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EFFECT OF RETROFIT METHOD ON MITIGATING PROGRESSIVE COLLAPSE OF TALL STEEL FRAME STRUCTURE

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Abstract: In this study, the effect of three retrofit methods 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 Defence (DoD) guidelines for resisting progressive collapse. The response is evaluated using 3-D nonlinear dynamic analysis. The studied model represents 6-bay by 3-bay 18-storey steel frame that is damaged by being subjected to six scenarios of sudden removal of one column in the ground floor. The response of the damaged frame 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 and tie forces for the beams of the studied building after being retrofitted.

1. Introduction

In progressive collapse, an initial localized damage or local failure spreads through neighbouring elements, possibly resulting in the failure of the entire structural. The ASCE 7-05 commentary suggests general design guidance for improving the progressive collapse resistance of structures. 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 (GSA, 2003) and (DoD, 2005) guidlines.

GSA and DoD guidelines recommended the use of the direct approach or the Alternate Path Method (APM). In this method, an analysis is conducted after a single column in the ground level is typically assumed to be suddenly missing. The alternate path method is mainly concerned with the vertical deflection or the chord rotation of the building after the sudden removal of a column.

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 and deciding on the level of protection against such potential event of sudden loss of a supporting column. 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 pre-defined level of damage or to prevent collapse of the building. This is similar to the Performance-Based Seismic Design (PBSD) adopted by several guides.

The retrofit strategy may involve targeted repair of deficient members, providing systems to increase stiffness and strength or 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

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. 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.

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 (see Fig. 1).

The effectiveness of the retrofit methods of damaged buildings is evaluated by comparing two performance indicator parameters, namely, chord rotations and tie forces of the beams after being upgraded to those of the original existing structure. The building with bay span of 6.0m was studied in order to evaluate the reduction factors in the three performances indicator parameters due to the three studied retrofit strategies.

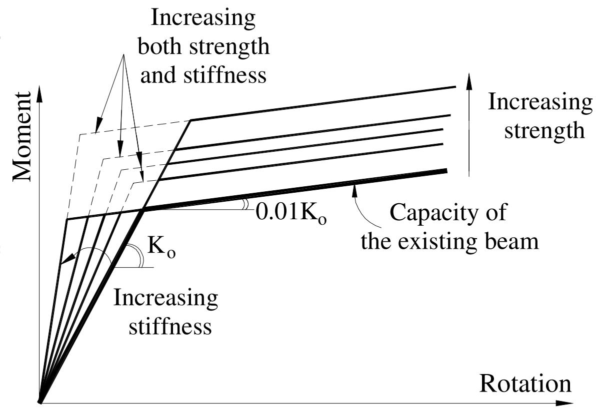

Fig. 1. Methods of upgrading the structure by increasing strength and/or stiffness of beams.

3. Details of the analytical models

3-D model of 18-storey high-rise steel moment resisting frame buildings having 3x6 bays in plan were constructed using Extreme Loading for Structures (ELS) software. The buildings have the same plan throughout the whole height. The sizes of the columns were kept constant for every three stories along the height; whereas two sizes for the beams where designed and kept constant for the whole height, namely, perimeter beams and internal beams. The studied models have bay span 6.0m in the two directions. The buildings were designed according to CISC-95 for gravity loading condition. Figs. 2 and 3 show the elevation and plan of the studied building, respectively, along with their respective column and beam sizes.

The frame where designed with slab thickness of 20cm, live load of 2.4 kPa, and a superimposed dead load of 2 kPa was taken to account for the equivalent load from interior partition. In the model, a bilinear stress-strain relationship of the steel members was taken, with F_y = 350 MPa, and strain hardening of 1%. 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 was taken into account in ELS software.

In the models, following assumptions were used:

(1) Loads from concrete slabs are applied directly on the beams without representation in the model; (2) Connections between the beam and the column maintains continuity; (3) Fixed support is considered; and (4) Increase of yield strength arising from the high rate of straining due to removal of column is neglected.

ELS software uses the Applied Element Method (AEM). 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.

