinear dynamic structural behavior. In this procedure the proposed load cases are different for deformation-controlled and force-controlled actions.

Nonlinear Static (NLS): in this procedure material and geometrical nonlinearities are considered in the model with removed vertical load-bearing element. To include both of dynamic effect due to sudden column loss and nonlinearity, the loads in the bays immediately adjacent to the removed element and at all floors above it, are amplified (UFC 4-023-03 2013). The results of ductile members' deformations and brittle members' strengths are compared to expected deformation capacities and maximum internal members' forces, respectively. Expected deformation capacities are exhibited in guidelines and standards.

Nonlinear Dynamic (NLD): this analysis is the most accurate and expensive procedure that is so sensitive to some parameters such as damping ratio, time step, plastic hinge definition and post-elastic stiffness ratio.

One of the most effective parameters in dynamic analysis is damping ratio which can influence the response of structure against column removal, in this paper a wide range of damping ratio is considered in order to investigate the effect of damping ratio in DIF for steel structures.

4.1 Procedure for Determining Dynamic Increase Factors (*DIF*)

As mentioned earlier, NLD is a time consuming and also sophisticated analysis, therefore a study was conducted to investigate needed essential factors to match the NLS procedures to the NLD procedure in the acceptable way. In this study damping ratio as an effective parameter has been considered. Structural deformation is taken into account according to ASCE 41 as the best metric for approximating structural damage (McKay 2008, McKay *et al*. 2012). Series of steel structures with different number of stories and bay lengths are designed to determine the influence of damping ratio variations on DIF. Used load combination is defined as 1.2D + 0.5L where D and L are the dead and live loads, respectively (ASCE/SEI 7-10 2010). Two steps should be conducted to determine the DIFs as follows:

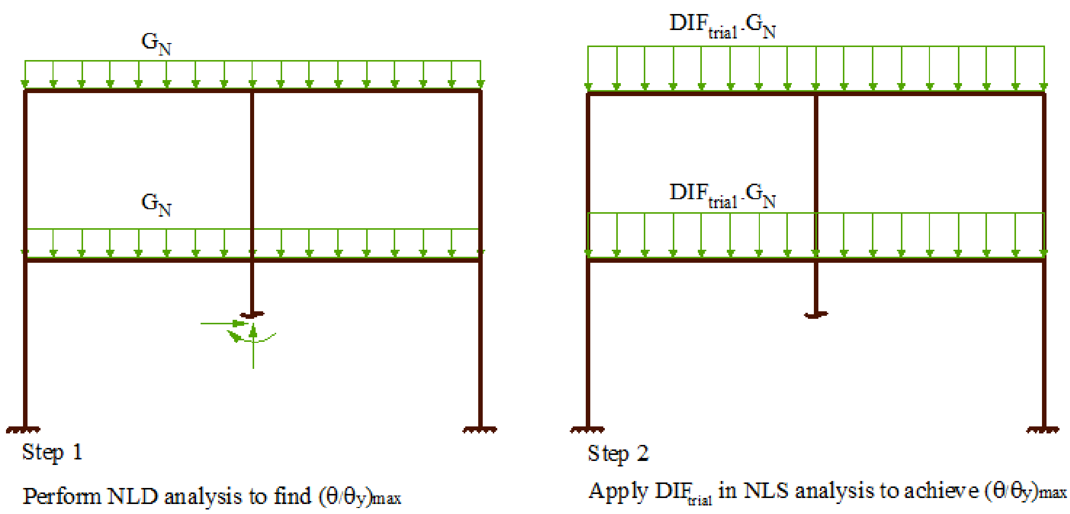

Fig. 1 Illustration of the steps to obtain the data point of DIF

(1) Performing a nonlinear dynamic analysis including ASCE extreme load case without any enhancement to obtain the maximum ratio of θ_p/θ_y among all members of the bays affected by the column removal location. Also, θ_p is the plastic rotation of a member obtained from NLD and θ_y is the yield rotation of the same member.

