



Effect of retrofit strategies on mitigating progressive collapse of steel frame structures

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ABSTRACT

In this study, the effect of three retrofit strategies on enhancing the response of existing steel moment resisting frames designed for gravity loads is investigated using Alternate Path Methods (APM) recommended in the General Services Administration (GSA) and the Department of Defense (DoD) guidelines for resisting progressive collapse. The response is evaluated using 3-D nonlinear dynamic analysis. The studied models represent 6-bay by 3-bay 18-storey steel frames that are damaged by being subjected to six scenarios of sudden removal of one column in the ground floor. Four buildings with bay spans of 5.0 m, 6.0 m, 7.5 m, and 9.0 m were studied. The response of the damaged frames is evaluated when retrofitted using three approaches, namely, increasing the strength of the beams, increasing the stiffness of the beams, and increasing both strength and stiffness of the beams.

The objective of this paper is to assess effectiveness of the studied retrofit strategies by evaluating the enhancement in three performance indicators which are chord rotation, tie forces, and displacement ductility demand for the beams of the studied building after being retrofitted.

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1. Introduction

In the past decades, there have been cases where buildings around the world have experienced partial or total progressive collapse under extreme abnormal loading conditions. In the “*Best practice for reducing the potential for progressive collapse in buildings*” published by NIST [1], the potential abnormal load hazards that can trigger progressive collapse are categorized as: aircraft impact, design/construction error, fire, gas explosions, accidental overload, hazardous materials, vehicular collision, bomb explosions, etc. As these hazards could be considered to have low probability of occurrence for structures of normal importance, thus they are either not considered in structural design or addressed indirectly by passive protective measures, yet they are seen to be important to be considered for important and susceptible structures. Most of these hazards have characteristics of acting over a relatively short period of time and result in dynamic responses. Despite the probability of the hazard occurrence, progressive collapse of a building has significant socio-economic impacts.

In progressive collapse, an initial localized damage or local failure spreads through neighbouring elements, possibly resulting in the failure of the entire structural system. The most viable approach to limiting this propagation of localized damage is to

maintain the integrity and ductility of the structural system. The ASCE 7-05 commentary [2] suggests a general design guidance for improving the progressive collapse resistance of structures, but it does not provide any specific implementation rules. Recent design procedures to mitigate the potential for progressive collapse in structures can be found in two design guidelines issued by the U.S. which are the General Service Administration (GSA) [3] and the Department of Defense (DoD) [4].

In a recent investigation, Kim and Kim [5] studied the response of steel moment resisting frames using alternative load path with different damage scenarios when a corner, a first edge and internal edge column are removed. Applying static, nonlinear static and dynamic analyses, they found that nonlinear dynamic analysis is the most precise, yet the results varied more significantly depending on the variables such as applied load, location of column removal, or number of building story. Also, they found that the potential for progressive collapse was highest when a corner column was suddenly removed, and that the potential for progressive collapse decreases as the number of storey increases. Fu [6] assessed the response of a 20-storey building subjected to sudden loss of a column for different structural systems and different scenarios of column removal. One of his concluded results is that under the same general conditions, a column removal at a higher level will induce larger vertical displacement than a column removal at ground level. Also, the researcher concluded that the dynamic response of the structure is mainly related to the affected loading area after the column removal.

GSA [3] and DoD [4] guidelines recommended the use of the direct approach or the Alternate Path Method (APM). In this

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Notations

GSA	General Service Administration
UFC	Unified Facilities Criteria (DOD 2005)
ESC	Edge Short Column
CC	Corner Column
IC	Internal Column
FIC	First Internal Column
ELC	Edge Long Column
FELC	First Edge Long Column
E	Modulus of elasticity
F_y	Yield strength of steel
K_0	Initial stiffness of the beam
M_p	Plastic moment of the beam
I	Moment of inertia of the beam
$\theta_{upgr...s}$	Upgraded chord rotation after increasing the strength only
$\theta_{upgr...s,k}$	Upgraded chord rotation after increasing the strength and stiffness
TF	Tie Force in the beam
μ_Δ	Displacement ductility demand of the beam
R_s^θ	Reduction factor in chord rotation due to increase in strength
R_k^θ	Reduction factor in chord rotation due to increase in stiffness
R_s^T	Reduction factor in tie force due to increase in strength
R_k^T	Reduction factor in tie force due to increase in stiffness
R_s^μ	Reduction factor in displacement ductility demand due to increase in strength
R_k^μ	Reduction factor in displacement ductility demand due to increase in stiffness
α_s	Strength factor due to increase in strength
α_k	Stiffness factor due to increase in stiffness
$\alpha_{s,k}$	Upgrading factor due to increase in strength or stiffness

method, a single column in the ground level is typically assumed to be suddenly missing, and an analysis is conducted to determine the ability of the damaged structure to bridge across the missing column. The APM is mainly concerned with the vertical deflection or the chord rotation of the building after the sudden removal of a column. The chord rotation is equal to the vertical deflection at the location of the removed column divided by the adjacent beams' span. As such, it is a threat-independent design-oriented method for introducing further redundancy into the structure to resist propagation of collapse.

Existing buildings that were designed for gravity loads or designed according to earlier codes are expected to have inadequate resistance to progressive collapse. Steel frame structures designed to earlier codes did not behave well during extreme hazard event due to insufficient carrying capacity [7]. One of the major challenges for a structural engineer is choosing a retrofit scheme for an existing steel structure with a potential for progressive collapse. Another challenge is deciding on the level of protection against such potential event of sudden loss of a supporting column. It is not a normal practice in retrofitting to attempt to make the existing structure comply with the present code provisions, as this approach may not be economic. Alternatively, it is proposed that the retrofit objectives for a structure that is susceptible to progressive collapse should rather depend on a performance-based criterion to ensure a predefined level of damage or to prevent collapse of the building. This approach is similar to the Performance-Based Seismic Design (PBSD) recently adopted by several guides [8,9].

The retrofit strategy may involve targeted repair of deficient members, providing systems to increase stiffness and strength or providing redundant load carrying systems by a structure system such as mega truss or vierendeel trusses at the top of the building or by using bracing systems that redistribute the loads through the entire structure. In general, a combination of different strategies may be used in the retrofitting of the structure.

2. Problem definition

The ductility of steel alone cannot guaranty that the steel building will not collapse under extreme loading. Progressive failure in steel buildings occurs due to insufficient strength in the beams that are needed to bridge the load from the removed column location to the adjacent columns. Upon column removal, the vertical load is transferred to the adjacent columns, where the resulting increase in the axial load of these columns is relatively small. On the other hand, the loss of a column will result in a significant increase in the flexure and shear demand on the adjacent beams. As such, upgrading the beams by increasing their strength and/or stiffness is expected to reduce the progressive collapse of steel buildings. In case of high hazard event where more than one column is expected to be lost, upgrading both beams and columns might be needed.

The objective of this paper is to assess the effectiveness of three different retrofit strategies for beams on the dynamic response of an existing high-rise steel structure when subjected to six damage scenarios by sudden removal of one of the columns at the ground level. The three studied retrofit schemes are by increasing the strength, stiffness, and both strength and stiffness of the beams. The effectiveness of the retrofit methods of damaged buildings is evaluated by comparing three performance indicator parameters, namely, chord rotation, tie forces, and displacement ductility demand of the beams after being upgraded to those of the original existing structure. Two sets of analyses are conducted. First set is conducted on a building with bay span of 6.0 m in order to evaluate the reduction factors in the three performance indicator parameters due to the three studied retrofit strategies. Second set is conducted on three buildings with spans of 5.0 m, 7.5 m, and 9.0 m in order to assess the effect of variation of bay span.

3. Details of the analytical models

Four 3-D models of 18-storey high-rise steel moment resisting frame buildings having 3×6 bays in plan were constructed using Extreme Loading for Structures (ELS) software [10]. The buildings have the same plan throughout the whole height. For each building, the sizes of the columns were kept constant for every three stories along the height; whereas two sizes for the beams were designed and kept constant for the whole height, namely, perimeter beams and internal beams. The studied models have bay spans of 5.0 m, 6.0 m, 7.5 m, and 9.0 m in the two directions. The buildings were designed according to CISC-95 [11] for gravity loading condition. Figs. 1 and 2 show the elevation and plan of the studied buildings, respectively, along with their respective column and beam sizes.