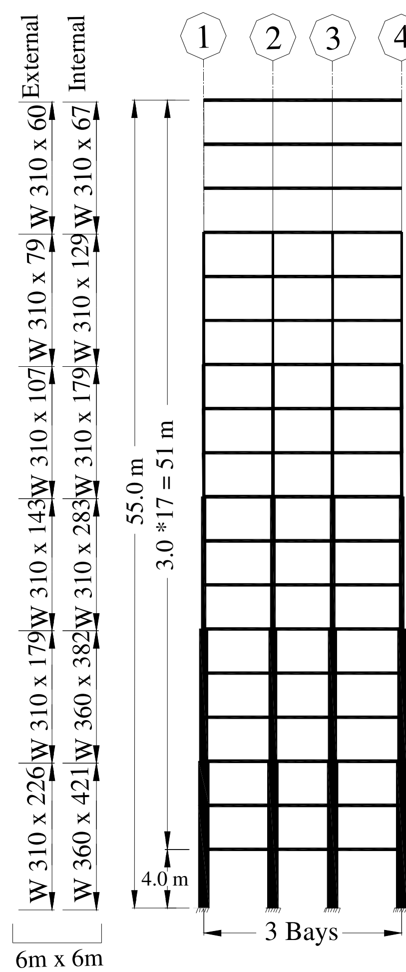

Fig. 2. Elevation of the studied building and column sizes.

4. Method of analysis

Progressive collapse of structures adopted Performance-Based Design Method (PBDM) as a practical way that depends on objective criteria. For steel frame buildings, 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. 3. For each case, the effect of three retrofitting strategies on the chord rotation (θ) and Tie Forces (TF) of the beams are evaluated. In the current 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 increase in strength or stiffness of 10, 25, 50, 100 and 300% from the original model, respectively.

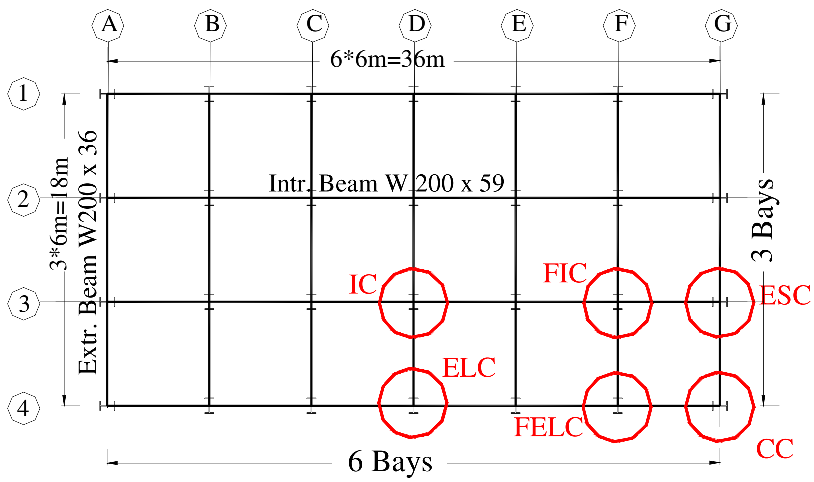

Fig. 3. Plan of the building, beam sizes and the six studied

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 the strength or the capacity of the section in proportion, where the plastic moment capacity of the section is $M_p=Z_x. F_y$, 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 and shear modulus, which will lead to increase 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 guideline, while the second is (1.25 D.L + 0.5 L.L) according to DoD 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.

5. Results and discussion

Fig. 4 shows the flow chart of the nonlinear dynamic analyses to evaluate the effect of three retrofit strategies on three performance indicators (θ, TF, and μ_Δ) for the studied building.

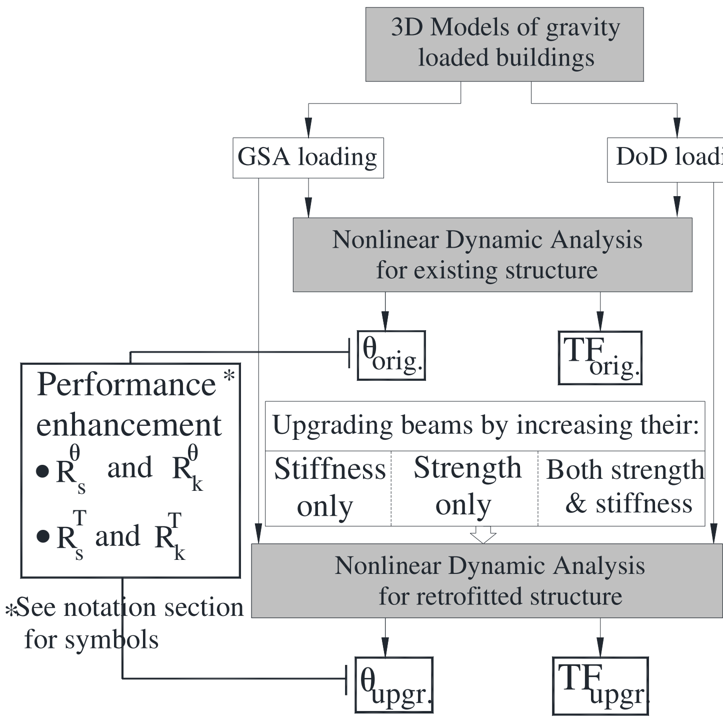

Fig.4.Flow chart to evaluate the effect of three retrofit strategies

5.1. Effect of retrofit strategy on chord rotation

As defined by 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.

