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PROGRESSIVE COLLAPSE ANALYSIS OF AN INDUSTRIAL RC BUILDING USING P-M INTERACTION: AN ANALYTICAL STUDY

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Abstract:
Progressive collapse is the eventual failure or proportionately substantial failure of a part of a structure caused by the propagation of a local failure from element to element across the system. This might be viewed as the ultimate failure of the structure or the failure of a substantial component of it. The progression of such a collapse may be precipitated by manmade, natural, intentional, or unintentional factors. During this phase, the system in the process of collapse redistributes the loads to prevent the loss of vital structural elements. In this study, SAP2000 was used to examine a reinforced concrete building. The recommendations of the General Services Administration (GSA) and the Department of Defence (DoD) were adopted to anticipate the total amount of collapse. The notion of the study is to take into account the most adverse scenario and suggest the best approach amongst the two, which may help in performing future studies in less time.

Keywords: Progressive Collapse, Linear Dynamic Analysis, Non-Linear Static Analysis, Non-Linear Dynamic Analysis, Bresler method, Demand Capacity Ratio (DCR).

INTRODUCTION

Progressive collapse is the spread of an initial local failure from element to element, resulting, eventually, in the collapse of an entire structure or a significant large part of it (ASCE_07-02, 2007). This can be stimulated by natural or manmade activities like earthquakes and explosions. To make the structure stable enough to withstand such a loss, beam-column joints shall be designed in such a way that they shall reciprocate under the bridging effect of large stresses and negative bending moment (Zhao, 2019)such as explosions or impacts, during their service life. It is, therefore, necessary not only to evaluate their safety under traditional loads and seismic action. The structural performances related to progressive collapse scenarios need to be investigated. The study of progressive collapse involves a dynamic problem, but unfortunately dynamic experiments on the behavior of the civil engineering structures under dynamic conditions are rare. In this research, beam-column sub-assemblage specimens were tested under dynamic load. The loading program consists in placing a large mass, as a dead load, on the top of the middle column of a beam-column sub-assemblage. The support under the middle column is suddenly removed for simulating the sudden loss of a column and the damage that will result in the structure. The loading system and supporting devices were specially designed for this test. The upper dead load can be changed by increasing or decreasing the applied mass to different specimens. The supports for the side column have a controlled rigidity in the horizontal direction and are designed to restrain rotation of the side-column. Thus, the boundary conditions are supposed to be similar to real situations. During the test, a laser was installed under the middle pillar to collect the falling velocity and a high-speed camera was used to visualize the whole process of failure. The images obtained from the camera were processed by Digital Image Correlation (DIC. Some of the famous examples of this phenomena are the collapse of the World Trade Towers in 2001, the collapse of the Murrah Federal Building in 1995, Oklahoma City, U.S.A., because of intentional manmade activities. And an unintentional manmade activity, gas cylinder blast, became a cause of collapse of Ronan Point Building in 1968, London, Britain (Abdelwahed, 2019).

The research has scope for the non-frequent progressive collapse phenomenon, the effect of which can be adverse based on the errors of both design method and specification in reciprocating the spread of damage in the subject project. The progress in this field of study may lead to minimizing damage during the flow of events. Secondly it shall favour superior structure against natural disasters. From the safety point of view more the life saved, the better it is. After initial damage, it will help insure the building does not give-in at others positions. Upon threat due to combatian scenario or assault intended to harm. In all situations the research shall favour better method of both design and specifications. In available literatures, it has been observed that the most appropriate guidelines are provided by the two agencies, GSA (2016) and DoD (2016) and the difference in recommendations are summarized and used for this study. Further, it has been observed that most of the researchers generally use guidelines of both the agencies to perform their studies. Hence, the need to perform such a study which may conclude the best guidelines is very high and the findings may reduce the time to perform future studies on progressive collapse.

