Effects of Retrofit Strategies on Mitigating Progressive Collapse of Steel Building

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Abstract— **Progressive collapse is the collapse of large part of a structure resulted by damage or failure of a relatively small part of it. Designing the buildings to resist progressive collapse is uneconomical due to intense loading effects associated with these hazards. There is an increasing trend of upgrading public buildings to resist progressive collapse caused by accidental loading like blast, vehicle collision, fire, earthquake etc. In this paper, different retrofit strategies to enhance the response of 4-storied steel moment resisting building is investigated using recommendations of the General Services Administration (GSA) guideline for resisting progressive collapse. The response of building to progressive collapse is evaluated using linear and nonlinear analysis. The building is subjected to different ground floor column loss scenarios. The response of the structure 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.**

Keywords: Collapse load, DCR, progressive collapse, displacement ductility, steel frame, retrofit, plastic hinge rotation

I. INTRODUCTION

In the past decades, many buildings around the world have experienced partial or total progressive collapse under extreme loading conditions. The extreme / accidental loadings can be generated by aircraft impact, design/construction error, fire, gas explosions, accidental overload, storage of hazardous materials, vehicular collision, bomb explosions etc. Progressive collapse occurs when relatively local structural damage causes a chain reaction of structural element failures, disproportionate to the initial damage, resulting in partial or full collapse of the structure. As the probability of occurrence of these accidental loadings is low for the structures of normal importance, thus these loadings are neither considered in structural design nor addressed by passive protective measures. Most of these hazards have characteristics of acting over a relatively short period of time and result in dynamic responses.

Many government agencies require that new public buildings should be designed to resist the effects of accidental loadings that can cause extreme damage. Some private building owners also intend to evaluate and upgrade their existing buildings to resist the progressive collapse. It may be possible to design buildings to resist such extreme loading without significant damage, but the loadings effects associated with these hazards are so intense that design measures necessary to provide such performance would result in high costs as well as impose unacceptable limitations on the architectural design of such buildings.

The most viable approach to resist the progressive collapse is to maintain the integrity and ductility of the structural system. The ASCE 7-05 [1] commentary suggests a general design guidance for improving the progressive collapse resistance of the structure. General Service Administration (GSA) guidelines [13] and the Department of Defense (DoD) guidelines [12] developed by U.S. give the design procedures to mitigate the potential for progressive collapse in structures. GSA and DoD guidelines recommended the use of the direct approach or the Alternate Path Method (APM) for mitigating progressive collapse. In APM, a single column in the ground level is assumed to be suddenly missing, and an analysis is conducted to determine the ability of the damaged structure to bridge across the missing column.

Kim and Kim [7] studied the response of steel moment resisting frames using APM with different damage scenarios. A corner column, a first edge and internal edge columns were removed for the study. Static, nonlinear static and nonlinear dynamic analysis procedures were followed. Fu [3] 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. Kim and Park [6], carried out the study of progressive collapse potential of three and nine-story special steel moment frames. Nonlinear static and dynamic procedures were followed. Kwasniewski [9], presented a case study of progressive collapse analysis of a selected 8-storey steel building. Non-linear dynamic analysis was followed using GSA guidelines. NIST guideline [10], includes “best practices” to reduce the likely hood of progressive collapse of buildings in the event of abnormal loading. It includes a discussion of an acceptable risk approach to progressive collapse. It also presents a worldwide review of progressive collapse provisions in various national design standards. In a recent investigation, Galal and El-Sawy [8] studied the different retrofit strategies on enhancing the response of existing steel moment resisting frames designed for gravity loads.

Existing buildings that are designed using Indian standards are expected to have inadequate resistance to progressive collapse. One of the major challenges for a structural engineer is choosing an appropriate retrofit scheme for existing steel building with a higher potential for progressive collapse. The retrofit strategy may involve targeted repair of deficient members, providing systems to increase stiffness and strength.

In this paper, the effect of three retrofit strategies on enhancing the response of 4- storied steel moment resisting frame building designed for gravity and lateral loads is investigated using Alternate Path Methods recommended in the General Services Administration (GSA) guideline for resisting progressive collapse. The response is evaluated using 3-D linear and nonlinear analysis using SAP2000 [11]. The building is subjected to four scenarios of sudden removal of one column from the ground floor. The response of the damaged structure 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 various performance indicators.