5.1.1. before upgrading

For the existing building, under GSA factored loading 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. 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 DoD loading.

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. 5. Similarly, it was found that 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.

Table 1. Maximum deflection and (chord rotation) for all scenarios under GSA loading and DoD loading

Removed column	GSA 2003	DoD 2005
ESC	1070 mm (10.1°)	1168 mm (11.0°)
CC	930 mm (8.8°)	1020 mm (9.6°)
IC	876 mm (8.3°)	973 mm (9.2°)
FIC	819 mm (7.8°)	921 mm (8.8°)
ELC	737 mm (7°)	822 mm (7.8°)
FELC	643 mm (6.1°)	728 mm (6.9°)

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 (ESC) because it has one column oriented on its strong axis and has higher number of bays in its direction, as shown in Fig. 5.

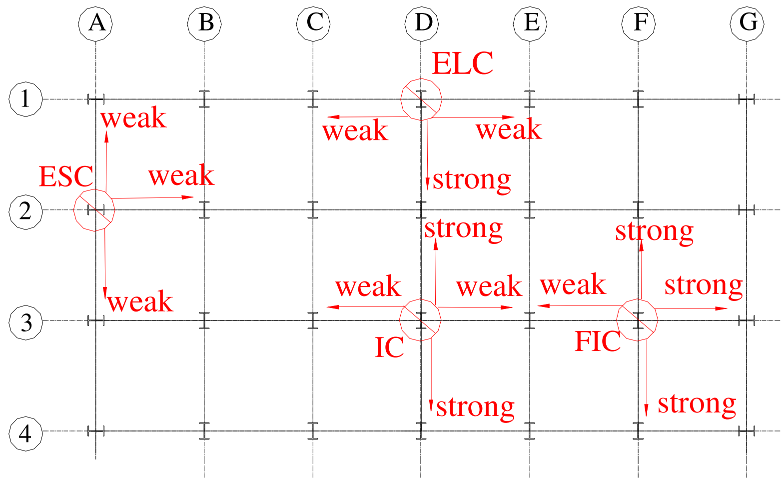

Fig. 5. Illustration of strong and weak connections for cases of removal of Edge Short (ESC), Edge Long (ELC), Internal (IC) and First Internal (FIC) Columns

5.1.2. 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 θ_{upgr}. to the chord rotation θ_{orig}. of the existing structure**.**

Fig. 6 shows the reduction factors in chord rotation (θ) for case of removing the Internal Column (IC) after increasing strength and/or stiffness. From Fig. 6(a), it can be seen that increasing the strength till a strength factor of 2 (α_s=2) has great effect on reduction in chord rotation R_s^{θ} , whereas negligible effect on the level of reduction in chord rotation R_s^{θ} was seen afterwards (reduction is less than 10% till α_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 compared to increasing the stiffness of the beams, especially for upgrading factors less than 2 (α_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 (α=2), which corresponds to 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 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 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.