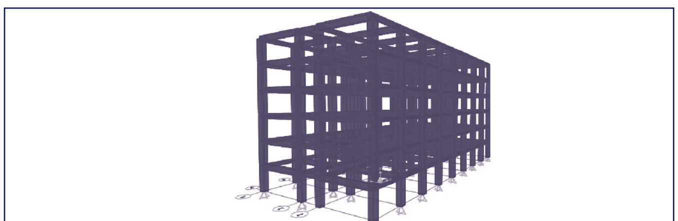
LITERATURE REVIEW

Progressive collapse comes with great non-linear problems like large rotations, displacements and collision between specimens. The damage is irreversible thus, the model to be analysed must consider the said above problems. There are two types of models, finite element model (FEM) and discrete element model (DEM). Three types of modeling falls under FEM to study progressive collapse which are Fine model, Simplified model and Multi-scale model. Fine model generates various elements respective to mechanical behaviour of structural members. This method is little bit complicated because it requires large calculation and thus, the use is restricted to sub-structure and specimens only. Luccioni et al. studied progressive collapse of a fine element RC structure for blast loads (Luccioni et al., 2004). The fine element is time consuming and laborious. Therefore, researchers prefer using simplified model over fine model for research. This method includes ultimate mechanical behaviour by using corrected structure or mechanical parameters. In a study (Zhang and Liu, 2008), authors have developed a model which can define the accumulated large deformation upto some extend. A deferent study (Lu et al., 2010) was performed to simulate failure or facture by using an element deactivation method on two models, fiber beam model and layered shell model. Lastly, for a multi-scale model all the structural members are modelled as a simplified model except those which are under complex stage. To simulate a structure member under complex stress stage preference is to use fine model. Hence, a multi-scale can be said as a combination of fine and simplified model. The authors (Khandelwal et al., 2008) have studied a high rise steel frame by using macro model. A macro model is used to describe material non-linearity and geological behaviour of the specimen. Similar researches using macro model was performed by many researchers (Bao et al., 2008; Talaat and Mosalam, 2007). On the other hand, another method i.e. DEM is similar to FEM in terms of mechanical behaviours. But simulation procedure is different. The method use rigid bodies connected to springs to reciprocate the mechanical behaviour of a structure. Movements of rigid bodies are determined by using Newton's Laws of motion whereas; internal forces are given by spring relationships. Authors (Munjiza et al., 2004) have created a method which uses both DEM and FEM and by comparing the results concludes that the proposed method replicate the progressive collapse more accurately and also simplifies the calculations and establishment of model. Sun et al. have studied progressive collapse of RC bridge under seismic effect using DEM (Sun et al., 2003). On the basis of available literature this may be said that the DEM gives unsatisfactory result before the occurrence of failure whereas; the FEM gives accurate result for the same stage. As the collapse occurs, FEM fails to account some major parameters like movement of ridged bodies and their collision. To account such parameters experimental studies were performed. In a study (Qian and Li, 2012a), authors have performed experiment on realistic beam-column concrete structures providing different spacing between stirrups and reinforcement ratios and concluded the mechanism of collapse, development of cracks and the load- displacement curves. The authors further performed a study (Qian and Li, 2012b) in which the dynamic collapse mechanism was studied on six structural models of 1/3 scale. Authors concluded that collapse can be decreased by keeping small span length. Different experimental study (Sadek et al., 2011)simulating a column removal scenario. The assemblies represent portions of structural framing systems designed as intermediate moment frames (IMFs was performed on the four frames, two of concrete and two of steel, to assess how seismic design affects the progressive collapse resistance of the frames. Wei-Jian et al. performed progressive collapse analysis on concrete frame beams and studied the mechanism and collapse resisting capacity of the modelled beams (Wei-Jian et al., 2008). Destruction of un-operational building by blasting can give better data to study progressive collapse. Hence, a study (Song and Sezen, 2009) was performed on Student Union Building of Ohio State University to calculated the DCR for different members at different locations. In different studies (Sasani et al., 2011, 2007; Sasani and Sagiroglu, 2010, 2008) multiple experiments were performed on real structures, without considering any live load for safety point of view. Experiments were performed by removing some structural components and the findings were in terms of internal force distribution and deformation time-history. Matthews *et al.* performed column removal experiment on a two floor RC frame structure. The column was removed by blasting and redistribution of gravity loads was studied by using dynamic amplification factor. Finding states that the whole structure was in elastic state even after the removal of column (Matthews et al., 2007). To assess robustness and progressive collapse resistance demand of a structure, theoretical studies were performed by many researchers. These two parameters must be considered apart from conventional design method to enhance performance of significant structures. The main focus of researchers is to study the non-linear dynamic effects on the structural capacity during collapse process. A parameter called Dynamic Amplification Factor (DAF) can be used with linear static method as a simplified design methodology to prevent progressive collapse. A study (Marjanishvili and Agnew, 2006) concluded that dynamic analysis is a better approach to study progressive collapse of structures by analysing a nine-storey steel moment resistant frame. In a different study, authors (Joshi et al., 2010) have compared the DCR values obtained from numerical analysis performed for both linear and non-linear cases and studied progression of collapse in a multi-storeyed RCC building. The researchers have concluded that the plastic hinges are initially developed on members with maximum DCR value.