II. Problem Definition

The ductility of steel cannot assure 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. A retrofit strategy using Fiber-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 additional continuous steel plates will increase both strength and stiffness of the beam.

The effectiveness of three different retrofit strategies for beams is assessed in this paper. 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 four performance parameters namely, DCR, chord rotation, displacement ductility of the beams and collapse load after being upgraded to those of the original existing structure.

III. Case of 4-Storey Steel Building

The building considered for the study is a four-story steel moment resisting frame structure [14].

A. Building geometry:

Building has six bays in the longitudinal direction and three bays in the transverse direction. The longitudinal direction has a uniform column spacing of 8.25 m while on the three-bay side columns are spaced at every 9.75 m. Floor-to-floor height for every story is 4.3 m. Typical floor plan of the building is shown in Fig. 1. Elevation of the building is shown in Fig. 2. Fig. 3 shows the three dimensional model of a structure.

B. Loadings for structural design:

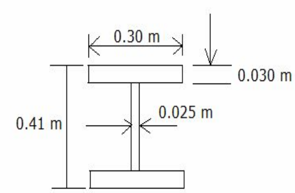
Dead Load:
- Self weight of the structure
- Thickness of the slab = 90mm
- Wall load = 19.7 kN/m at every floor except roof on periphery.

Live Load:
- 1.9 kN/m^2 distributed uniformly across the entire floor area including roof.

Seismic Load [5]:
- Zone: III
- Importance factor: 1
- Response reduction factor: 5
- Soil type: 2

After considering critical load combinations, the structure is designed [4] and sections provided are shown in Table I:

TABLE I: Column and beam schedule

PB001-PB004	ISMB500
SB001-SB004	ISMB450
C1-C28	Built up I section 

C. Loading for Progressive collapse Analysis

Alternate load path method suggested by GSA is followed for progressive collapse analysis of structure. In this method column at ground floor is removed depending on case. The structure is subjected to gravity loading as per guidelines and demand in terms of shear force, bending moment and axial force is evaluated from the analysis results. Capacity at critical sections is obtained from original design. If Demand Capacity Ratio (DCR) exceeds permissible values, the element is considered as failed. In the static and dynamic progressive collapse analysis after column removal following vertical load shall be applied downward on the structure.

Load = 2(DL + 0.25LL) for linear and non liner static analysis
Load = (DL + 0.25 LL) for linear and nonlinear dynamic analysis

