STABILITY ASSESSMENT OF STEEL MOMENT FRAMES AGAINST PROGRESSIVE COLLAPSE

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ABSTRACT

Analyzing progressive collapse plays a pivotal role in diagnosing structure stability caused by earthquake, explosion, car crashes, and fire and so on. In the current research, condition of progressive collapse has been analyzed in structures which have been designed based on current codes in Iran. For this purpose, a couple of steel buildings with steel moment frame systems that have many stories and various bay, have been evaluated. With sudden removal of each column, the possibility of bridging over other elements has been studied; besides, alternative path method introduced by the UFC 4-023-03 code has been applied in the assessment process. Results indicate that the beams located on the highest floor do not have a suitable performance and in case of sudden removal of each of the columns, it will be impossible to bridge over other elements and it will face collapse and progressive collapse in spite of the fact that other members are resistant against the collapse.

KEYWORDS:

Progressive Collapse, Non-linear Time History Analysis, Moment Resisting Frames.

1. INTRODUCTION

Progressive collapse means gradual destruction of a part of a construction resulting from uncommon damage and expansion of this destruction to other parts of the construction. The damage can be caused by an explosion, earthquake, being hit by a vehicle or a sudden collapse etc. The damage is often applied to the structure dynamically and during a short time period. After the destruction of Ronand's Building and engineers' focus on progressive collapse, a wave of research on protective methods or reducing the structure's potential against progressive collapse started. In the beginning the result was in the form of some changes in codes; however, after a few years when a couple of similar happenings occurred, such as September 11 terrorist attacks, separate codes were set to reduce or protect destruction. Two of these codes which deal with progressive collapse separately are Department of Defense (DoD) and General Service Administration(GSA). The approach of these codes is as follows:

If one of the members of the main structure is destroyed suddenly, the rest of the members are able to bridge over other elements and have an alternative path to transfer the load[1 and 2]. All structures were designed in accordance with the steel design codes of Iran [3 and 4]. It is to be noted that the Iranian codes are generally similar to UBC 97 though the load and resistance factors are slightly different. DoD code in particular deals with controlling progressive collapse in the form of a series of UFC codes, which is the basis of the current paper[1].

Kim and Kim[5],have studied the performance of steel moment resisting frame (SMRF) against progressive collapse. In this paper which has been conducted on three steel moment resisting frame the performance of the structures has been an evaluated using DoD2005 and GSA2003 codes. Also, structures have been analyzed using these three analyses: Linear Static (LS), Linear Dynamic (LD), and Nonlinear Dynamic (ND).The results of the survey are the indicative of the fact that the condition of corner columns compared to similar columns is very week and enjoys a high potential to expand destruction. The cause of such a problem is redundancy of extra elements to bridge the load. In addition, as the number of floors increases the probability of progressive collapse in lower floors decreases. Although analyzing the structure using Linear Static is simpler, in some cases it is more conservative. It should also be mentioned that the results of nonlinear dynamic analysis depends on different factors such as: materials, position of the columns, load, number of stories etc. Khandelwal and Tawil[6], have run a search on measuring the structure resistance using Pushdown method. In this approach the structure is evaluated in three different modes as follows:

Uniform Pushdown
Span Pushdown
Incremental Dynamic Pushdown

The results of the paper are the indicative of the fact that GSA code method is more conservative than DoD. Liu [7]worked on optimized method of SMRF against progressive collapse. In his paper each of the structures, regarding the genetic algorithm, were optimized in normal disposition (NWD), Linear Static (PCLS), Nonlinear Static (PCNS), Nonlinear Dynamic (PCND) and in the end the weight of the structures were compared. After redesigning the structures, in the optimized mode the amount of needed profile weight increase in three dispositions [(PCLS), (PCNS) and (PCND)] were calculated as follows: 38.8%, 13.2%, and 8.2%. In addition, the present progressive collapse design optimization successfully produces a seismic IMF design that satisfactorily meets the UFC alternate path criteria by enhancing load redistribution capability through appropriate member sizing. Meanwhile, the cost of constructing this frame is considerably reduced through minimization of the overall steel weight. As it is obvious from the results it was stated that designing with the use of linear static is not economical.

2. Analyzing Methods of Progressive Collapse

In order to study the building's structure against progressive collapse using the alternative path method in DoD code, there are three analyses (LS, NS, and ND) to control the behavior of main members such as beams and columns which are destroyed suddenly.