(2) Performing a NLS analysis with the same model as step 1 and a trial DIF is applied to the ASCE extreme event load case. The maximum ratio of θ_p/θ_y is recorded and compared to the ratio measured in step 1. Then the DIF is modified and the model is re-run until the maximum ratio of θ_p/θ_y matches to the corresponding ratio in dynamic analysis. Fig. 1 illustrates the steps for DIF calculation.

5. Modeling

A series of three and ten-storey buildings with different bay lengths were analyzed and designed in order to assess effective parameters in structures against column removal.

The buildings have different bays of 3, 4.5, 6, and 9 meters and a typical floor plan is illustrated in Fig. 2. The gravity loads are summarized in Table 1. The members' sizes of the buildings with different span lengths are presented in Tables A1-A7 in Appendix A.

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

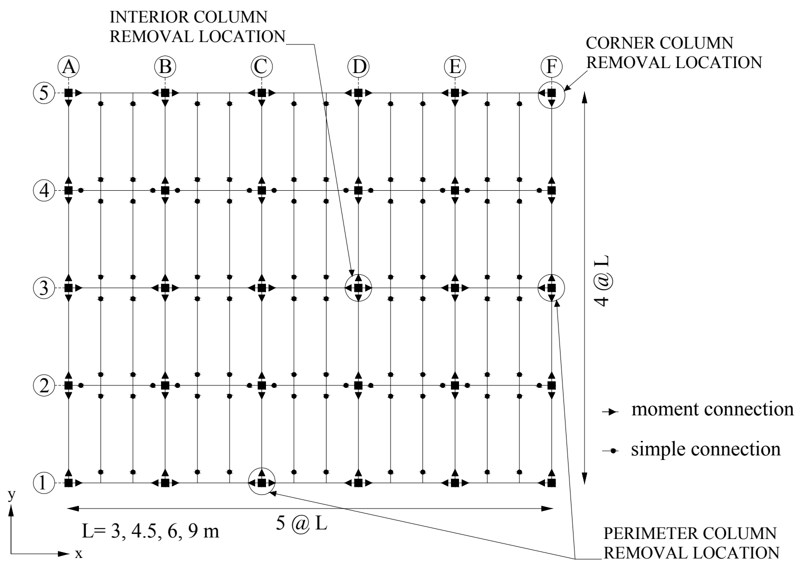

Fig. 2 Typical floor and column removal locations

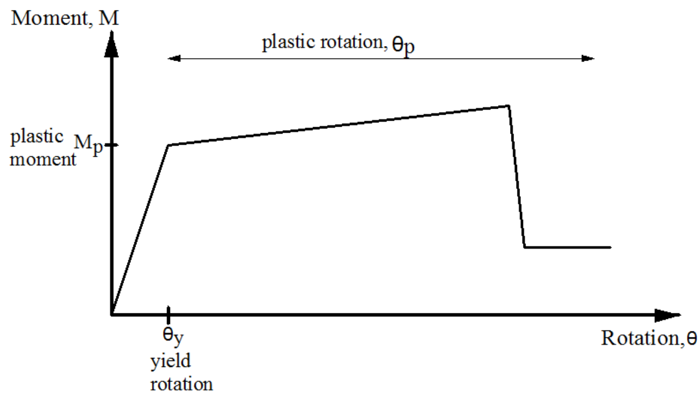

Fig. 3 Moment-Rotation curve and definition of yield and plastic rotations

According to UFC4-023-03, the gravity load combination of 1.2D + (0.5L or 0.2S) should be used. To calculate the plastic moment capacity of steel beams, a strength reduction factor of 0.9 was used. The yield and ultimate strengths of all steel members in the 3D models were considered as 235.36 and 362.85 MPa, respectively. The modulus of elasticity and over strength factor used in the analysis were taken as 200000 MPa and 1.05, respectively. Both geometric and material nonlinearities (by modeling concentrated plastic hinges) are considered for the beams and columns.