Where, DL = dead load and LL = live load

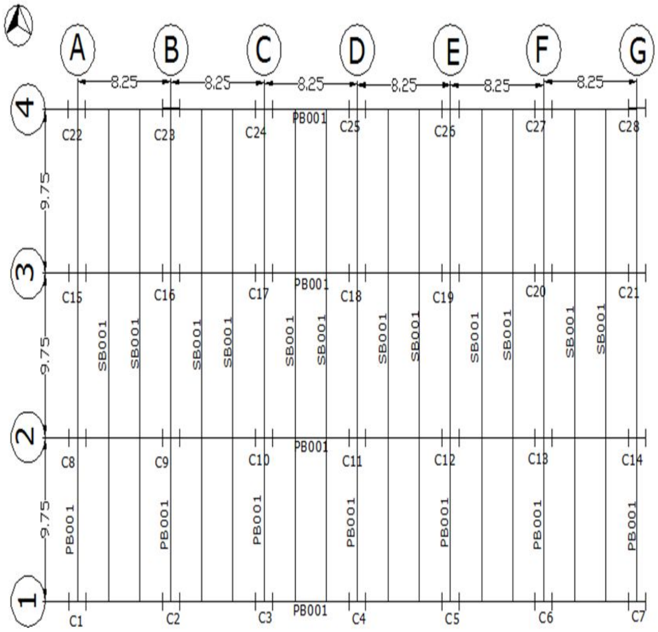


Fig. 1 Plan of 4-storey moment resistant steel building

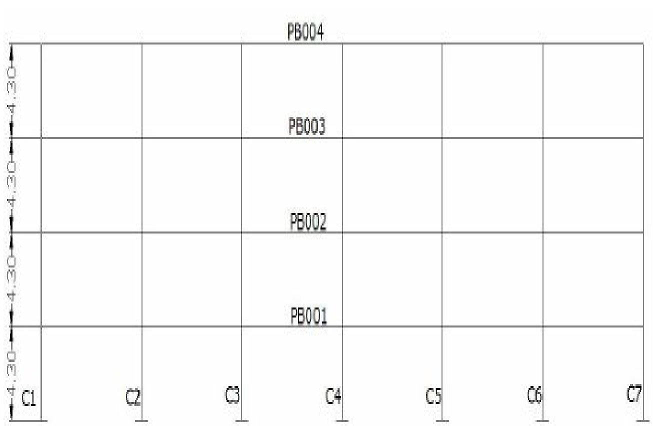


Fig. 2 Elevation of 4-storey moment resistant steel building

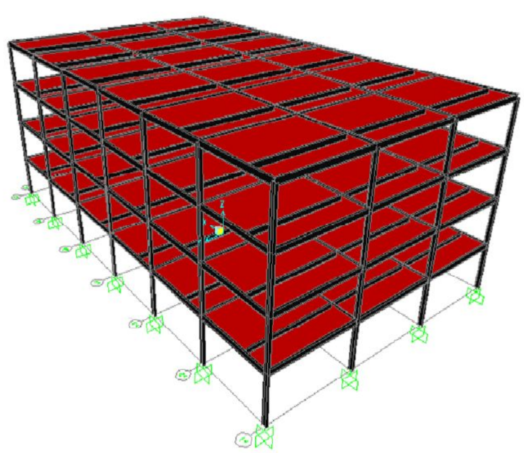


Fig. 3 Three dimensional model of structure

IV. RETROFITTING OF BUILDING

Out of four column removal scenarios i.e. middle column of longer side, middle column of shorter side, corner column and interior column, only one column removal case is considered for the presentation in this paper. Column removal of C15 is considered. Column is removed from the ground level. There are total three retrofitting strategies adopted as follows:

A. Retrofit strategy-1:

In this strategy, yield strength of beams and columns are increased. Initially the yield strength adopted for analysis was 250 MPa. In this strategy, the values of yield strengths considered for retrofitting are 300 MPa and 340 MPa. So in this strategy, the increase of the strength is achieved up to 20% and 36% respectively.

B. Retrofit strategy-2:

In this strategy, stiffness of beams and columns are increased. Increase in the stiffness is achieved by increasing the modulus of elasticity and shear modulus up to 20% and 36%.

C. Retrofit strategy-3:

In this strategy, both the stiffness and strength of the beams and columns are increased. Increase in the stiffness and the strength of the sections is achieved by increasing the thickness of the flanges of the section by 20% and 36%.

After the loss of column C15, highly stressed nearby columns C8, C22, and C16 are retrofitted and affected beams above the column removal positions are retrofitted. DCR for beam can be found out as:

$$DCR = \frac{Q_{UD}}{Q_{UC}}$$

Where,

Q_{UD} = Acting force (demand) determined in member or connection (moment, axial force, shear and possible combine forces).

Q_{UC} = Expected ultimate, un-factored capacity of the member.

DCR for beams and columns are found out at each floor. Permissible value of DCR for regular steel building is 2.0. DCR for columns are calculated as per following equation:

$$\frac{P}{P_y} + \frac{M_{pc}}{1.18M_p} \leq 1$$

Where,

P = Axial force.

P_y = Yield strength of axially loaded section = (A_s f_y).

A_s = Effective cross section area of the member.

f_y = Yield strength of the section.

M_{pc}= Maximum moment acting in a member.

M_p = Plastic moment capacity of the section.

If the value of DCR exceeds 1, the column is considered to be failed. Fig. 4 and 5 show the DCR for flexure for beams before and after retrofitting by 20% and 36% strength increase respectively.

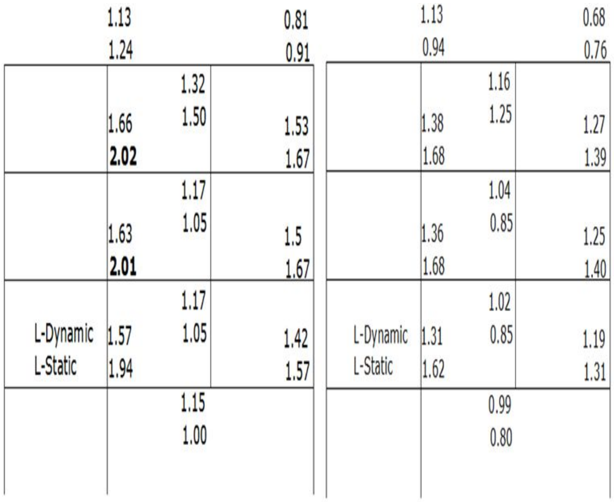


Fig. 4 DCR of beam before and after retrofitting (20% strength increase)