The frame columns and beams were designed to carry a slab thickness of 200 mm. The floors are subjected to a live load of 2.4 kPa, representing a load of an office building, and a superimposed dead load of 2 kPa was taken into account for the equivalent load from interior partition, mechanical and plumbing loads. In the model, a bilinear stress–strain relationship of the steel members was taken, with $F_y = 350$ MPa, and strain hardening of 1% as shown in Fig. 3. Modulus of elasticity, shear modulus, and Poisson's ratio for steel were taken as 200 GPa, 81.5 GPa, and 0.2, respectively. In the model, the inherent damping due to yielding of steel was taken into account as stated in the technical manual of ELS [10], whereas the external damping was neglected.

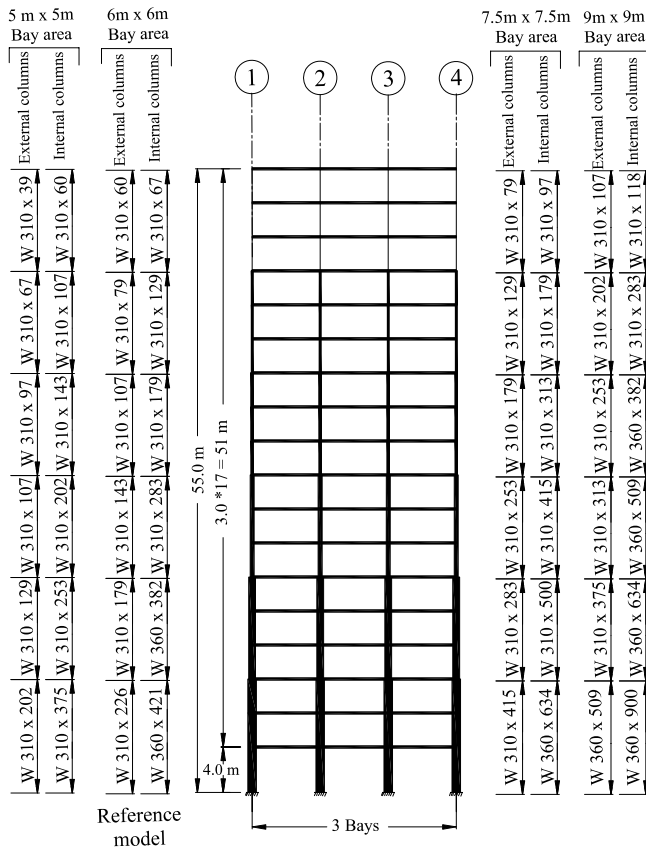


Fig. 1. Elevation of the studied buildings and column sizes.

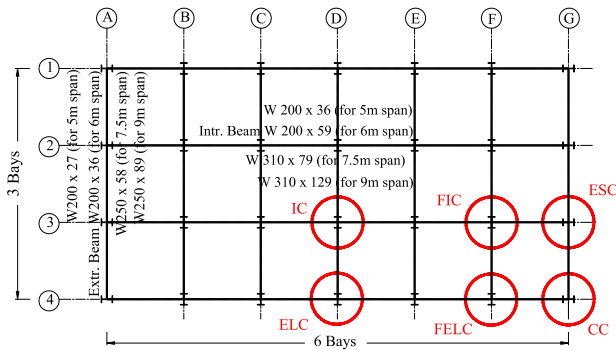


Fig. 2. Plan of the studied buildings, beam sizes and the six studied column removals.

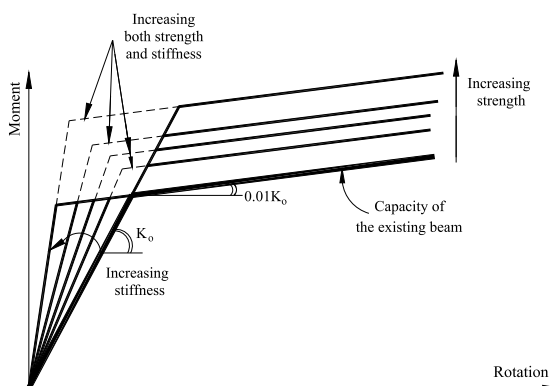


Fig. 3. Methods of upgrading the structure by increasing strength and/or stiffness of beams.

In the analytical models, the following assumptions were used:

(1) Loads from concrete slabs are applied directly on the beams according to area method without representing the slab in the analytical model; (2) Connections between the beam and the column maintains continuity; (3) Support conditions at foundation is considered to be fixed; and (4) Increase of yield strength arising from the high rate of straining due to sudden removal of column is neglected. Fig. 4(a) shows an illustration of the 3-D model used in the nonlinear dynamic analysis of the studied structures using ELS [10] software and Fig. 4(b) shows the different components of the studied model in ELS at a location of the removed column.

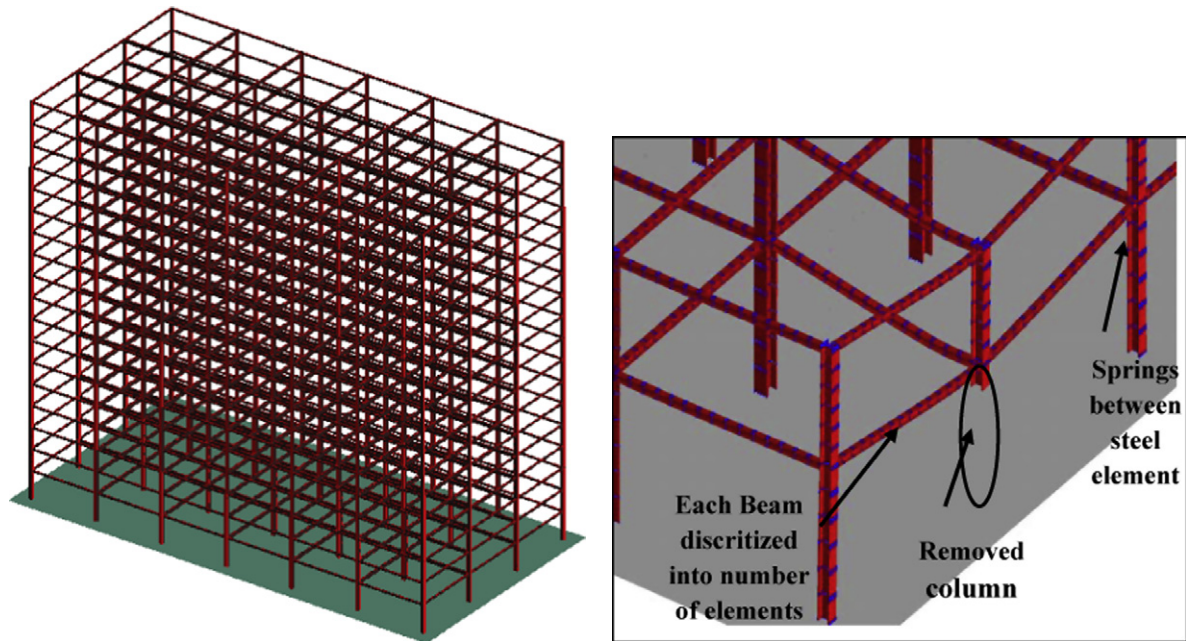
ELS software [10] uses the Applied Element Method (AEM) which is capable of predicting the discrete behaviour of the structure to higher degree of accuracy. AEM is capable of carrying out static and dynamic analyses. AEM has relative advantage to Finite Element Method (FEM) that the elements are capable of separation thus can simulate the real collapse of the structure, whereas the FEM does not possess such characteristic due to the continuity between elements where no separation can occur which lead to singularity in its geometric matrix. In ELS program, failure of the structure occurs in case of element separation or crushing. Element separation or crushing occurs when the springs connecting the elements reach a strain value of 0.1. The ELS nonlinear solver is capable of analyzing the structural behaviour during elastic and inelastic modes including the automatic detection and generation of plastic hinges, buckling, cracks, and collapse. Resulting debris and its impact on structural elements is automatically analyzed and calculated.