a) GSA 2003

b) DOD 2005

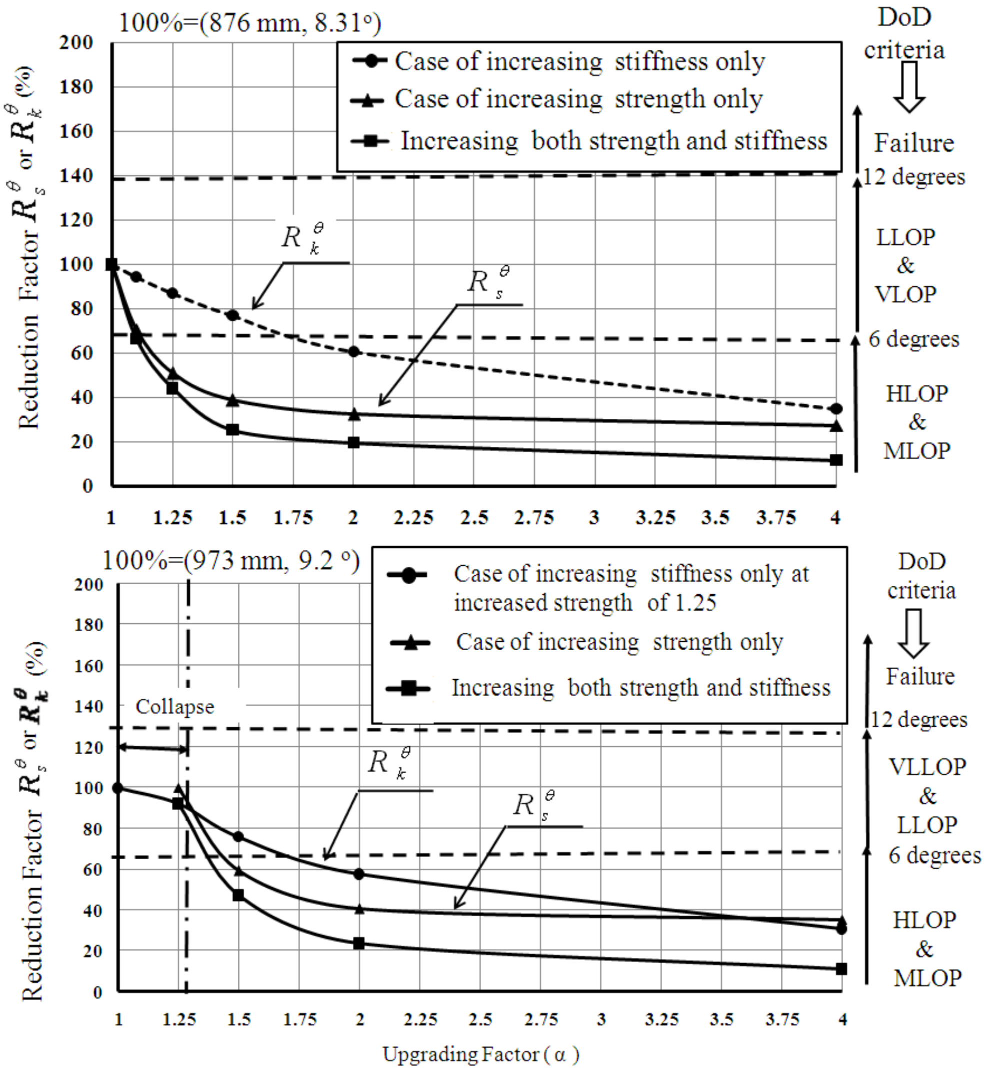

Fig. 6. Reduction factors in chord rotation (θ) for case of removing the Internal Column after increasing strength and/or stiffness only for R_s^θ & R_k^θ for loading according to : a) GSA2003; b) DOD2005.

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 DoD loads), as shown in Fig 6(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.

[1] $\theta_{upgr.,s} = R_s^\theta \, .\theta_{orig.}$

[2] $\theta_{upgr.,k} = R_k^\theta \, .\theta_{orig.}$

Using the above equations, the chord rotation after upgrading can be estimated. It was also concluded that for 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:

[3] $\theta_{upgr.s,k} = R_k^\theta \, . \, R_s^\theta \, . \, \theta_{orig.}$,

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

Tie Force (TF) in beams, which is an axial tension force exerted in the beam under deflection due to the catenary action of the beam, is obtained from the analysis and compared to the limits stated by DoD guideline. For the studied building, the limit value of the tie force according to DoD guideline for 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.

5.2.1. 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 2. This force is more than four time what is estimated using DoD guideline. On the other hand, tie forces exerted in adjacent beams in case of removal of a First Internal Column were 625 kN, which is about 55% that of IC, yet still higher than the values defined by DoD. For perimeter column (i.e. ESC, ELC, FELC and CC) the arising tie forces where in the vicinity of 400 kN which is almost three times that estimated by DoD. Also, among the perimeter columns, the scenario of removing (LEC) Column resulted in relatively higher tie force

Table 2. 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 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

In case of DoD loading, the model showed that the existing building will collapse for all scenario of column removal, 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 DoD loads showed similar behaviour to that of the GSA loading (as shown in Table 2).

The above mentioned behaviour, that interior columns (i.e. IC and FIC) exerted higher tie forces compared to perimeter ones, could be attributed to the fact that interior columns are 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 DOD guidelines. This observation was also concluded by Liu et al. who found that the tie force in beam of 7-storey model was very high compared to BS 5950 [BSI, 2000].

5.2.2. 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 beams TF_{upgr}. to the tie force of the original beams TF_{orig}.. Alternatively, for 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 (α_s=1.25). This is due to the collapse of the original model, thus it doesn't have values for tie forces. Fig. 6 shows the reduction factors in ties force (TF) for case of removing the (IC) after increasing strength and/or stiffness under GSA loading.