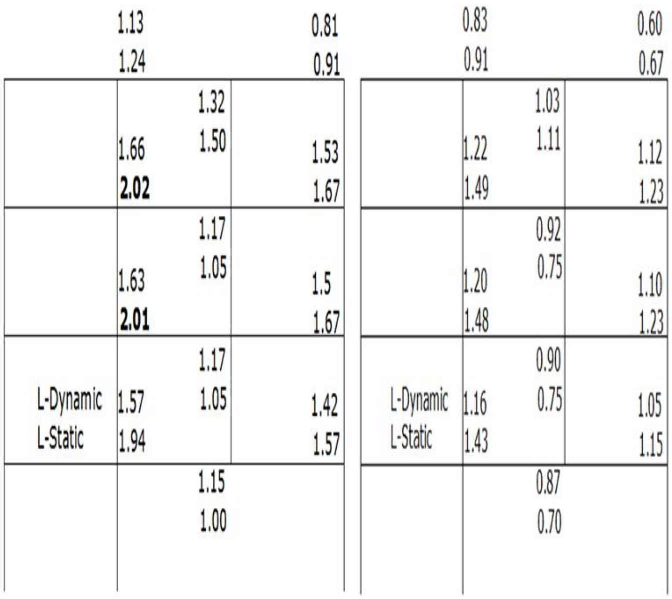


Fig. 5 DCR of beam before and after retrofitting (36 % strength increase)

L-static and L-dynamic show that DCR is found by linear static analysis and linear dynamic analysis procedure. For beams, DCR is calculated at three points left, center and right side of the column removal position as shown in the figures. DCR found from linear static procedures are greater than DCR found by linear dynamic analysis procedure. DCR values exceeding 2.0 collapse of beam and it cannot take more loads.

For progressive collapse analysis, a nonlinear static analysis method implies a stepwise increase of amplified (by a factor of 2) vertical loads until maximum amplified loads are attained or until the structure collapses. This method is also called the vertical pushover analysis. This vertical pushover analysis method is load controlled. For nonlinear analysis procedure using SAP2000, automatic hinge properties are assigned to a frame element. For default moment hinges, SAP2000 uses Table 5-6 of FEMA-356 [2]. For each degree of freedom, there is a force displacement (moment-rotation) curve that gives the yield value and the plastic deformation following yield. This is expressed in terms of a curve with values at five points, A-B-C-D-E, as shown in Fig. 6.

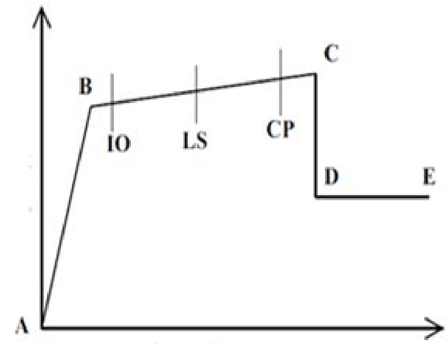


Fig. 6 Moment rotation curve

Point A is always origin, B represents yielding, C represents the ultimate capacity for pushover analysis, D represents a residual strength and E represents total failure. There are additional deformation measures at points IO (immediate occupancy), LS (Life safety), and CP (Collapse prevention). FEMA defines permissible values for plastic rotation of hinges at each stage i.e. IO, LS and CP. In SAP2000, default M3 hinges are assigned to beams at both the ends and default P-M2-M3 hinges are assigned to columns at both the ends. After nonlinear static analysis procedure, plastic hinge rotation is found out at the collapse state and compared with the permissible values of plastic hinge rotation. The graph of vertical deflection vs. percentage of load is plotted for the column removal case. Percentage of load is found by summation of the reactions obtained at the supports for each analysis step divided by total load applied. Fig. 7 shows the formation of first plastic hinge and plastic hinges at collapse along with downward displacement. Fig. 8 shows the maximum collapse load and corresponding maximum support rotation before retrofitting.