2.1. Linear Static Approach

In this method which is considered the simplest one, to check the members against progressive collapse, to rectify the effects resulting from member geometry and dynamic load in gravity loading of upper members of removed column a factor called mLIF is applied. Regarding the type of structure whether steel, concrete etc, and also the kind of joints, a factor called magnifying coefficient (m) is extracted. It is necessary to mention that mLIF is the smallest magnifying coefficient (m) of elements which are joined to the beam. In the current research, the amount of m has been taken into account regarding the improved rigid joint. The loading coefficient (Ω) is calculated concerning the mLIF. This factor is applied to combine loading in the area above the removed column. Increased gravity loads for floor areas above removed column or wall is $GLD=\Omega LD[(0.9 \text{ or } 1.2) D+ (0.5L \text{ or } 0.2S)]$ and gravity loads for floor areas away from removed column or Wall is G = (0.9 or 1.2) D + (0.5 L or 0.2 S).In Figure 1 you can see how the load is applied. In these equations, D is Dead load including façade loads; L is Live load and S is Snow load.

After applying the new load on the structure, 0.002∑P is applied in each design as a lateral load on each side of the structure. ∑P is the sum of live and dead loads without coefficient on each story. To assess the resistance of members, the ratio of member capacity, which is determined through dividing current capacity by demand capacity ratio of the member's section (DCR), is calculated. If the calculated DCR is bigger than mLIF, that member is considered vulnerable and it should be either redesigned or reinforced.

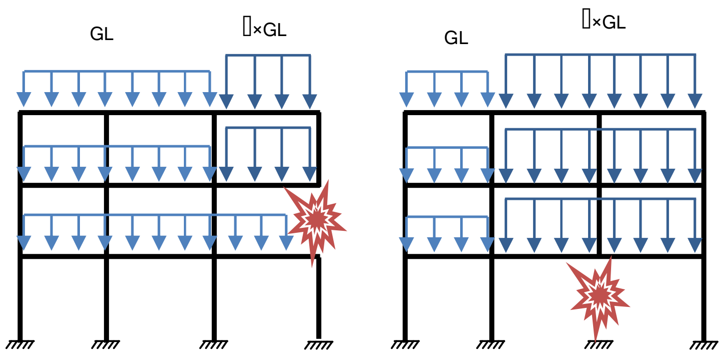

Figure1: Loading application process in DoD code

2.2. Nonlinear Static Approach

In Nonlinear Static Method (NS) the factor which results from dynamic loading effect is added to the gravity loading of the removed column's upper members. This coefficient is calculated and extracted based on the defined tables in code regarding the type of structure as well as the type of joints. To calculate the deformation-controlled and force-controlled actions, simultaneously apply the following combination of gravity and lateral loads:

Increased gravity loads for floor areas above removed column or wall is GN = ΩN [(0.9 or 1.2) D + (0.5L or 0.2S)] and Gravity Loads for Floor Areas Away From Removed Column or Wall is G = (0.9 or 1.2) D + (0.5L or 0.2S). After applying the new load on the structure, 0.002∑P is applied in each design as a lateral load on each side of the structure. ∑P is the sum of live and dead loads without coefficient on each story. Now after removing the column, if the defined joints in members pass the determined criterion area that member is considered vulnerable and it should be either redesigned or reinforced.

2.3. Nonlinear Dynamic Approach

In Nonlinear Dynamic Method which includes the real behavior of the structure to evaluate members first the existing interior forces in the node of the removed column are calculated. Then the mentioned column is removed from the numerical modeling and analogous reactions in the mentioned node are applied. To calculate displacement and specifying the condition of plastic joints of the members' elements, the incoming reactions are removed from the structure in a form of an impact load. It is necessary to mention that the time period of removing the load is time history algorithm; moreover, its duration according to the DoD code Equals to one tenth of the period associated with the structural response mode for the vertical motion of the bays above the removed column(Figure2).