METHODOLOGY

The study is performed using finite element modelling. A space frame of six storeys was modelled in SAP2000. The frame is 32m x 14.5m in plan. It has eight bays of 4m each along the longitudinal direction and two bays of 6m and one of 2.5m along the transverse direction, as shown in figure (1).

Figure 1 SAP2000 Model



All the columns are square in cross-section with same dimension whereas, the beams are rectangular in cross-section with different dimensions. For the calculation of superimposed dead loads, thickness of the exterior walls is considered as 230mm, thickness of partition walls and depth of all the slabs are considered as 150mm. All the dead and live loads are as per the Indian codal provision (Bureau of Indian Standards, 2016). Following the general construction practice of India, the columns are designed for M40 grade of concrete whereas, the beams are made-up of M25 grade of concrete. Grade of steel, Fe415 (HYSD bars), is same for beams and columns. The details of the structure are presented in table (1).

Table 1. Details of building parameters

Items		Description
Storey Height		3m
Column cross-section		600mm x 600mm
Beam cross-section	At all the floors except the roof level	300mm x 450mm for beams spanning 2.5m and 4m
		400mm x 600mm for the beams spanning 6m
	At roof level	250mm x 450mm

The model was first analysed and the base shear correction was done as per Bureau of Indian Standards (2002, 2016). For the analysis, only gravity loads are considered because the probability of occurrence of an earthquake or any other event at the same time of failure of a structural member is too low. Hence, the model is analysed for (DL+0.25LL) load case only and the forces in all the members were noted as the capacity of the members for the calculation of the Demand Capacity Ratio (DCR) as per equation (1).

$$DCR = {M_P}/{M_{\max}} \qquad (1)$$

Where, Mmax is the bending moment of the member (capacity) and MP is the expected ultimate moment capacity of the member. After noting the capacity of the members, the model is designed following the Bureau of Indian Standards (2002, 2016). All the supports were kept fixed for this analysis and designing, after which, the supports were changed to pinned for the study on progressive collapse. The building with pinned supports will experience significantly large bending moment because, the bending moment will not carry over to the support ends and hence provide the worst case scenario. The model is analysed for both linear and non-linear cases. The Linear Dynamic Analysis (LDA) was performed following the recommendations of GSA (2016) whereas, the Non-Linear Static Analysis (NLSA) and Non-Linear Dynamic Analysis (NLDA) were performed as per the recommendations of GSA (2016) and DoD (2016). The static cases accounts a sudden column loss whereas, the dynamic effect can only be seen if the failure of structural member is gradual. For the linear case, the amount of collapse was obtained by calculating the DCR. The members for which the calculated values of DCR is greater than or equals to 2 are said as collapsed because the structure is symmetrical in plan as well as in elevation (GSA, 2016). The guidelines of the agencies (DoD, 2016; GSA, 2016) used for the analyses are summarized in the table (2).