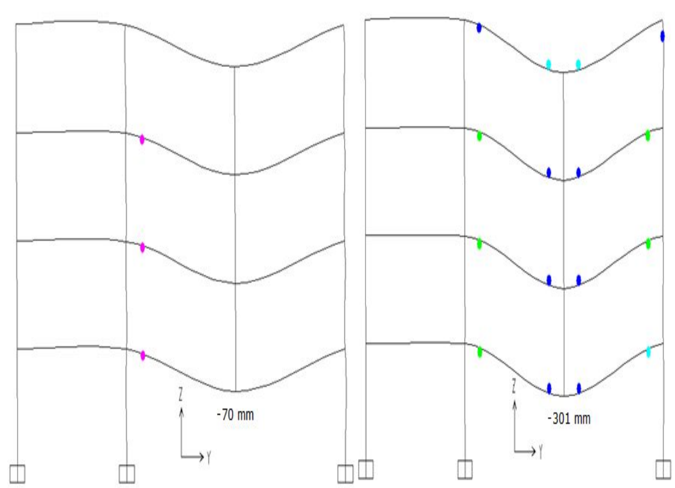


Fig. 7 Hinges at yield and at collapse

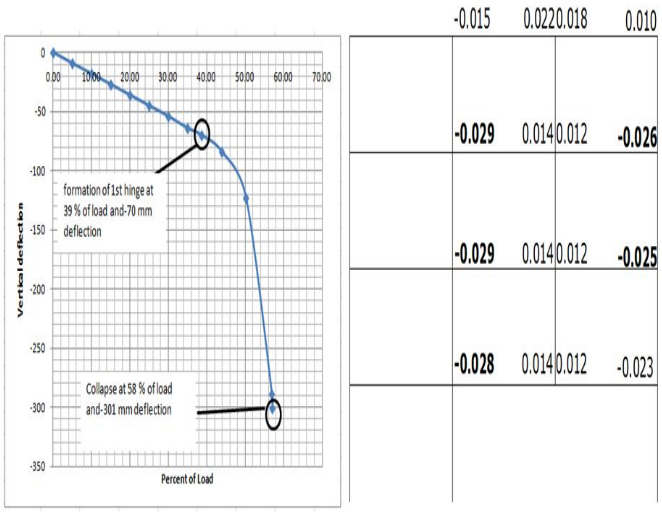


Fig. 8 Vertical displacement v/s percentage of load and corresponding support rotation before retrofitting

The nonlinear dynamic procedure for progressive collapse analysis is considered as very efficient method in which a primary load bearing structural element is removed dynamically and the structural material is allowed to undergo nonlinear behavior. This allows larger deformations and energy dissipation through material yielding, cracking and fracture. Time-history method with initial condition methodology is used to carry out nonlinear dynamic analysis in SAP2000 software. Fig. 9 and 10 shows the displacement for nonlinear dynamic analysis before and after retrofitting by 20% and 36% strength increase respectively. Displacement ductility is found by taking the ratio of ultimate displacement to yield displacement (when the first plastic hinge occurs in the section).

Fig. 9 shows that nonlinear dynamic analysis could not successfully completed because the model has a plastic hinge that failed or mechanism has formed. Maximum support rotation is found by taking the ratio of maximum displacement to the length of the adjacent beam from where the column is removed.

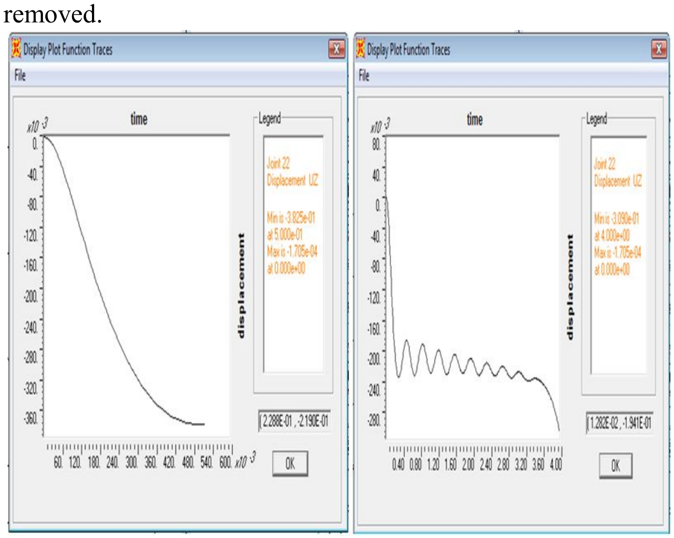


Fig. 9 Displacement before and after retrofitting (20% strength increase)