From Fig. 7, it is found that upgrading the beams by increasing their strength only up to a strength factor α_s =2 leads to significant reduction in the tie forces, whereas additional increase in the strength factor beyond α_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 α_k =2 has a linear trend on the reduction factor for tie force, and similar to the case of increasing strength, increasing stiffness beyond α_k =2 has insignificant effect on enhancing the reduction in the tie forces. Similar trend in the reduction factors in ties forces of the beams when the building is loaded with DoD loading.

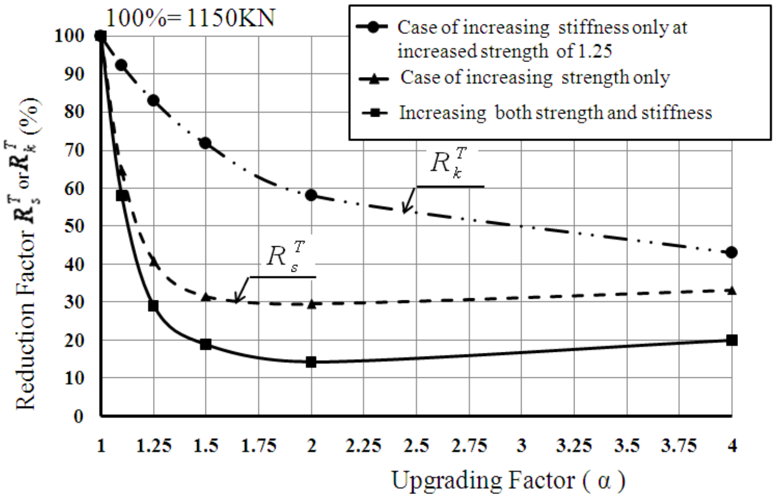

Fig. 7. Reduction factors in Tie Force (TF) for case of removing the Internal Column after increasing strength and/or stiffness only for GSA Loading.

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), the reduction factors in tie force due to increase stiffness R_k^T and strength R_s^T for different levels of upgrading factor α are shown in Fig. 7.

[4] $\mathrm{TF}_{\mathrm{upgr.,s}} = R_s^T \;.\; \mathrm{TF}_{\mathrm{orig.}}$

[5] $\mathrm{TF}_{\mathrm{upgr.,k}} = R_k^T \;.\; \mathrm{TF}_{\mathrm{orig.}}$

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

[6] $\mathrm{TF}_{\mathrm{upgr.,s,k}} = R_s^T \;.\; R_k^T \;.\; \mathrm{TF}_{\mathrm{orig.}}$

6. Conclusions

3-D nonlinear dynamic analysis was conducted on a 18 storey steel gravity frame using Alternative Path Method (APM) to predict the performance enhancement in the chord rotation (θ) and Tie Force (TF) after being retrofitted using three different schemes and subjected to six scenarios of column removals at its ground level according to GSA and DoD criteria. The building with bay span of 6.0m in order to evaluate the reduction factors in the three performances indicator parameters due to the three studied retrofit strategies. 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 two performance indicators.
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 case of increasing strength only and that arising from case of increasing stiffness only.
3) For the studied buildings, all column removal scenarios where the building is loaded according to 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 (APM).

4) Tie Forces exerted in beams of the existing building calculated from nonlinear dynamic analysis using ELS software is more than three times of the limits stated by DoD guideline for all studied buildings, which confirms similar findings by other researchers and more research is needed for estimating Tie Forces.
5) For all studied buildings, chord rotation and tie force in case of loss of Internal and Edge Long Column scenarios are more than those arising from 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 and Tie Force.
6) Tie force in scenario of removing Edge Long Column is higher than that exerted in scenario of Edge Short Column removal for all studied buildings due to the higher number of bays in edge long direction.

From the above conclusions it can be seen that the choice of the most suitable rehabilitation scheme to safeguard against 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 are needed for the conclusions to be generalized.

Also, it should be noted that the current analysis does not address the influence of the rehabilitation schemes on the natural period and modes of vibration of the structure and their effect on the seismic design of the retrofitted building.

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Notations

$\theta_{upgr.,s}$, θ_{upgr} Upgraded chord rotation after increasing the strength only and both strength and stiffness, respectively.

R_s^θ, R_k^θ Reduction factor in chord rotation due to increase in strength and stiffness, respectively.

R_s^T, R_k^T Reduction factor in tie force due to increase in strength and stiffness, respectively.

α_s, α_k, $\alpha_{s,k}$ Strength Factor due to increase in strength, stiffness and both strength and stiffness

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