Table 2 Guidelines of GSA (2016) and DoD (2016)

Agency	Load case(s) for analysis	Location of column removal
	Linear Dynamic	
(GSA, 2016)	2(DL+0.25LL) on the entire structure	Outer middle column about longer and shorter bay and one corner column upto three floors
	Non-Linear Static	
(GSA, 2016)	2(DL+0.25LL) on the whole structure	Outer middle column about longer and shorter bay and one corner column upto three floors
(DoD, 2016)	2(1.2DL+0.5LL) on the adjacent bays and floors above the removed column (1.2DL+0.5LL) on the rest part of the building	Outer middle column about shorter and longer bay, at ground floor, intermediate floor and at the top floor
	Non-Linear Dynamic	
(GSA, 2016)	(DL+0.25LL) on the whole structure	Outer middle column about longer and shorter bay and one corner column upto three floors
(DoD, 2016)	(1.2DL+0.5LL) on the whole structure	Outer middle column about shorter and longer bay, at ground floor, intermediate floor and at the top floor

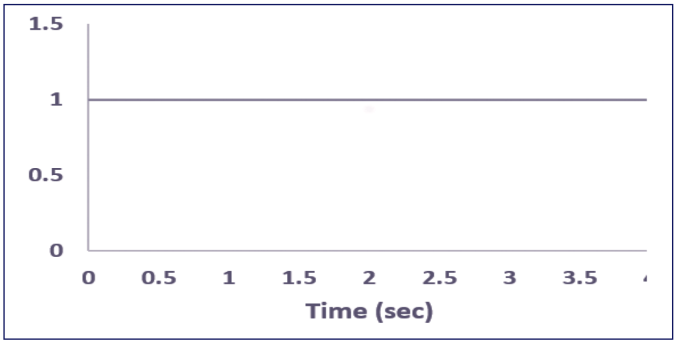
Whereas, for the non-linear cases, the amount of collapse was obtained by the chord rotation criterion (FEMA-356, 2000) as tabulated in table (3). To check the rotation, hinges are assigned in the model.

Table 3. Chord rotation capacity (FEMA-356, 2000)

State of Damage	Chord Rotation Capacity
Immediate Occupancy (IO)	$\theta_y + 10\%\ of\ (\theta_u - \theta_y)$
Life Safety (LS)	$\theta_y + 60\%\ of\ (\theta_u - \theta_y)$
Collapse Prevention (CP)	$\theta_y + 90\%\ of\ (\theta_u - \theta_y)$

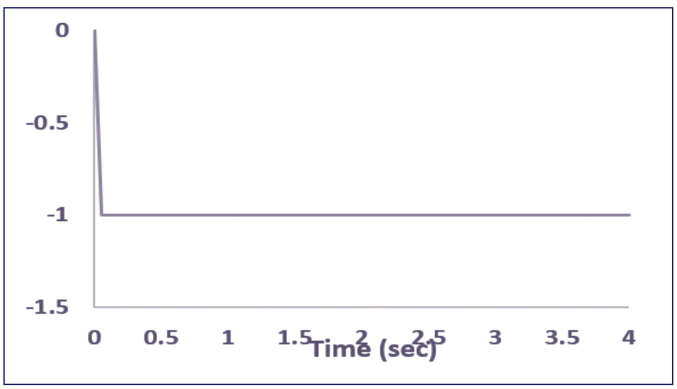
For the LDA, columns were removed from the recommended locations and the nodes above and below the removed columns were assigned with the same magnitude of forces, which were obtained as the forces due to gravity load in the structure without column removal. These assigned forces acts in the opposite direction and therefore, portray the same stable condition of the model as it was before the removal of column. Subsequently, two functions are defined to perform this analysis which will demonstrate the structural behaviour upto four seconds. First function is a constant function with magnitude 1 which begins with the initial time i.e. zero seconds and continues till four seconds as shown in Figure 2. This function is assigned to all the dead and the live loads.

Figure 2. Constant Function



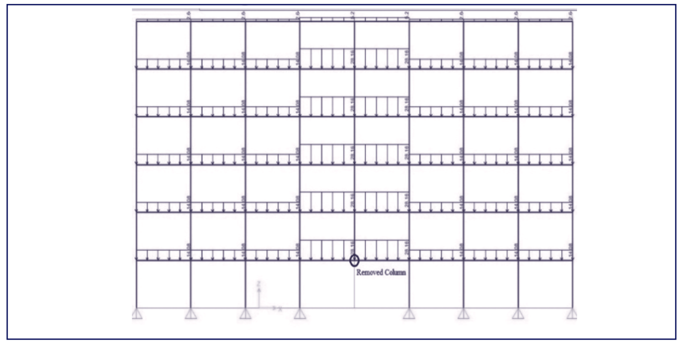
On the other hand, the second function begins with the initial time i.e. zero seconds and decreases gradually till 0.05 seconds beyond which it becomes constant upto four seconds as shown in Figure 3. This gradually decreasing function is assigned to the node(s) at the removed column locations. This function portrays gradual loss of column at the specific location after 0.05 seconds from the start time of the analysis. After completion of analysis, forces in each member were noted as demand and the DCR was calculated.