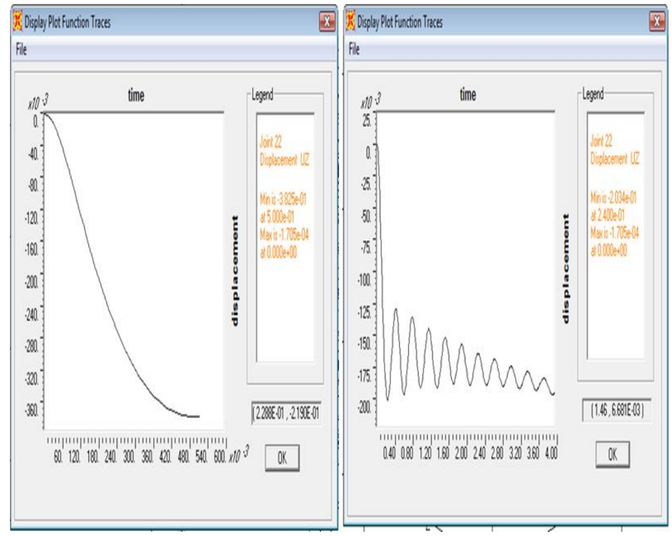


Fig. 10 Displacement before and after retrofitting (36% strength increase)

Table II shows the maximum value of DCR for beam and column, collapse load displacement ductility and plastic hinge rotation before and after retrofitting for strategy-1.

TABLE II: SUMMARY FOR RETROFIT STRATEGY-1

	Without retrofitting	20% strength increase	36% strength increase
DCR for beam	2.02	1.68	1.43
DCR for column	0.73	0.61	0.54
Collapse load	58%	68.3%	77%
Displacement ductility	5.23	3.72	1.99
Plastic hinge rotation	0.033	0.026	0.013

Table III shows the maximum value of DCR for beam and column, collapse load displacement ductility and plastic hinge rotation before and after retrofitting for strategy-2.

TABLE III: SUMMARY FOR RETROFIT STRATEGY-2

	Without retrofitting	20% stiffness increase	36% stiffness increase
DCR for beam	2.02	2.02	2.01
DCR for column	0.73	0.73	0.71
Collapse load	58%	58%	58%
Displacement ductility	5.23	4.35	3.80
Plastic hinge rotation	0.033	0.026	0.023

Table IV shows the maximum value of DCR for beam and column, collapse load displacement ductility and plastic hinge rotation before and after retrofitting for strategy-3.

TABLE IV: SUMMARY FOR RETROFIT STRATEGY-3

	Without retrofitting	20% strength and stiffness increase	36% strength and stiffness increase
DCR for beam	2.02	1.83	1.68
DCR for column	0.73	0.66	0.61
Collapse load	58%	72.5%	79%
Displacement ductility	5.23	2.43	2.09
Plastic hinge rotation	0.033	0.014	0.010

V. SUMMARY AND CONCLUSIONS

Three retrofit strategies are applied for column removal of C15. Comparison between DCR, maximum collapse load, displacement ductility and plastic hinge rotations are carried out for building before retrofitting and after retrofitting. From the obtained results, the following conclusions can be drawn:

- It has been found that upgrading the beams and columns by increasing their strength only is more effective than increasing their stiffness only in enhancing the four performance indicators.
- Reduction in DCR for beams and columns with retrofit strategy-1 is higher compared to retrofit strategy-2. For retrofit strategy-2 it can be observed that there is no change or very negligible change in DCR values of beams and columns.
- Collapse load found from nonlinear static analysis increases significantly for retrofit strategy-1 from 58% to 77% while there is no change in collapse load found with retrofit strategy-2.
- There is significant decrease in displacement ductility from 5.23 to 1.99 in retrofit strategy-1 compared to 5.23 to 3.80 in retrofit strategy-2.
- The value of DCR decreases with retrofit strategy-3 but reduction in values are lesser compared to retrofit strategy-1. For retrofit strategy-1 DCR for beam changes from 2.02 to 1.43 but for retrofit strategy-3 DCR for beam changes from 2.02 to 1.68.
- Collapse load increases more for retrofit strategy-3 from 58 % to 79% compared to retrofit strategy-1.
- Reduction in displacement ductility is found to be more in the case of retrofit strategy-1 compared to retrofit strategy-3.
- The choice of the most suitable retrofitting 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 case. 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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