The SAP2000 program (2011) is used as computational tool to perform several APM analyses on a variety of steel moment frame buildings in order to investigate the load redistribution behavior in the structure after column loss. The concentrate hinge properties are determined according to according to ASCE 41-13 (2013). To define the acceptance criteria and modeling parameters, multiples of the yield rotation was used based on ASCE 41. The used moment-rotation diagram for beam and column are shown schematically in Fig. 3 where θ_P is the plastic rotation of the beam or column and θ_y is the rotation at yield that is calculated from ASCE 41-13 (2013). For post-yield of steel members, a strain hardening slope equal to 3% of the elastic slope is considered. For these analyses, the damping ratios were set from 1 to 15% (1, 5, 10 and 15%). The column removal time and time step are taken as 1/20 and1/200 of the vertical natural period, respectively (McKay *et al*. 2012).

5.1 Location of column removal

According to UFC4-023-03 (2013) columns or walls should be removed at the internal and external of the building plan so in this study columns at interior, corner, and perimeter of the plan are removed and analyses are carried out. Due to symmetry in the plan, only four columns in first storey of each structure are chosen and removed which is shown in Fig. 2. Under each column removal scenario, nonlinear dynamic and static analyses are carried out according to the procedures described in steps 1 and 2 of Section 4.1 to obtain the maximum ratio of θ_p/θ_y in the structure.

6. Results and discussion

Results from the analyses are provided to show the parameters that affect DIF. These results demonstrate that DIF depends on both ratios of plastic and yield rotations and also the damping ratio of structure. Eq. (7) is suggested by UFC4-023-03 (2013) to calculate DIF for steel structures that depends the ratio of plastic and yield rotation only.

$$DIF = 1.08 + \frac{0.76}{0.83 + (\theta_p / \theta_y)} \tag{7}$$

To assess the effect of damping ratio on DIF, some parts of the obtained results from analysis are summarized in Table 2. In this table, the DIFs obtained from UFC4-023-03 (2013) and also directly concluded from analyses with different damping ratios for two specific ratios of θ_p/θ_y are presented.

According to this table, it is obvious that for a particular ratio of θ_p/θ_y, by increasing damping ratio in the structures, the DIF as recommended by the UFC is constant but the DIFs obtained from direct analyses are variable. The last column of Table 2 shows differences between DIFs which become more by increasing damping ratio and DIF decreases by increasing damping ratio in structures. Therefore, the DIF formula which was suggested in UFC gives conservative values for more than 1% damping ratio of steel structures.

The obtained data points of final DIF vs. θ_p/θ_y for all columns removal of each assumed damping ratio are plotted in Figs. 4-7. The symbol for each data point denotes the specific location of column removal and the structures total number of storeys. These figures show the data points for three and ten-storey buildings. It can be seen that the number of storey is not an effective parameter in DIF however, the most effective parameter is the ratio of θ_p/θ_y and damping ratio.

Curve fitting is carried out to drive an equation for each damping ratio. A general form of equation is considered to evaluate the effect of damping ratio on DIF. The considered general equation to calculate DIF is as follows

$$DIF = A - \frac{B \times (\theta_p / \theta_y)}{C + (\theta_p / \theta_y)} \tag{8}$$

In this equation there are three parameters which relate to damping ratio. The A parameter is obtained from elastic single degree of freedom system with different damping ratios. To establish

Table 2 Comparison of UFC suggested and direct analyses DIFs with different damping ratios

θ_p / θ_y	DIF (UFC)	Damping ratio (%)	DI (Direct Analyses)	$\frac{DIF_{UFC} - DIF_{ANALYSIS}}{DIF_{UFC}} \times 100$
≈3.7	1.24	1	1.30	-4.8
		5	1.24	0.0
		10	1.13	8.9
		15	1.07	13.7
≈2	1.35	1	1.4	-3.7
		5	1.25	7.4
		10	1.19	11.85
		15	1.13	16.3

Table 3 Obtained parameters in suggested equation

Parameter	
A	$2 - 2.54 \times \zeta$
B	$0.9 - 1.81 \times \zeta$
C	$0.84 - 2.15 \times \zeta$

the B and C parameters, the data point of these parameters are plotted against damping ratio. A linear fit of data points was performed to drive the equation for each of the B and C parameters. The A, B and C parameters are presented in Table 3 where ζ is the damping ratio.

Fig. 4 plots the data point for structures with 1% damping ratio and compares the fitted curve with the suggested curve of UFC design guide.