In the AEM method, the structural members (beams and columns) are discretized into small rigid elements that are connected through contact points on their surfaces. Each contact point has three springs, one normal and two shears. The stiffness of each spring depends on the area it serves. Each rigid element contains 6 degrees of freedom (3 rotations and 3 translations). The stiffness matrix components corresponding to each degree of freedom are determined by assuming a unit displacement in the studied direction and by determining forces at the centroid of each element. The stiffness matrix of the springs connected to the surface of each rigid element is calculated by summing up all the stiffnesses produced by all springs of that element. Finally, the assembly of all discretized elements' stiffnesses in the structure results in the global stiffness matrix of the entire structure (detailed information are available in [10,12,13]).

4. Method of analysis

Recent advancements in the analysis of progressive collapse of structures adopted Performance-Based Design Method (PBDM) as a practical way that depends on objective criteria. For steel frame buildings with rigid connection, the chord rotation of beam after removal of a column was defined as an important criterion that addresses PBDM. The DoD states that for High Level of Protection (HLOP) and Medium Level of Protection (MLOP) against progressive collapse, the limit for chord rotation is 6-degrees, whereas this limit increases to 12-degrees for Low Level of Protection (LLOP) and Very Low Level of Protection (VLLOP).

Six cases of column removal at ground level are studied as shown in Fig. 2. For each case, the effect of three retrofitting strategies on the chord rotation (θ), Tie Forces (TF), and displacement ductility demand of the beams (μ_Δ) are evaluated. Fig. 3 shows a schematic of the moment curvature relation of the beams when rehabilitated using the three studied retrofit strategies. A retrofit strategy using Fibre-Reinforced Polymer (FRP) composites to strengthen the existing beam is expected to contribute to the strength, without significant contribution to the stiffness of the beam. A retrofit strategy that strengthens an existing beam using



(a) Illustration of the 3-D model used in the nonlinear dynamic analysis of the studied structures using ELS.

(b) Zoom-in of part of the model showing its different components.

Fig. 4. Snapshots for the studied model from the ELS [10] software.

additional continuous steel plates will increase both strength and stiffness of the beam. On the other hand, strengthening a beam using intermittent steel plates will result in an increase in the stiffness without altering the strength of the beam. In the present analyses, the effect of increasing the strength and/or stiffness up to a level of 4 times that of the original beam was considered. In this study, an upgrading factor, α , that represents the increase in strength, α_s , or stiffness, α_k , or both, $\alpha_{s,k}$, of the retrofitted beam is introduced. The assessment of the performance of retrofitted beams was evaluated at upgrading factors of 1.1, 1.25, 1.5, 2 and 4 which correspond to an increase in strength or stiffness of 10%, 25%, 50%, 100% and 300% from the original model, respectively.

In the analyses, the increase of strength was conducted by changing the yield strength (F_y) using the factor α_s , which leads to increase of strength or the capacity of the section in proportion, where the capacity of the section is $M_p = Z_x \cdot F_y$, where Z_x is the section modulus. On the other hand, increasing the stiffness of the beam using the upgrading factor α_k was achieved by increasing both modulus of elasticity (E) and shear modulus (G), which will lead to an increase in the stiffness of the beam. Finally, increase of both strength and stiffness was conducted by increasing the thickness of flanges that increase both strength (plastic moment) and stiffness (moment of inertia), proportionally.

In the conducted nonlinear dynamic analyses, two load combinations to represent the gravity load are used. The first load combination is (1.0 D.L + 0.25 L.L) which follows the GSA [2] guideline, while the second is (1.25 D.L + 0.5 L.L) according to the DoD [3] guideline, where D.L and L.L are the dead load and live load applied on the structure, respectively. These two load combinations were applied in each scenario of removing a column.

5. Results and discussion

This section describes the findings of the analyses of the modeled buildings. In Section 5.1, the results of the 6.0 m × 6.0 m (designated as reference model) are shown, whereas Section 5.2 illustrates the effect of changing the bay size on the response. Figs. 5a and 5b show two flow charts of the nonlinear dynamic

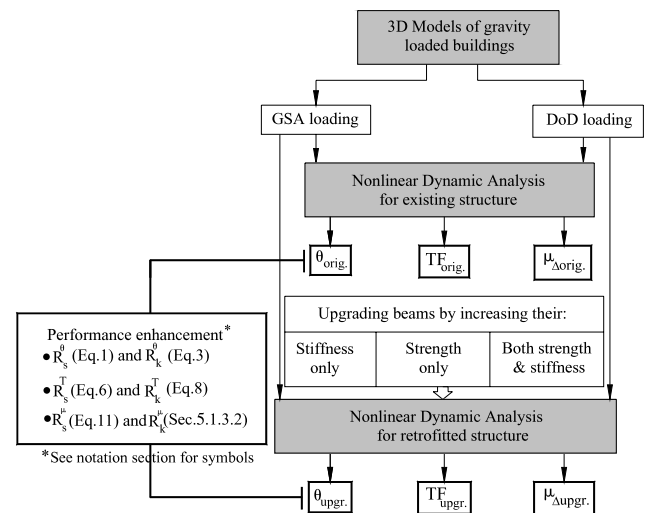


Fig. 5a. Flow chart of the nonlinear dynamic analysis for the reference model to evaluate the effect of three retrofit strategies on three performance indicators (θ , TF , and μ_{Δ}).

analyses, (a) for reference model and (b) for the effect of variation in bay span, to evaluate the effect of three retrofit strategies on three performance indicators (θ , TF , and μ_{Δ}) for the studied buildings.

5.1. Results of reference model

5.1.1. Effect of retrofit strategy on chord rotation (θ)

As defined by the DoD and GSA the chord rotation, θ , is equal to the deflection under the removed column divided by the adjacent span; therefore, the chord rotation can be calculated from the deflection under the removed column.

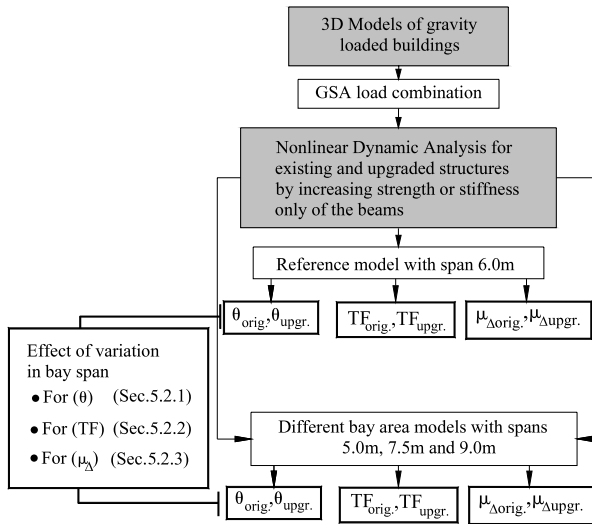


Fig. 5b. Flow chart of the nonlinear dynamic analysis to evaluate the effect of variation in bay span on the three performance indicators (θ , TF, and μ_{Δ}).

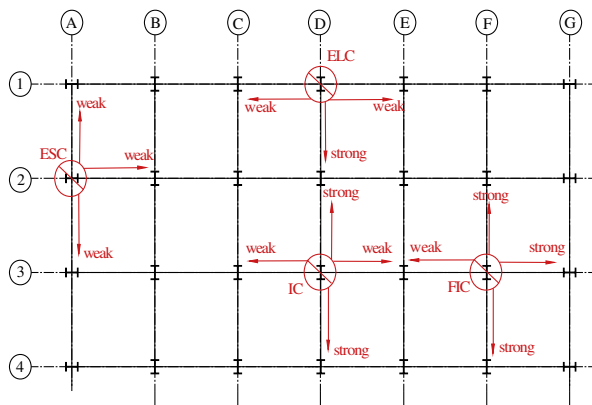


Fig. 6. Illustration of strong and weak connections for the cases of removal of Edge Short (ESC), Edge Long (ELC), Internal (IC) and First Internal (FIC) Columns.