Figure 3. Gradual Function



The NLSA can be performed by two methods; displacement controlled method and load controlled method. This study uses the second method of performing NLSA by considering an immediate loss of column which begins with a zero initial condition. As per the guidelines of GSA (GSA, 2016), this analysis shall be performed considering a load case (DL+0.25LL) with a DAF on the entire structure. Therefore, the whole structure was analyzed for 2(DL+0.25LL), where 2 is the DAF and the amount of collapse was noted by referring to the rotation criteria (FEMA-356, 2000). Whereas, as per the guidelines of DoD (DoD, 2016), the NLSA shall be performed in such a way that the DAF is not applied on the entire structure and its use is restricted to the adjacent bays and floors above the removed column. Therefore, the analysis is done for two load combinations, 2(1.2DL+0.5LL) on the adjacent bays and the floors above the removed column and (1.2DL+0.5LL) on the rest of the structure. In SAP2000, it is not possible to perform an analysis by using two different load cases at the same time. Hence, to take into account the two independent scenarios, the density of the material and dead loads are increased by 20% and the live loads are reduced to 50% of the initial magnitude. Thereafter, for the adjacent bays and floors above the removed column, the DAF was used as shown in Figure 4. Subsequently, the NLSA was performed by making a new load case as (DL+LL) which accounts the DoD (DoD, 2016) guidelines.

Figure 4. Distribution of loads for NLSA following the DoD (DoD, 2016) guidelines



The approach of performing NLDA is a bit similar to that of the LDA. The forces obtained as the design forces of the members are assigned to the nodes above and below to the removed column because, these forces will keep the structure in equilibrium, as it was before the loss of column. To incorporate the dynamic effect, the gradually decreasing function explained in the LDA part and shown in Figure 2b, was assigned to column removal location. As a limitation of the software, the NLDA must began with another NLA which may be dynamic or static. Therefore, the analysis which follows the guidelines of GSA (GSA, 2016), began with a non-linear static case of (DL+0.25LL) and for DoD (DoD, 2016), it began with a non-linear static case of (1.2DL+0.5LL). The amount of collapse was noted as per the rotation criteria (FEMA-356, 2000).

RESULTS AND DISCUSSION

The model comprises of 216 columns and 354 beams which makes a total of 570 structural members. In the LDA, after calculating the DCR from Equation 1 and referring to the Figure 5, it has been found that when the corner column was removed then 12 beams which were supporting the corner side of the building just above the removed column got collapsed along with the 5 columns which were supporting those beams.

Figure 5. DCR for corner column removal at the ground floor

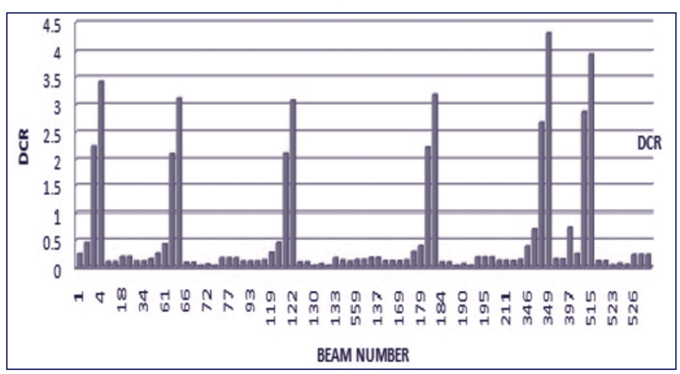


Figure 6 shows the ground floor corner column removal case and Figure 7 is for removal of middle column along the longitudinal bay. The column shown in red color is the corner column which is removed and the members which are shown in red color are the failed members whereas, the members shown in black color are safe even after the stress reversal.