According to Fig. 4, the obtained DIF equation from curve fitting is as follows

$$DIF = 1.09 + \frac{0.74}{0.83 + (\theta_p / \theta_y)} \tag{9}$$

By comparing Eq. (7) with the Eq. (9) which suggested by UFC design guide, it is observable that these two equations are almost identical when damping ratio of structure is 1%.

As shown in Figs. 4-7, by increasing damping ratio, the difference between the recommended and UFC suggest DIF increases. It is noticed that for a specific ratio of θ_p/θ_y the DIF which suggested that UFC for more than 1% damping ratio is conservative.

Fig. 7 depicts the data points of DIF vs. θ_p/θ_y for all column removal of structure with 15% damping ratio. It is observable from the figure that by increasing the ratio of θ_p/θ_y, the effect of damping ratio on DIF decreases. For example, for $\theta_p/\theta_y = 1.0$, the calculated ratio of recommended and UFC DIF is 0.8 while this ratio for $\theta_p/\theta_y = 6.0$ is 0.87. It means that damping ratio is more

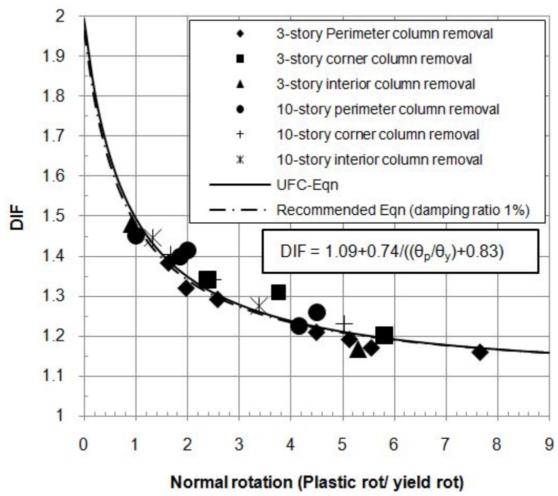

Fig. 4 Dynamic increase factor as a function of normal rotation with 1% damping ratio

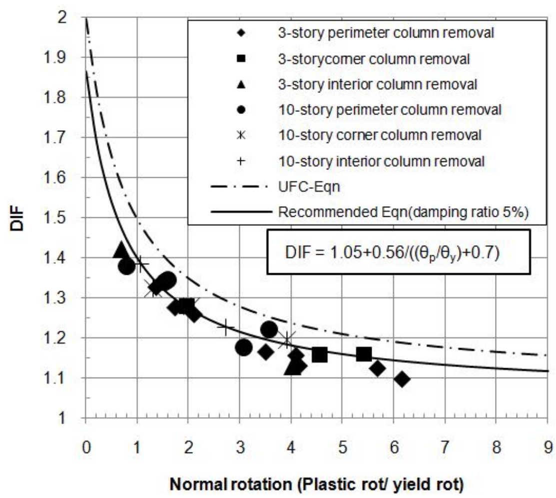

Fig. 5 Dynamic increase factor as a function of normal rotation with 5% damping ratio

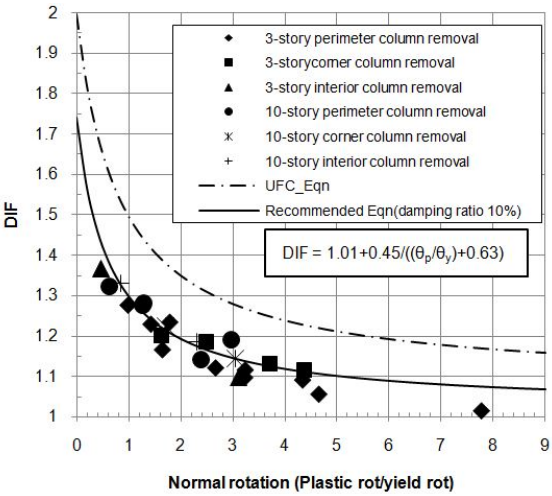

Fig. 6 Dynamic increase factor as a function of normal rotation with 10% damping ratio