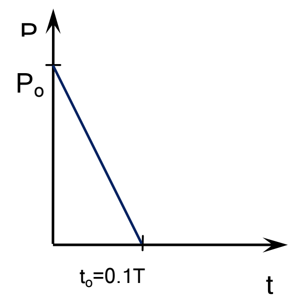

Figure2: Speed of unloading in Nonlinear Dynamic Approach

3. RESEARCH METHOD

In this paper, different scenarios of removing the column are based on the introduced locations in codes which behaviorally speaking is divided into three groups: Middle Column (MC), Edge Column (EC) and Corner Column (CC). Figure3, shows the location of each group in the plan and Figure4, depicts the location of the height of destruction. To control the accuracy of performed analyses, we have used the comparison of the solved example in DoD2009regulation. In this example, a 3-floor steel structure was evaluated under definite loading based on linear static analysis whose destruction points have been shown in Figure5. Regarding the section specifications, first the factor (m) and then mLIF were compared. It needs to be mentioned that to assess this example the code has used Sap2000 software[8].After analysis, the amounts of created DCR's and plastic joints in the introduced structure in the code were compared with that of modeled structure and the difference is very small.

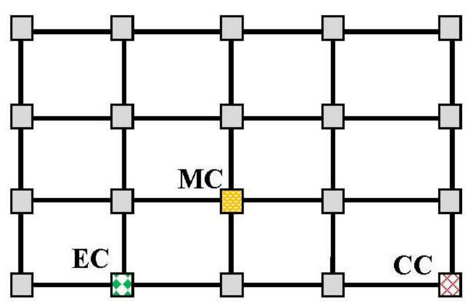

Figure 3: Categorization of column destruction position in stories plan

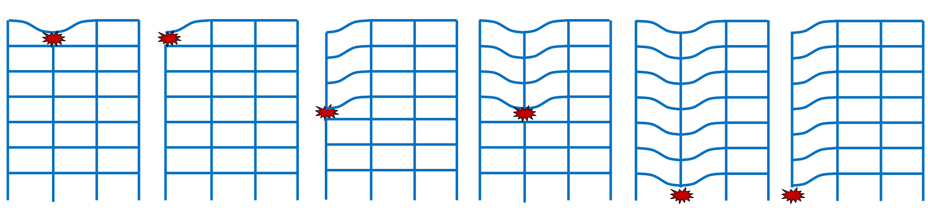

Figure 4: Categorization of column destruction position in height

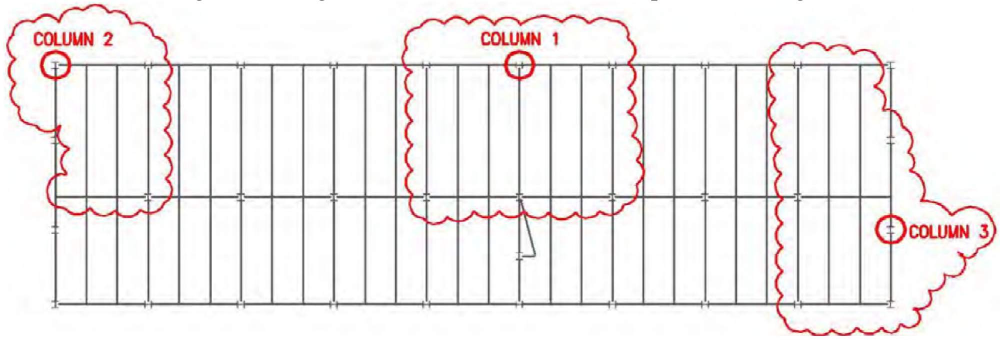

Figure 5: Destruction of the intended columns in DoD code example

3.1. The Studied Models

In this paper, 24 structure models in Ordinary MRF with following specifications have been studied:
Number of stories: 3,5,7 and 10
Number of Spans: 3 and 5
Various span length,4, 5 and 6 meters

The height of stories in all models is the same and it Equals 3.4 meters. The gravity loading for all the constructions have been considered 600 kg per square meters on floors and live load has been considered based on the residential use. The roof dead load the same as the floor dead load and its live load (Snow load) was considered 150 kg per square meters. The SAP2000 is used as a computational tool for investigating the load redistribution behavior within the damaged steel frame after a column is removed.

The steel material has yield strength of 345 MPa and a tensile strength of 450 MPa. All in all, 216 destruction conditions for each of the studied situations were evaluated. Moment frame specifications have been shown in tables 1 to 3.