5.1.1.1. Before upgrading. For the existing building, under GSA factored loading (D.L + 0.25 L.L) all six scenarios of column removal did not fail. The worst case was found to be the removal of Edge Short Column (ESC) which gives the highest deflection of 1070 mm, while the least of them was removal of First Edge Long Column (FELC) with deflection of 640 mm, as shown in Table 1.

Also, it was found that the removal of First Internal Column (FIC) and (FELC) give smaller deflection than those of the corresponding deflection in removal of Internal Column (IC) and Edge Long Column (ELC), respectively. This could be attributed to the orientation of the four columns adjacent to the removed one; i.e. in case of removal of IC, it had two columns oriented along their strong axis and two columns oriented along their weak axis, while removal of FIC had three columns oriented along their strong axis and one on its weak axis as shown in Fig. 6. Similarly, it was found that the removal of FELC has smaller deflection than the case of removal of ELC. This can be attributed to the orientation of columns surrounding ELC, where it had one column oriented on its strong axis and two columns on their weak axis, while removal of FELC had two columns oriented on their strong axis and one on its weak axis.

Also, the deflection of removal of ESC is found to be the largest deflection and rotation and this could be due to that the three beams projected from the removed column are connected to the adjacent three columns through their weak axes and connected to small number of bays. On the other hand, the scenario of removal of ELC shows smaller deflection than the scenario of removal of

Table 1

Maximum deflection and (the corresponding chord rotation) for all column removal scenarios for the existing building under GSA loading and for upgraded building by strength factor of 1.25 under the DoD loading.

Removed column	GSA 2003	DoD 2005
Edge Short Column	1070 mm (10.1°)	1168 mm (11.0°)
Corner Column	930 mm (8.8°)	1020 mm (9.6°)
Internal Column	876 mm (8.3°)	973 mm (9.2°)
First Internal Column	819 mm (7.8°)	921 mm (8.8°)
Edge Long Column	737 mm (7°)	822 mm (7.8°)
First Edge Long Column	643 mm (6.1°)	728 mm (6.9°)

ESC, because it has one column oriented on its strong axis and has higher number of bays in its direction, as shown in Fig. 6.

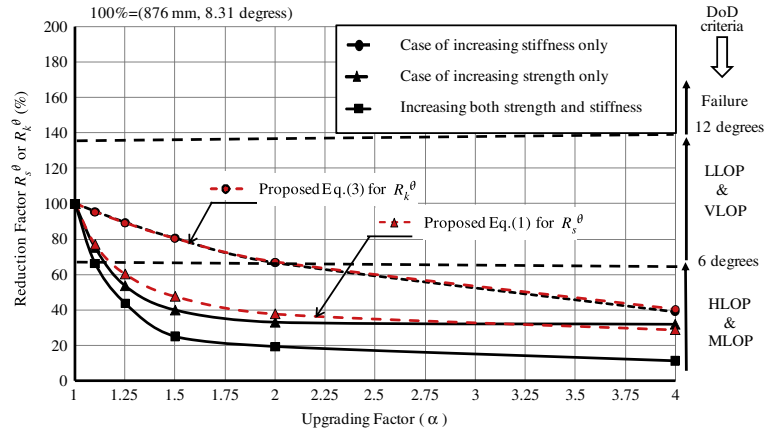
5.1.1.2. After upgrading. In this section, the effect of upgrading the beams by increasing strength and/or stiffness is investigated. Two reduction factors R_s^θ and R_k^θ are introduced and defined as the reduction factor of chord rotation after increasing strength and stiffness factor, respectively, and are equal to the percentage of the ratio of upgraded chord rotation $\theta_{upgr.}$ to the chord rotation $\theta_{orig.}$ of the existing structure.

Fig. 7 shows the reduction factors in chord rotation (θ) for the case of removing the IC after increasing strength and/or stiffness. Also, two proposed equations for the reduction factors R_s^θ and R_k^θ are plotted in Fig. 7. From Fig. 7(a), it can be seen that increasing the strength till a strength factor of 2 ($\alpha_s = 2$) has a great effect on the reduction in chord rotation R_s^θ , whereas negligible effect on the level of reduction in chord rotation R_k^θ was seen afterwards (reduction is less than 10% till $\alpha_s = 4$). On the other hand, this is not the case for the value of the reduction factor R_k^θ due to the increase in stiffness factor α_k which decreases approximately linearly. It can be also seen that increasing the strength of the beams has more effect on reducing the chord rotation when compared with increasing the stiffness of the beams, especially for upgrading factors less than 2 ($\alpha_s < 2$). The latter observation is valid for all six scenarios of column removal. From the analysis, it was found that for upgrading the beams by an upgrading factor of 2 ($\alpha = 2$), which corresponds to an increase in either strength or stiffness by 100% from existing model, reduction factor of chord rotation after increase in strength only and stiffness only for all six scenarios were around 35% and 65%, respectively, which means that retrofit strategy of increasing strength only is more effective than increasing stiffness only.

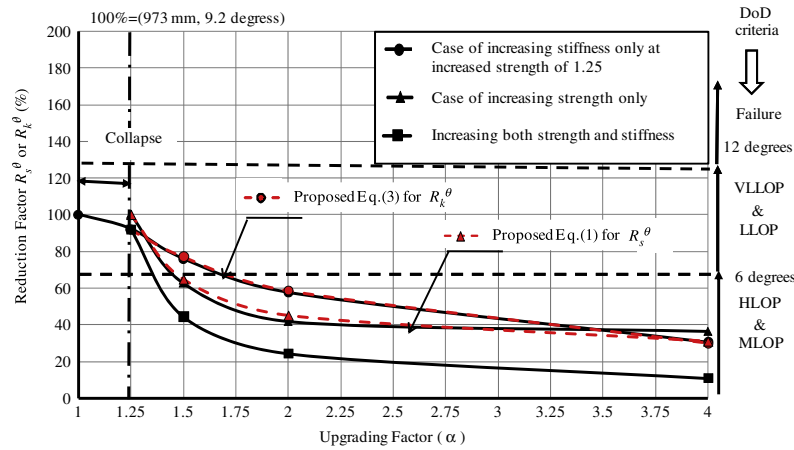
For the case of increasing both stiffness and strength, the analysis showed that the reduction factor in chord rotation $R_{s,k}^\theta$ at different upgrading factor, $\alpha_{s,k}$, was simply the product of both reduction factors R_s^θ and R_k^θ .

Since the original model subjected to load combination of the DoD had failed, thus increasing stiffness only did not prevent the failure because the beams does not have sufficient capacity to resist the loads. Therefore, the effective retrofit strategy in this case is by increasing strength only. As such, the reduction factor in chord rotation R_k^θ in case of increasing the stiffness of the beams is associated with an increase in strength by 1.25 of that of the original structure (subjected to the DoD loads), as shown in Fig. 7(b). In the same manner, R_s^θ is calculated with respect to the model after increasing strength of beams by 1.25 of that of the original model. Also, Table 1 shows the deflection and chord rotation of the beams after upgrading by strength factor of 1.25 for all scenarios of column removal.

In this study, two equations for the reduction in chord rotation due to increasing stiffness R_k^θ and strength R_s^θ for different levels of upgrading factor α are proposed. Eq. (1) gives the values of R_s^θ as a function of α_s , and Eq. (3) gives the values of R_k^θ as a function of α_k . The coefficients "a" and "b" in both equations are given for the different cases of column removal in Table 2(a,b) for loading



(a) GSA 2003.



(b) DoD 2005.

Fig. 7. Reduction factors in chord rotation (θ) for the case of removing the Internal Column after increasing strength and/or stiffness only and the proposed equations for R_s^θ & R_k^θ for loading according to: (a) GSA2003; (b) DOD2005.