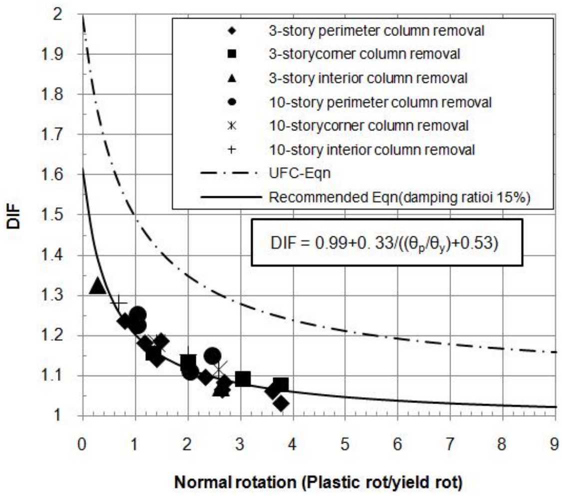

Fig. 7 Dynamic increase factor as a function of normal rotation with 15% damping ratio

effective on structures with low range of ductility to reduce the DIF.

It should be noted that in this study, a DIF in nonlinear static analysis is used to match the maximum ratio of θ_p/θ_y to a corresponding ratio in nonlinear dynamic analysis. In this procedure, the gravity loads in affected bays upon column removal is uniformly amplified. It is possible that this load pattern may not be the best method of amplifying the loads.

7. Conclusions

One of the most accurate procedures to assess the progressive collapse potential of a structure is nonlinear dynamic analysis which is so sensitive to damping ratio. In this study the effect of damping ratio on dynamic increase factor (DIF) in a steel moment frame building is investigated. For this purpose, a series of moment frame structures with different span lengths and number of storeys are designed. Damping ratios of 1, 5, 10, and 15% are considered to assess the effect of this parameter on DIF. At first the analyses are carried out according to the described procedure by considering 1% damping ratio to verify the method and in the following the step by step analyses conducted with damping ratio of 5, 10, and 15%. It is found that by increasing damping ratio, the ratio of θ_p/θ_y decreased, thus the progressive collapse strength of the structure increased. It is observed that by increasing damping ratio in a structure, the suggested DIF in UFC design guideline for use in nonlinear static analysis was more conservative than the nonlinear dynamic analysis with damping ratio of more than 1%. Therefore, an empirical formula is recommended for different damping ratios that can be used in nonlinear static analysis. The results indicate that the effect of damping ratio is related to structural ductility. This means that the reduction rate of the DIF in structures with low range of ductility is more than in structures with a high range of ductility.

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Appendix – A

To consider a wide type of sections and configurations of the structure, different member sections were used in different frames. Each 3-D building includes five and six frames in the *x* and *y* directions, respectively. In all buildings, just two frames were simple frame while others were moment resistant frames. All the beams and columns that were mentioned in the specified frames were the same in each storey. The members' sizes for the three and ten-storey buildings with different span lengths are presented in Tables A1-A7.

Table A1 Beams and columns sections of 10-storey building with 9 meters span lengths

Axe	1 , 5 (x-z) Moment frame		2 , 4 (x-z) Simple frame		3 (x-z) Moment frame		A , F (y-z) Moment frame	B, C, D, E, (y-z) Moment frame
Storey	Beam	Column	Beam	Column	Beam	Column	Beam	Beam
1	IPE 600	TUBE 400×400×28	IPE 550	TUBE 380×380×40	IPE 750×137	TUBE 400×400×40	IPE 500	IPE 500
2	IPE 600	TUBE 400×400×25	IPE 550	TUBE 380×380×30	IPE 750×137	TUBE 400×400×35	IPE 500	IPE 500
3	IPE 600	TUBE 400×400×25	IPE 550	TUBE 380×380×30	IPE 750×137	TUBE 400×400×35	IPE 500	IPE 500
4	IPE 600	TUBE 400×400×20	IPE 550	TUBE 380×380×20	IPE 750×137	TUBE 400×400×28	IPE 500	IPE 500
5	IPE 600	TUBE 400×400×20	IPE 550	TUBE 380×380×20	IPE 750×137	TUBE 400×400×28	IPE 500	IPE 450
6	IPE 550	TUBE 380×380×20	IPE 550	TUBE 320×320×20	IPE 750×137	TUBE 400×400×28	IPE 500	IPE 450
7	IPE 550	TUBE 380×380×20	IPE 550	TUBE 320×320×20	IPE 600	TUBE 360×360×22.2	IPE 450	IPE 450
8	IPE 550	TUBE 300×300×16	IPE 550	TUBE 320×320×20	IPE 600	TUBE 360×360×22.2	IPE 450	IPE 400
9	IPE 550	TUBE 300×300×16	IPE 550	TUBE 280×280×16	IPE 600	TUBE 360×360×22.2	IPE 450	IPE 400
10	IPE 400	TUBE 300×300×16	IPE 450	TUBE 280×280×16	IPE 500	TUBE 300×300×16	IPE 360	IPE 360