The calculation process of DCR is very tedious as it requires deep attention and takes long time in noting the capacity as well as the demand of the structural members. Hence, the need of the hour is to propose an alternative of DCR with the same accuracy and precision. Bresler (Bresler, 1960) has presented a framework to plot interaction curves for columns subjected to uni-axial and bi-axial moments which states that a column is safe for every value of P-M interaction which lies within the interaction curve.

Figure 6. Collapsed members when corner column at ground floor was removed

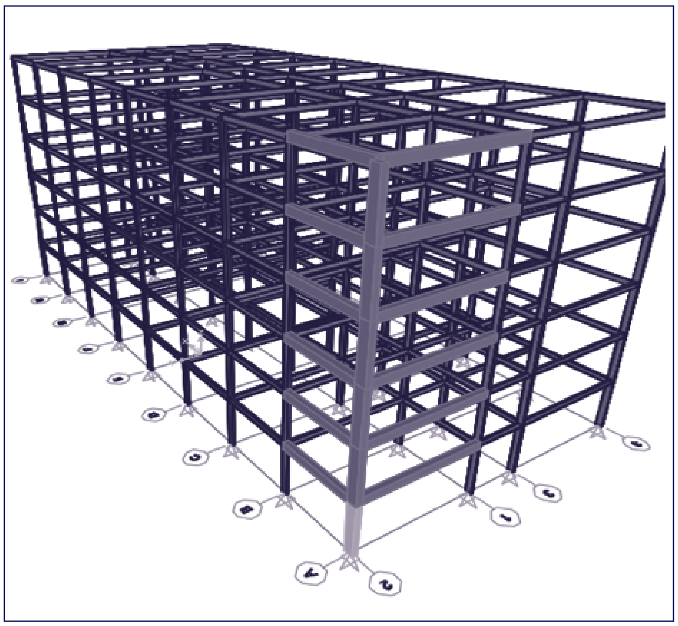
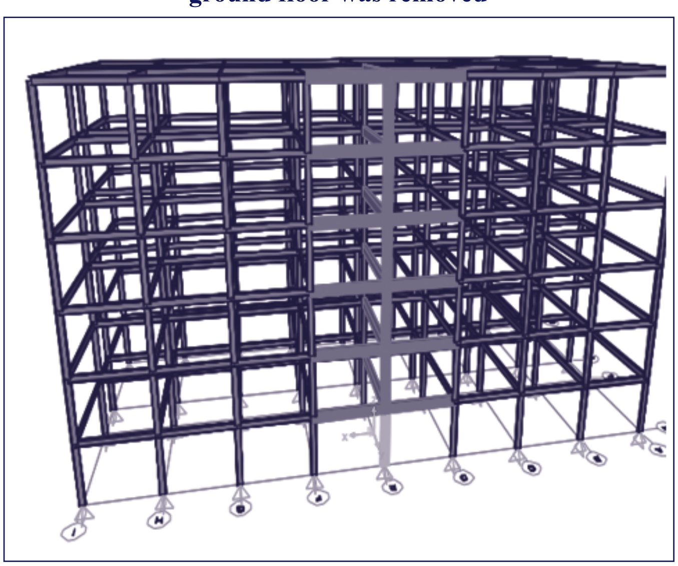


Figure 7. Collapsed members when corner column at ground floor was removed



Therefore, the interaction curves for columns of the model after column loss were plotted and the amount of collapse was observed. In Figure 8 and Figure 9, the interaction curves are shown for the ground floor corner column removal case. The curve shown in Figure 8 is for the column which is at the floor above to the removed column and the curve shown in Figure 9 is for the column at the top floor. The demand for P-M_2 in both the cases are falling outside the interaction curve whereas, the demand for P-M_3 is lying within the curve. Hence, these columns are failed in the P-M_2 interaction. These curves are plotted for each column of the model and it was found that the amount of collapse obtained from the DCR and the interaction curves came exactly the same.