Table 2

Values of “a” and “b” coefficients in Eqs. (1) and (3) for estimating the reduction factors R_s^θ and R_k^θ for chord rotations due to increasing strength and stiffness, respectively, when subjected to:

(a) GSA loading				
Removed column	R_s^θ $\frac{100\alpha_s/[a.\alpha_s + b]}{a}$		R_k^θ $\frac{1000/[a.\alpha_k + b]}{a}$	
	a	b	a	b
Internal Column	4.30	−3.30	4.90	5.10
Corner Column	5.10	−4.10	6.35	3.65
Edge Long Column	4.44	−3.44	6.40	3.60
Edge Short Column	6.10	−5.10	7.86	2.14
First Edge Long Column	4.10	−3.10	6.82	3.18
First Internal Column	4.85	−3.85	6.00	4.00
(b) DoD loading				
Removed column	R_s^θ $\frac{100\alpha_s/(a.\alpha_s + b)}{a}$		R_k^θ $\frac{1000/(a.\alpha_k + b)}{a}$	
	a	b	a	b
Internal Column	4.28	−4.10	8.30	0.53
Corner Column	4.52	−4.4	8.06	0.93
Edge Long Column	3.81	−3.51	8.30	0.78
Edge Short Column	5.25	−5.31	7.65	2.60
First Edge long Column	3.60	−3.25	7.85	2.22
First Internal Column	4.28	−4.10	7.87	1.50

using the GSA and DoD, respectively. The proposed equations for calculating the reduction factors R_s^θ and R_k^θ are as follows:

$$R_s^\theta = \frac{100.\alpha_s}{a.\alpha_s + b} \quad (1)$$

where

$$\theta_{upgr.,s} = R_s^\theta . \theta_{orig}. \quad (2)$$

$$R_k^\theta = \frac{100}{a.\alpha_k + b} \quad (3)$$

where

$$\theta_{upgr.,k} = R_k^\theta . \theta_{orig}. \quad (4)$$

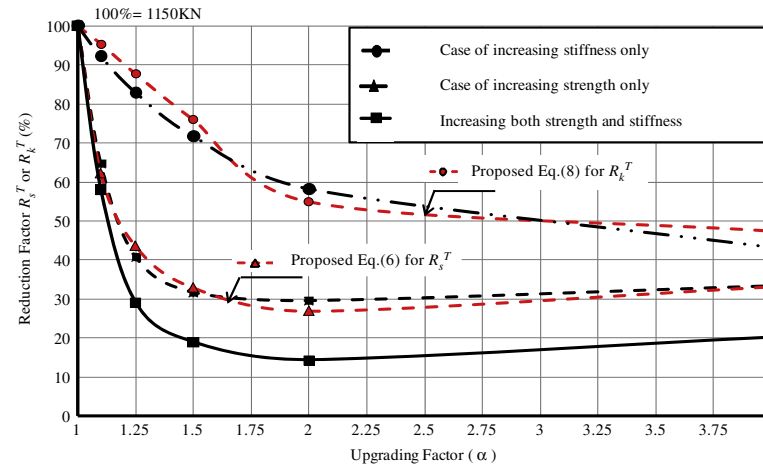
Using the above equations, the chord rotation after upgrading can be estimated. It was also concluded that for the case of retrofitting the beams by increasing both stiffness and strength the chord rotation after upgrading $\theta_{up.,s,k}$ can be predicted by the following equation:

$$\theta_{upgr.,s,k} = R_k^\theta . R_s^\theta . \theta_{orig}. \quad (5)$$

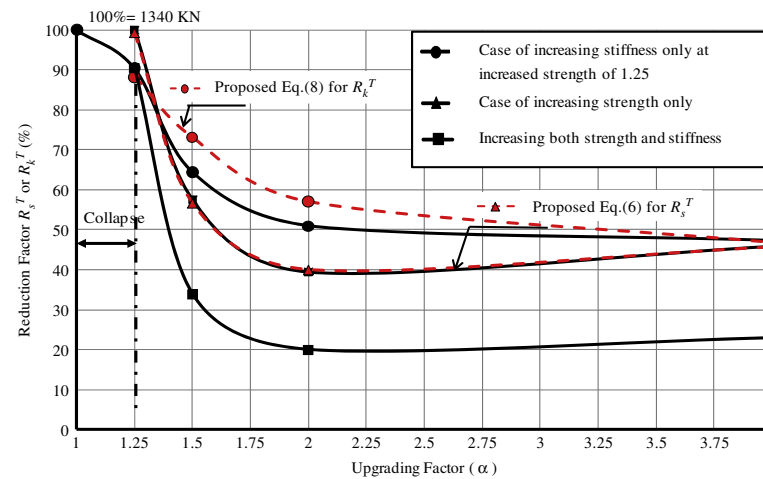
where R_k^θ and R_s^θ can be obtained from Eqs. (1) and (3) and their corresponding coefficients in Table 2.

5.1.2. Effect of retrofit strategy on Tie Forces (TF)

Tie Force (TF) in beams, which is an axial tension force exerted in the beam under high deflection due to the catenary action of the beam, is obtained from the nonlinear dynamic analysis and compared with the limits stated by the DoD guideline. For the



(a) GSA 2003.



(b) DoD 2005.

Fig. 8. Reduction factors in Tie Force (TF) for the case of removing the Internal Column after increasing strength and/or stiffness only and the proposed equations for R_s^T & R_k^T for loading according to: (a) GSA2003; (b) DOD2005.

studied building, the limit value of the tie force according to the DoD guideline for the cases of removal of any internal column (i.e. IC and FIC) and perimeter column (i.e. ESC, ELC, FELC or CC) is equal to 264 and 137 kN, respectively.

5.1.2.1. Before upgrading. In case of GSA Loading, it was found that the tie forces in the beams reached a value of 1150 kN (in case of removal of Internal Column), as shown in Table 3. This force is more than four times of what is estimated using the DoD guideline. On the other hand, tie forces exerted in adjacent beams in case of removal of a FIC were 625 kN, which is about 55% that of IC, yet still higher than the values defined by the DoD. For perimeter column (i.e. ESC, ELC, FELC and CC), the arising tie forces were in the vicinity of 400 kN which is almost three times of that estimated by the DoD. Also, among the perimeter columns, the scenario of removing ELC resulted in a relatively higher tie force.

In case of the DoD loading, the model showed that the existing building will collapse for any scenario of column removal, as mentioned in Section 5.1.1.2, whereas a level of strengthening of beams by 1.25 deemed the building safe against collapse. For the latter case, the value of tie forces for different cases of column removal using the DoD loads showed similar behaviour to that of the GSA loading, but with different values (as shown in Table 3).

The above mentioned behaviour, that interior columns (i.e. IC and FIC) exerted higher tie forces when compared with perimeter ones, could be attributed to the fact that the interior columns are

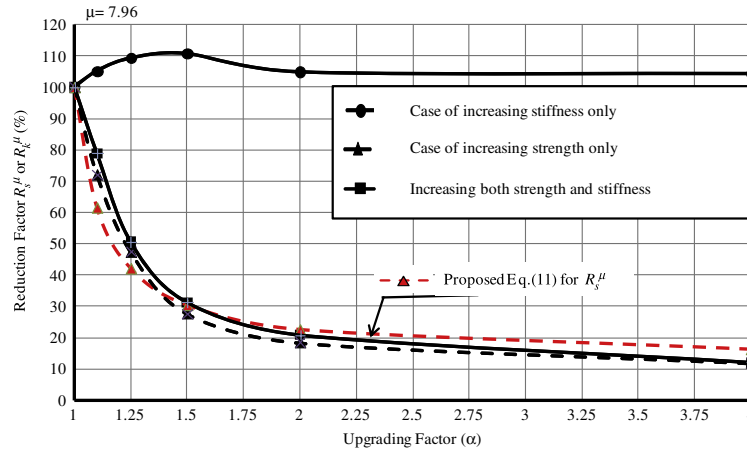
Table 3

Tie Forces (kN) in beams for all column removal scenarios for GSA loading of the existing building under GSA loading and for upgraded building by strength factor of 1.25 under the DoD loading.