Table A2 Beams and columns sections of 10-storey building with 6 meters span lengths

Axe	1 , 5 (x-z) Moment frame		2 , 4 (x-z) Simple frame		3 (x-z) Moment frame		A , B , C, D, E, F (y-z) Moment frame
Storey	Beam	Column	Beam	Column	Beam	Column	Beam
1	IPE 450	TUBE 340×340×25	IPE 300	HE450B	IPE 500	TUBE 340×340×28	IPE 400

Table A2 Continued

Axe	1 , 5 (x-z) Moment frame		2 , 4 (x-z) Simple frame		3 (x-z) Moment frame		A , B , C, D, E, F (y-z) Moment frame
Storey	Beam	Column	Beam	Column	Beam	Column	Beam
2	IPE 450	TUBE 340×340×25	IPE 300	HE450B	IPE 500	TUBE 340×340×25	IPE 400
3	IPE 450	TUBE 340×340×20	IPE 300	HE450B	IPE 500	TUBE 340×340×25	IPE 400
4	IPE 450	TUBE 340×340×20	IPE 300	HE450B	IPE 500	TUBE 340×340×25	IPE 360
5	IPE 450	TUBE 340×340×20	IPE 300	HE360B	IPE 500	TUBE 300×300×20	IPE 360
6	IPE 400	TUBE 300×300×16	IPE 300	HE360B	IPE 500	TUBE 300×300×20	IPE 360
7	IPE 400	TUBE 300×300×16	IPE 300	HE360B	IPE 450	TUBE 300×300×16	IPE 330
8	IPE 400	TUBE 300×300×16	IPE 300	HE300B	IPE 400	TUBE 280×280×14.5	IPE 330
9	IPE 360	TUBE 300×300×16	IPE 300	HE300B	IPE 400	TUBE 280×280×14.5	IPE 300
10	IPE 300	TUBE 300×300×16	IPE 270	HE300B	IPE 360	TUBE 280×280×14.5	IPE 270

Table A3 Beams and columns sections of 10-storey building with 4.5 meters span lengths

Axe	1 , 5 (x-z) Moment frame		2 , 4 (x-z) Simple frame		3 (x-z) Moment frame		A , F (y-z) Moment frame	B, C, D, E, (y-z) Moment frame
Storey	Beam	Column	Beam	Column	Beam	Column	Beam	Beam
1	IPE 360	TUBE 300×300×16	IPE 240	HE300B	IPE 400	TUBE 300×300×20	IPE 300	IPE 300
2	IPE 360	TUBE 300×300×16	IPE 240	HE300B	IPE 400	TUBE 300×300×16	IPE 300	IPE 300
3	IPE 360	TUBE 300×300×16	IPE 240	HE300B	IPE 400	TUBE 300×300×16	IPE 300	IPE 300
4	IPE 360	TUBE 280×280×14.2	IPE 240	HE280B	IPE 360	TUBE 280×280×14.2	IPE 300	IPE 300
5	IPE 360	TUBE 280×280×14.2	IPE 240	HE280B	IPE 360	TUBE 280×280×14.2	IPE 270	IPE 300
6	IPE 330	TUBE 280×280×14.2	IPE 240	HE280B	IPE 360	TUBE 280×280×14.2	IPE 270	IPE 300
7	IPE 300	TUBE 240×240×12.5	IPE 240	HE240B	IPE 330	TUBE 240×240×12.5	IPE 240	IPE 270