Figure 8. Interaction curve for corner column at the first floor

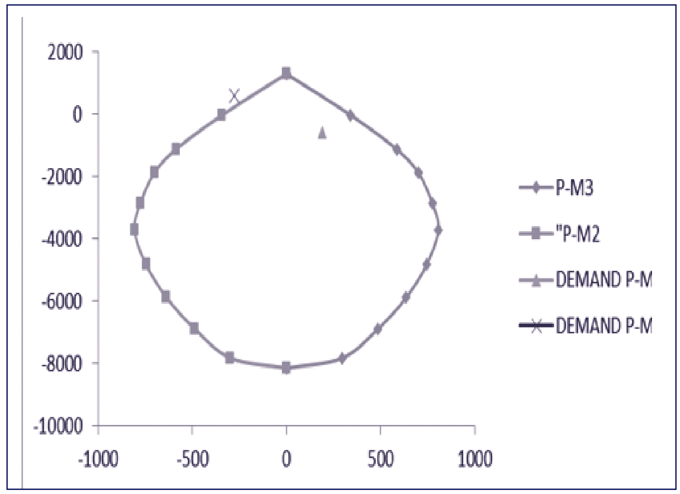
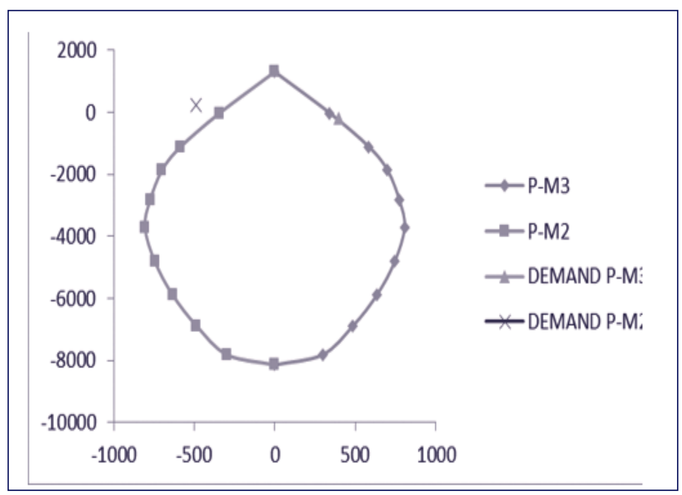


Figure 9. Interaction curve for corner column at the top floor



Therefore, finding the amount of collapse by plotting interaction curves may be considered as an alternative to the DCR calculation which is accurate and comparatively less time taking. The amount of collapse occurred in the structure from the LDA performed by following the GSA (GSA, 2016) for removal of column at different locations is summarized in Table 4.

Table 4. Results of LDA following GSA (GSA, 2016) guidelines

Column Removal Location		No. of Members Collapsed	%age of Collapse	Floor Area Collapsed (m^2)
G.F.	Corner Column	17	2.98	26
	Longer Face Column	23	4.04	52
	Shorter Face Column	6	1.10	10
F.F.	Corner Column	9	1.57	26
	Longer Face Column	12	2.11	52
	Shorter Face Column	5	0.89	10
S.F.	Corner Column	11	1.93	26
	Longer Face Column	8	1.40	52
	Shorter Face Column	4	0.70	10

The comparison between the results obtained from the NLSA and NLDA by following the guidelines of both the agencies (DoD, 2016; GSA, 2016) are presented in Table 5 and Table 6 respectively. The comparison is done only for the column removal locations at the ground floor (G.F.) and second floor (S.F.), which is the intermediate floors (I.F.), because these floors are common in both the agencies (DoD, 2016; GSA, 2016).

The results presented in Table 5 indicates that the collapsed area is same for both the standards despite of the huge difference in load case criteria. At the ground floor, when the corner column is removed, 2.98% of the total built-up area got collapsed whereas, 4.04% of the built-up area got collapsed when the middle column was removed about the longer and shorter bay. Subsequently, for the second/intermediate floor, when the corner column was removed, total collapse was 1.93% of the built-up area whereas, the collapsed area increases to 2.63% for the middle corner removal cases about the shorter and the longer bay.