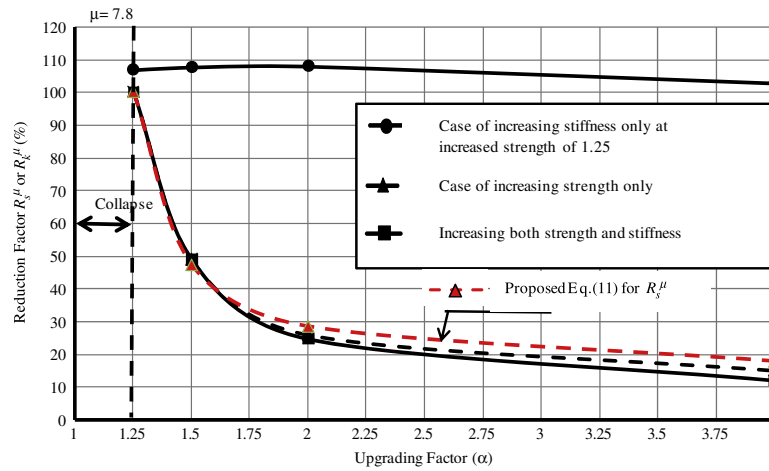
Removed column	GSA 2003	DoD 2005
Internal Column	1150	1340
Corner Column	410	460
Edge Short Column	400	450
Edge Long Column	500	640
First Edge long Column	390	490
First Internal Column	625	720

supporting bigger tributary area (more loads), which lead to higher tension forces in the beams after they exert their full flexural capacity. Similar to the cases of GSA loading, it was found that the exerted tie force in all scenarios is more than three times that of the value estimated by the DOD guideline. This observation was also concluded by Liu et al. [14] who found that the tie force in the beam of a 7-storey model was very high when compared with BS 5950 [BSI, 2000].

5.1.2.2. After upgrading. Similar to the reduction factors defined for the chord rotation, two reduction factors R_s^T and R_k^T , are introduced and defined as the reduction factors of tie forces after increasing strength only and stiffness only, respectively, and are equal to the percentage of the ratio of the tie force of upgraded



(a) GSA 2003.



(b) DoD 2005.

Fig. 9. Reduction factors in displacement ductility demand (μ_Δ) for the case of removing the Internal Column after increasing strength and/or stiffness only and the proposed equations for R_s^μ & R_k^μ for loading according to: (a) GSA2003; (b) DOD2005.

beams $TF_{upgr.}$ to the tie force of the original beams $TF_{orig.}$. Alternatively, for the DoD, these ratios are defined as the percentage of the ratio of the Tie Force of upgraded beams $TF_{upgr.}$ to the tie force of the beams after increasing strength by 1.25 times ($\alpha_s = 1.25$). This is due to the collapse of the original model, thus it does not have values for tie forces.

Fig. 8 shows the reduction factors in tie force (TF) for the case of removing the IC after increasing strength and/or stiffness along with two proposed equations for the reduction factors R_s^T and R_k^T .

From Fig. 8(a), it is found that upgrading the beams by increasing their strength only up to a strength factor $\alpha_s = 2$ leads to a significant reduction in the tie forces, whereas additional increase in the strength factor beyond $\alpha_s = 2$ does not enhance the reduction in the tie forces. On the other hand, increasing the stiffness of the beams up to a stiffness factor of $\alpha_k = 2$ has a linear trend on the reduction factor for tie force, and similar to the case of increasing strength, increasing stiffness beyond $\alpha_k = 2$ has an insignificant effect on enhancing the reduction in the tie forces. Fig. 8(b) shows a similar trend in the reduction factors in tie forces of the beams when the building is loaded with the DoD loading.

After conducting the nonlinear dynamic analysis on the building using the three retrofit strategies and the six scenarios of column removal when subjected to the two cases of loading (GSA and DoD), two equations for estimating the reduction factors in tie force due to an increase in stiffness R_k^T and strength R_s^T for different

levels of upgrading factor α are proposed. Eq. (6) gives the values of R_s^T as a function of α_s , and Eq. (8) gives the values of R_k^T as a function of α_k . The coefficients "a", "b" and "c" in both equations are given for different cases of column removal in Table 4(a,b) for loading using the GSA and DoD, respectively. The proposed equations for calculating the reduction factors R_s^T and R_k^T are as follows:

$$R_s^T = \frac{100 \cdot \alpha_s}{a \cdot \alpha_s^2 + b \cdot \alpha_s + c} \quad (6)$$

where

$$TF_{upgr.,s} = R_s^T \cdot TF_{orig.} \quad (7)$$

$$R_k^T = \frac{1000}{a \cdot \alpha_k^2 + b \cdot \alpha_k + c} \quad (8)$$

where

$$TF_{upgr.,k} = R_k^T \cdot TF_{orig.} \quad (9)$$

It was also concluded that for case of retrofitting the beams by increasing both stiffness and strength, Tie Force in beam after upgrading $TF_{up.,s,k}$ can be predicted by the following equation:

$$TF_{upgr.,s,k} = R_s^T \cdot R_k^T \cdot TF_{orig.} \quad (10)$$

where R_k^T and R_s^T can be obtained from Eqs. (6) and (8) along with their corresponding coefficients in Table 4.

Table 4

Values of “a”, “b” and “c” coefficients in Eqs. (6) and (8) for estimating the reduction factors R_s^T and R_k^T for tie forces in beams due to increasing strength and stiffness, respectively, when subjected to:

(a) GSA loading						
Removed column	R_s^T $100\alpha_s/(a.\alpha_s^2 + b.\alpha_s + c)$			R_k^T $1000/(a.\alpha_k^2 + b.\alpha_k + c)$		
	a	b	c	a	b	c
Internal Column	−1.38	10.60	−8.22	11.30	−1.30	0
Corner Column	−1.30	9.80	−7.50	11.20	−1.20	0
Edge Long Column	−1.13	8.42	−6.29	−0.15	9.45	0.70
Edge Short Column	−1.32	9.27	−6.95	−0.40	5.70	4.70
First Edge Long Column	−1.10	7.90	−5.80	−1.10	12.50	−1.40
First Internal Column	−0.01	4.91	5.10	−0.610	5.23	−3.62
(b) DoD loading						
Removed column	R_s^T $100\alpha_s/(a.\alpha_s^2 + b.\alpha_s + c)$			R_k^T $1000/(a.\alpha_k^2 + b.\alpha_k + c)$		
	a	b	c	a	b	c
Internal Column	−1.13	8.66	−7.80	−2.30	15.70	−4.70
Corner Column	−1.10	8.60	−7.70	−2.20	15.60	−4.60
Edge Long Column	−1.10	8.32	−7.45	−2.19	15.40	−4.00
Edge Short Column	−1.20	8.50	−7.50	−2.27	15.44	−3.96
First Edge long Column	−0.92	7.00	−6.05	−2.30	15.10	−2.50
First Internal Column	−0.76	5.80	−4.80	−2.40	16.00	−4.80

Table 5

Displacement ductility demand, μ_Δ , of the beams adjacent to removed columns of the existing building under the GSA and DoD loadings.

Removed column	GSA 2003	DoD 2005
Edge Short Column	9.8	8.9
Corner Column	8.5	7.9
Internal Column	8.0	7.7
First Internal Column	7.5	7.1
Edge Long Column	6.7	6.3
First Edge Long Column	6.1	5.6

5.1.3. Effect of retrofit strategy on displacement ductility demand (μ_Δ)

Displacement ductility demand is defined as the ratio of the deflection under the removed column for each case to the yield deflection (Δ_y) of the adjacent beams. Yield deflection can be calculated by pushdown analysis that can determine the linear portion in the force deflection curve. Pushdown analysis is conducted using nonlinear static analysis without proceeding by performing dynamic analysis.

5.1.3.1. Before upgrading. GSA and DoD guidelines limit the maximum displacement ductility demand in the beams to a value of 20. In all scenarios of column removal under the GSA and DoD loading for the studied building, the maximum displacement ductility demand reached was 10, which is half of the limit stated by the GSA and DoD. The highest ductility demand occurs from the scenario of removing ESC, while the least value arises from FELC. This trend is similar to that of the chord rotation and deflection. Table 5 shows the displacement ductility demand μ_Δ , of the beams adjacent to removed columns of the existing building under the GSA and DoD loadings.