Table A3 Continued

Axe	1 , 5 (x-z) Moment frame		2 , 4 (x-z) Simple frame		3 (x-z) Moment frame		A , F (y-z) Moment frame	B, C, D, E, (y-z) Moment frame
Storey	Beam	Column	Beam	Column	Beam	Column	Beam	Beam
8	IPE 300	TUBE 240×240×12.5	IPE 240	HE240B	IPE 300	TUBE 240×240×12.5	IPE 240	IPE 240
9	IPE 300	TUBE 240×240×12.5	IPE 240	HE240B	IPE 300	TUBE 240×240×12.5	IPE 240	IPE 240
10	IPE 200	TUBE 240×240×12.5	IPE 240	HE240B	IPE 240	TUBE 240×240×12.5	IPE 200	IPE 200

Table A4 Beams and columns sections of 3-storey building1 with 6 meters span lengths

Axe	1 , 5 (x-z) Moment frame		2 , 4 (x-z) Simple frame		3 (x-z) Moment frame		A , F (y-z) Moment frame	B, C, D, E, (y-z) Moment frame
Storey	Beam	Column	Beam	Column	Beam	Column	Beam	Beam
1	IPE 330	TUBE 180×180×14.2	IPE 300	HE260B	IPE 360	TUBE 180×180×14.2	IPE 330	IPE 300
2	IPE 330	TUBE 180×180×14.2	IPE 300	HE260B	IPE 360	TUBE 180×180×14.2	IPE 300	IPE 270
3	IPE 300	TUBE 180×180×10	IPE 300	HE260B	IPE 360	TUBE 180×180×10	IPE 240	IPE 240

Table A5 Beams and columns sections of 3-storey building2, with 6 meters span lengths

Axe	1 , 5 (x-z) Moment frame		2 , 4 (x-z) Simple frame		3 (x-z) Moment frame		A , F (y-z) Moment frame	B, C, D, E, (y-z) Moment frame
Storey	Beam	Column	Beam	Column	Beam	Column	Beam	Beam
1	IPE 330	TUBE 180×180×14.2	IPE 300	HE260B	IPE 330	TUBE 180×180×14.2	IPE 330	IPE 270
2	IPE 330	TUBE 180×180×14.2	IPE 300	HE260B	IPE 330	TUBE 180×180×14.2	IPE 270	IPE 270
3	IPE 300	TUBE 180×180×10	IPE 300	HE180B	IPE 300	TUBE 180×180×10	IPE 270	IPE 240

Table A6 Beams and columns sections of 3-storey building with 4.5 meters span lengths

Axe	1 , 5 (x-z) Moment frame		2 , 4 (x-z) Simple frame		3 (x-z) Moment frame		A , F (y-z) Moment frame	B, C, D, E, (y-z) Moment frame
Storey	Beam	Column	Beam	Column	Beam	Column	Beam	Beam
1	IPE 270	TUBE 180×180×12.5	IPE 240	HE220B	IPE 270	TUBE 180×180×12.5	IPE 270	IPE 240
2	IPE 270	TUBE 180×180×12.5	IPE 240	HE220B	IPE 270	TUBE 180×180×12.5	IPE 240	IPE 220
3	IPE 200	TUBE 180×180×10	IPE 240	HE180B	IPE 220	TUBE 180×180×10	IPE 200	IPE 200

Table A7 Beams and columns sections of 3-storey building with 3 meters span lengths

Axe	1 , 5 (x-z) Moment frame		2 , 4 (x-z) Simple frame		3 (x-z) Moment frame		A , B, C, D, E, F (y-z) Moment frame
Storey	Beam	Column	Beam	Column	Beam	Column	Beam
1	IPE 200	TUBE 160×160×12.5	IPE 160	HE220B	IPE 200	TUBE 160×160×12.5	IPE 180
2	IPE 200	TUBE 160×160×12.5	IPE 160	HE220B	IPE 200	TUBE 160×160×12.5	IPE 180
3	IPE 160	TUBE 160×160×10	IPE 160	HE180B	IPE 160	TUBE 160×160×10	IPE 180