Table 5. Comparison between the results of NLSA following GSA and DoD guidelines (DoD, 2016; GSA, 2016)

COLUMN REMOVAL LOCATION No. of Members Collapsed		GSA (2016) and DoD (2016)		
		%age of Collapse	Floor Area Collapsed (m^2)	
G.F.	CORNER	17	2.98	26
	LONGER FACE	23	4.04	52
	SHORTER FACE	23	4.04	36
S.F.	CORNER	11	1.93	26
	LONGER FACE	15	2.63	52
	SHORTER FACE	15	2.63	36

Likewise, the results presented in Table 6 shows the comparison of results for the NLDA. Similar to the case of NLSA, the comparison is done for the common locations of column removal. At the ground floor, when the corner column and middle column about longer bay is removed, the amount of collapse came equal following the guidelines of both the agencies (DoD, 2016; GSA, 2016). For the corner removal case at the ground floor, the collapsed area is 26 m2 and it increases to 52 m2 for the case of middle column removal about the longer bay. Whereas, at the same floor level, when the middle column about the shorter bay was removed, there was no collapse observed following the GSA (GSA, 2016) guidelines, eighteen members which were attached to the removed column or supporting the floors above are reaching to their plastic moment capacity only.

Table 6. Comparison between the results of NLDA following GSA and DoD guidelines (DoD, 2016; GSA, 2016)

COLUMN REMOVAL LOCATION No. of Members Collapsed		GSA (2016)	DoD (2016)
		No. of Members Collapsed	No. of Members Collapsed
G.F.	CORNER	17	17
	LONGER FACE	23	23
	SHORTER FACE	18 MEMERS REACHING TO PLASTIC MOMENT CAPACITY ONLY	23
S.F.	CORNER	11	11
	LONGER FACE	15	15
	SHORTER FACE	18 MEMERS REACHING TO PLASTIC MOMENT CAPACITY ONLY	15

whereas, 23 structural members collapsed for the analysis performed following the DoD (DoD, 2016) guidelines. Similarly, at the second/intermediate floor, the results obtained by removing the corner column and middle column about the longer bay are same for both the agencies (DoD, 2016; GSA, 2016) i.e. 1.93% and 2.63% respectively is the percentage of area collapsed and when the middle column about the shorter bay was removed, 12 members reaches to their plastic moment capacity by GSA (GSA, 2016) and no member was collapsed whereas, by DoD (DoD, 2016), 52 m2 area got collapsed.

CONCLUSIONS

The phenomena of progressive collapse may be trigged by any natural or manmade activity. Earthquakes, tsunamis, tornados and landslides are few amongst the natural activities whereas, fire, blast or any other type of manmade impact may initiate such phenomena of progressive collapse. Blast or any other type of strong impact has a tendency for sudden loss of a structural member whilst, majority of the natural as well as the manmade activities will account the gradual loss of the members. Hence, studies which are based on the blast or any other type of impact may be restricted to perform the NLSA only as it accounts the sudden loss of structural member(s) and those studies which are based on the other type of triggering events such as earthquakes, fire etc. which are generally responsible for gradual loss of the member(s), may perform the NLDA only.

Referring to the results obtained from the NLSA, the amount of collapse by following the guidelines of both the agencies (DoD, 2016; GSA, 2016) is coming exactly the same, even when the structure is analyzed for different load combinations i.e. 2(DL+0.25LL) and 2(1.2DL+0.5LL). Hence, this may be concluded that there is a scope to revise the DAF or the load combinations or both because even with this notable difference in the present load cases the amount of collapse was not affected. Additionally, this may also be concluded from the results of the NLDA that the guidelines of DoD (DoD, 2016) are more suitable to perform the study of NLDA as the number of collapsed members are more which makes it the worst case scenario for the modelled structure.

Furthermore, as discussed earlier, the calculation process of DCR is very tedious, hence the proposed approach of plotting interaction curves for column to check its status may be used as an alternate to the DCR calculation. The Eq. 1 consider bending alone as a parameter to calculate the DCR. As per the matter of fact that a column shall be analyzed for the combination of load and moments (P-M2 and P-M3) instead of analyzing it individually for load and for moments. Hence, using the Eq. 1 to calculate the value of DCR becomes obsolete for the case of columns and it shall only be used for the beams. The proposed approach will significantly reduce the time to perform the LDA for the reason that the number of columns will be comparatively less to the number of beams and once a column is found to be failed in P-M combination, the beams attached to it will also be considered as failed.

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