5.1.3.2. After upgrading. Similar to the reduction factors defined previously, two reduction factors R_s^μ and R_k^μ for the case of increasing strength only and stiffness only, respectively, are introduced and defined as the percentage of the ratio of the ductility demand of upgraded beams, μ_{upgr} , to the ductility demand of the original beams, μ_{orig} . For the case of the DoD loading, these ratios are defined as the percentage of the ratio of the ductility demand of upgraded beams μ_{upgr} to the ductility demand of the beams after increasing strength by 1.25 times ($\alpha_s = 1.25$) due to the collapse of the existing building if not retrofitted (i.e. at $\alpha_s = 1.0$).

Table 6

Values of “a” and “b” coefficients in Eq. (11) for estimating the reduction factors for ductility demand in beams due to increasing strength, R_s^μ , when subjected to the GSA and DoD loading, respectively.

Loading case	Increase strength only $R_s^\mu = 100\alpha_s/(a.\alpha_s + b)$	
	a	b
GSA loading	8.0	−7.0
DoD loading	7.7	−8.4

It was observed that upon increasing the strength only of the beams, the displacement ductility demand decreases and this is attributed to the decrease in maximum deflection along with an increase in yield deflection which leads to a decrease in the displacement ductility demand. On the other hand, increasing the stiffness only of the beams results in a reduction in both the maximum deflection and yield deflection at almost the same rate. This resulted in the fluctuation of the values of the displacement ductility demand within a range of $\pm 15\%$ of its original values. Thus, it can be said that strengthening the beams by increasing their stiffness only has no significant effect on their displacement ductility demand. This means that increasing both strength and stiffness will lead to a similar behaviour for ductility as that of increasing of strength only. Fig. 9 shows the reduction factors in displacement ductility demand (μ_Δ) for the case of removing the IC after increasing strength and/or stiffness only and the proposed equations for R_s^μ and R_k^μ for the GSA and DoD loading.

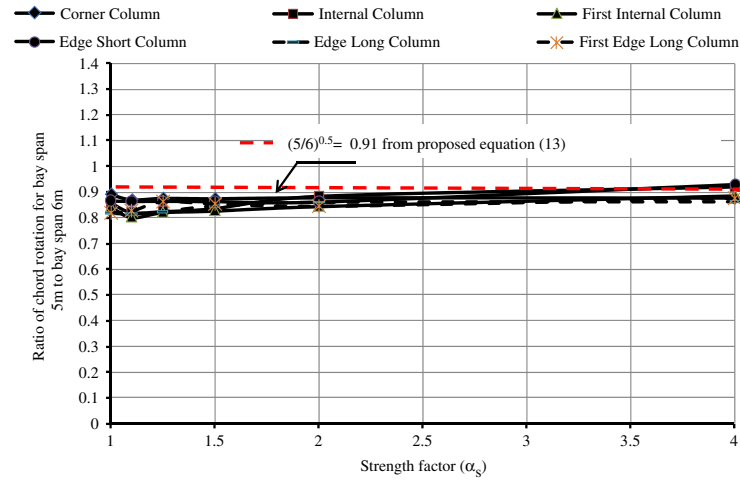
Since R_k^μ does not change significantly, its values is taken constant and equal to 100%. Eq. (11) is proposed to calculate the values of R_s^μ for different levels of increase strength α_s , where the coefficients “a” and “b” are shown in Table 6. The coefficients had almost the same values for different scenarios of column removal under loading criteria, i.e. either GSA or DoD.

$$R_s^\mu = \frac{100.\alpha_s}{a.\alpha_s + b} \quad (11)$$

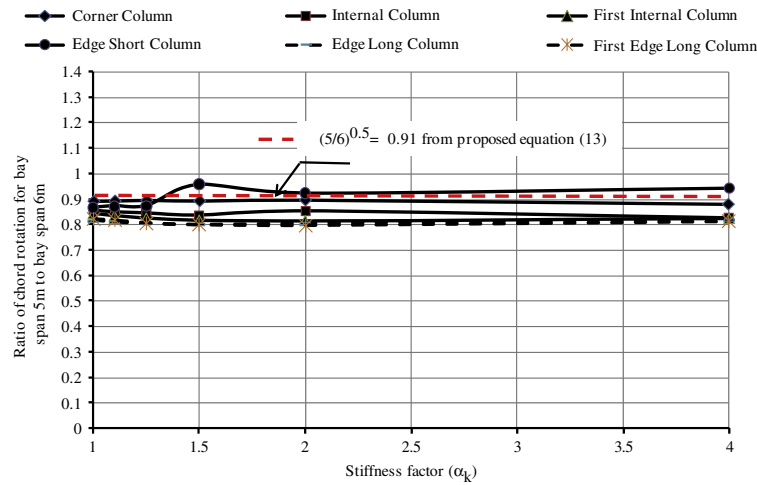
where

$$\mu_{upgr..s} = R_s^\mu . \mu_{orig}. \quad (12)$$

Using these reduction factors, the displacement ductility demand in the beam after upgrading can be estimated according to Eq. (12). It was also concluded that for the case of retrofiting



(a) Retrofitting by increasing strength only.



(b) Retrofitting by increasing stiffness only.

Fig. 10. Effect of changing the bay span from 6.0 m (reference model) to 5.0 m on the chord rotation for the six scenarios of column removal: (a) after retrofitting by increasing strength only; (b) retrofitting by increasing stiffness only.

the beams by increasing both stiffness and strength, displacement ductility demand in beam after upgrading $\mu_{up.,s,k}$ can be considered approximately equal to $\mu_{up.,s}$ as the reduction factor for retrofitting by increasing stiffness only R_s^u is about 100%.

It is worth mentioning that the values of coefficients in Tables 2, 4 and 6 had a coefficient of determination, R^2 , values that ranged from 0.9 to 1.0.

5.2. Effect of variation of bay span

In this section, the effect of variation of bay span on the values of the chord rotation, tie force, and displacement ductility demand (for the cases of building before and after upgrade) were studied by considering three other different spans of 5.0 m, 7.5 m and 9.0 m.

5.2.1. Effect of variation of bay span on chord rotation (θ)

It was found that the most critical case for models of spans 5.0 m, 6.0 m (reference model) and 7.5 m was the scenario of removing ESC, whereas for the model with span 9.0 m the most critical case was removal of CC. For all models with different spans, it can be concluded that the perimeter column loss scenario is more critical than the interior column loss scenarios. In addition, it could be said that as the span increases significantly the removal of corner column scenario will be the most critical.

Models with spans of 5.0 m, 7.5 m and 9.0 m showed similar responses to the reference model (i.e. with 6.0 m bay span). Vertical deflection in case of removal of IC and ELC is more than that of FIC and FELC, respectively. Also, the vertical deflection in case of removing ESC is more than that in case of removal of ELC.

Fig. 10 shows the effect of changing the bay span from 6.0 m (reference model) to 5.0 m on the chord rotation for different upgrade factors α_s and α_k for the six scenarios of column removal. From the figure, it can be seen that the average value for the six scenarios of column removal fluctuates around 0.91. Similar behaviour was obtained from the analysis of the 7.5 m and from 6.0 m to 9.0 m buildings, where the effect of changing the bay span from 6.0 m to 7.5 m and 9.0 m were 1.12 and 1.22, respectively. These values were found to be close to the square root of the ratio of spans. Eq. (13) shows the effect of changing the bay span on the chord rotation for original or upgraded buildings.

$$\frac{\theta_{orig,1}}{\theta_{orig,2}} = \frac{\theta_{upgr.,1}}{\theta_{upgr.,2}} = \left[\frac{L_1}{L_2} \right]^{0.5} \quad (13)$$

where L_1 and L_2 are two different bay spans.

Table 7 shows the ratio of values of the chord rotations for different spans as obtained from the analysis and as estimated by Eq. (13). The table shows that the difference between the values estimated by the equation and those obtained from dynamic analysis is insignificant.

Table 7
Ratios of chord rotations values (average of six scenarios of column removal) for different spans as obtained from the analysis, and as obtained from Eq. (13), as well as the (percentage of error).

	Span 5.0 m	Span 6.0 m	Span 7.5 m	Span 9.0 m
Span 5.0 m	1, 1	1.17, 1.10 (−6.3%)	1.33, 1.23 (−7.7%)	1.40, 1.34 (−4.2%)
Span 6.0 m	0.85, 0.91 (6.8%)	1, 1	1.13, 1.12 (−1.3%)	1.20, 1.22 (2%)
Span 7.5 m	0.76, 0.82 (7.4%)	0.89, 0.90 (0.8%)	1, 1	1.06, 1.10 (3.3%)
Span 9.0 m	0.72, 0.75 (4%)	0.84, 0.82 (−2.4%)	0.95, 0.91 (−3.5%)	1, 1

Table 8
Ratios of tie forces values (average of six scenarios of column removal) for different spans as obtained from the analysis, and as obtained from Eq. (14), as well as the (percentage of error).

	Span 5.0 m	Span 6.0 m	Span 7.5 m	Span 9.0 m
Span 5.0 m	1, 1	1.84, 1.73 (−6%)	3.53, 3.37 (−4.3%)	6.10, 5.83 (−4.4%)
Span 6.0 m	0.55, 0.58 (5.9%)	1, 1	1.92, 1.95 1.7%	3.31, 3.37 (1.8%)
Span 7.5 m	0.29, 0.30 (3.8%)	0.53, 0.51 (−2.6%)	1, 1	1.74, 1.73 (−0.5%)
Span 9.0 m	0.17, 0.17	0.30, 0.30	0.58, 0.58	1, 1

Table 9
Ratios of displacement ductility demand values (average of six scenarios of column removal) for different spans as obtained from the analysis, and as obtained from Eq. (15), as well as the (percentage of error).

	Span 5.0 m	Span 6.0 m	Span 7.5 m	Span 9.0 m
Span 5.0 m	1, 1	1.15, 1.20 (4%)	1.45, 1.50 (3%)	1.66, 1.80 (8.6%)
Span 6.0 m	0.87, 0.83 (−3.9%)	1, 1	1.26, 1.25 (−0.9%)	1.44, 1.50 (4.3%)
Span 7.5 m	0.69, 0.67 (−3.2%)	0.79, 0.80 (0.8%)	1, 1	1.14, 1.20 (5%)
Span 9.0 m	0.60, 0.56 (−8%)	0.7, 0.67 (−4.2%)	0.88, 0.83 (−5%)	1, 1

5.2.2. Effect of variation of bay span on Tie Forces (TF)

Similar observations to the reference model (with 6.0m span) were found in the models with spans of 5.0 m, 7.5 m and 9.0 m. The removal of IC and ELC exerts higher tie forces than the removal of FIC and FELC, respectively. Also, tie forces exerted in the scenario of removal of ELC is higher than that in the scenario of removal of ESC.

From the analysis, it was observed that the average value of the ratio of the tie forces for buildings with bay spans of 5.0 m, 7.5 m, and 9.0 m when compared with the building with bay span 6.0 m for the six scenarios of column removal fluctuates around 0.57, 1.95, and 3.375, respectively. These values were found to be close to the ratio of spans cubed. Eq. (14) shows the effect of changing the bay span on the tie forces for original or upgraded buildings.

$$\frac{TF_{orig,1}}{TF_{orig,2}} = \frac{TF_{upgr.,1}}{TF_{upgr.,2}} = \left[\frac{L_1}{L_2} \right]^3 \quad (14)$$

where L_1 and L_2 are two different bay spans.

Table 8 shows the ratio of values of the tie forces for different spans as obtained from the analysis and as estimated by Eq. (14). The table shows that the difference between the values estimated by the equation and those obtained from dynamic analysis is insignificant.

It is worth mentioning that Eq. (14) shows that the variation in tie forces are proportional to the (variation in span)³, whereas the present recommendations of the DoD states that the tie forces are proportional to the area served, i.e. variation in tie forces are proportional to (variation in span)². This could justify the low estimated values of the tie forces by the DoD when compared with the obtained values from analysis shown in Table 3. Observations of low estimated values of tie forces by the DoD were also reported by Liu et al. [14].

5.2.3. Effect of variation of bay span on displacement ductility demand (μ_Δ)

Highest ductility demand was found to be in the case of removing ESC for spans 5.0 m, 6.0 m and 7.5 m, whereas for the bay span of 9.0 m the removal of corner column was found to result

in the highest ductility demand among all other scenarios. Also, it was found that the effect of the location of the removed column on the level of displacement ductility demand follows similar trend as that observed for chord rotations (i.e. Tables 1 and 5).

From the analysis, it was observed that the average value of the ratio of the displacement ductility demand for buildings with bay spans of 5.0 m, 7.5 m, and 9.0 m when compared with the building with bay span 6.0 m for the six scenarios of column removal fluctuates around 0.83, 1.25, and 1.50, respectively. These values were found to be close to the ratio of spans. Eq. (15) shows the effect of changing the bay span on the displacement ductility demand for original or upgraded buildings.

$$\frac{\mu_{orig,1}}{\mu_{orig,2}} = \frac{\mu_{upgr.,1}}{\mu_{upgr.,2}} = \left[\frac{L_1}{L_2} \right] \quad (15)$$

where L_1 and L_2 are two different bay spans.

Table 9 shows the ratio of values of the displacement ductility demand for different spans as obtained from the analysis and as estimated by Eq. (15). The table shows that the difference between the values estimated by the equation and those obtained from dynamic analysis is insignificant.

6. Conclusions

A 3-D nonlinear dynamic analysis was conducted on a high-rise steel gravity frame using the APM to predict the performance enhancement in the chord rotation, tie force and displacement ductility demand after being retrofitted using three different schemes and subjected to six scenarios of column removals at its ground level according to the GSA and DoD criteria. Two sets of analyses were conducted. First set was conducted on a building with a bay span of 6.0 m in order to evaluate the reduction factors in the three performance indicator parameters due to the three studied retrofit strategies. Equations for estimating the reduction factors for chord rotation, tie forces, and displacement ductility demand were proposed. Second set was conducted on three buildings with spans of 5.0 m, 7.5 m, and 9.0 m in order to assess the effect of variation of bay span on the proposed equations. The following conclusions can be drawn from the results of the studied cases:

- (1) Upgrading the beams by increasing their strength only is more effective than increasing their stiffness only in enhancing the three performance indicators; chord rotation, tie force, and displacement ductility demand.
- (2) The reduction factor in case of upgrading both strength and stiffness of the beams is found to be equal to the numerical product of the reduction factor arising from the case of increasing strength only and that arising from the case of increasing stiffness only.
- (3) For the studied buildings, all column removal scenarios where the building is loaded according to the DoD resulted in a collapse of the building, which was not the case when the building was loaded according to GSA criteria. This highlights the importance of further research for clear identification of the combination of loads that can better represent gravity loading in alternative load path method.
- (4) The level of tie force exerted in the beams of the existing building calculated from nonlinear dynamic analysis using ELS software is more than three times of the limits stated by the DoD guideline for all studied buildings, which confirms similar findings by other researchers. This highlights a need for more research to identify appropriate estimations for Tie Forces.
- (5) For all studied buildings, chord rotation, tie force and displacement ductility demand in case of loss of Internal and Edge Long Column scenarios are more than those arising from the case of First Internal and First Edge Long Column removal scenarios, respectively. This could be attributed to the orientation of the columns adjacent to the removed one; the higher the number of adjacent columns oriented along their strong axes, the lower the chord rotation, Tie Force and displacement ductility demand.
- (6) Tie force in the scenario of removing Edge Long Column is higher than that exerted in the scenario of Edge Short Column removal for all the studied buildings due to the higher number of bays in edge long direction.
- (7) Effect of varying the bay span on chord rotation was found to be proportional to (ratio between spans)^{0.5}.
- (8) Effect of varying the bay span on tie force was found to be proportional to (ratio between spans)³, whereas in the DoD guideline it is proportion to the area serviced, i.e (ratio between spans)².
- (9) Effect of varying the bay span on displacement ductility demand is approximately directly proportional to the ratio between the bay spans.

From the above conclusions, it can be seen that the choice of the most suitable rehabilitation scheme to safeguard against the progressive collapse should consider the loading criteria, the targeted level of safety, and the desired performance parameter

needed to be enhanced. It is important to clarify that the results drawn are for the specific studied cases. More models for different structure configurations and capacities should be considered and more analysis including cost analysis is needed for the conclusions to be generalized.

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