4. The Evaluation Procedure

In this paper, we have used SAP2000-ver14 software to analyze the progressive collapse. Because of the requirement of the DoD code, all the analyses have been done employ a three-dimensional assembly of elements and components. The defined plastic joints are in accordance with tables 5-6 of ASCE 41 code[9]. For each of the Nonlinear Dynamic (ND), Nonlinear Static (NS) and Linear Static (LS) analyses, a combination of presented loads in DoD code have been applied. The type joints of this evaluation are improved welded unreinforced flange with bolted web (Figure5).

Table1: Specifications of structural members (span length 6 meters)

Length Beam 6 m	3 Story		5 Story		7 Story		10 Story	
	Beam (IPE)	Column (Box)	Beam (IPE)	Column (Box)	Beam (IPE)	Column (Box)	Beam (IPE)	Column (Box)
Story1	360	35-1.2	450	45-1.5	500	45-2	550	55-2
Story2	360	30-1	450	45-1.5	500	45-1.5	550	50-2
Story3	300	25-1	450	40-1.2	500	45-1.5	550	50-2
Story4			400	35-1.2	500	40-1.2	550	45-2

Story5			300	30-1	450	40-1.2	550	45-2
Story6					400	35-1.2	550	45-1.5
Story7					300	30-1	450	45-1.5
Story8							450	40-1.2
Story9							400	40-1.2
Story10							360	35-1.2

5. DISCUSSION

After analyzing the fore mentioned structures for each of (ND), (NS) and (LS) analyses, besides calculation of subsidence, vibration period, study of DCR's, etc, also comparison of structure's response to each other, we can assess the steel moment resisting frame against Progressive Collapse.

Table2: Specifications of structural members (span length 5 meters)

Length Beam 5 m	3 Story		5 Story		7 Story		10 Story	
	Beam (IPE)	Column (Box)	Beam (IPE)	Column (Box)	Beam (IPE)	Column (Box)	Beam (IPE)	Column (Box)
Story1	330	30-1	360	45-1.5	450	45-1.5	500	50-2
Story2	330	30-1	360	45-1.5	450	40-1.2	500	45-2
Story3	240	25-1	360	40-1.2	450	40-1.2	500	45-2
Story4			330	35-1.2	450	35-1.2	500	45-1.5
Story5			270	30-1	400	35-1.2	500	45-1.5
Story6					330	30-1	450	40-1.2
Story7					270	25-1	450	40-1.2
Story8							400	35-1.2
Story9							360	35-1.2
Story10							270	30-1

Table3: Specifications of structural members (span length 4 meters)

Length Beam4 m	3 Story		5 Story		7 Story		10 Story	
	Beam (IPE)	Column (Box)	Beam (IPE)	Column (Box)	Beam (IPE)	Column (Box)	Beam (IPE)	Column (Box)
Story1	270	30-1	330	40-1.2	360	40-1.2	450	45-2
Story2	240	30-1	330	35-1.2	360	40-1.2	450	45-1.5
Story3	220	25-1	330	30-1	360	35-1.2	450	45-1.5
Story4			300	30-1	360	35-1.2	450	40-1.2
Story5			240	25-1	330	30-1	450	40-1.2
Story6					300	30-1	450	35-1.2
Story7					240	25-1	400	35-1.2
Story8							360	35-1.2
Story9							300	35-1.2
Story10							270	30-1

5.1. Situation of beams

After analyzing the structures' response in three modes [Nonlinear Dynamic (ND), Nonlinear Static (NS) and Linear Static (LS)], the results indicates that in all the models, including 3, 5, 7 and 10-story buildings, the beams which have been located on the highest floor do not have a suitable performance because they become unstable quickly when the column is removed, which can due to unclear redundancy in the last beam, particularly lack of upper column. In addition, all the studied areas' beams are capable of bridging over other members, except for the beams located on the highest floor that have been tested with Nonlinear Dynamic (ND), Nonlinear Static

(NS) method. It is necessary to mention that the results of Linear Static (NS) analysis concerning the floor beams shows the percentage of destroyed beams compared to total number of upper beams in the removed column area (Nbf/Nbt) increases as the proportion of span length (L) to beam's height (d) goes up. Nbf is the percentage of destroyed beams in the upper area of removed column and Nbt is the total number of upper beams in the removed column area. Generally speaking, if the proportion of L/d does not exceed 9 (L/d<9) the destruction potential in linear static method will be zero. In Figure6, the average percentage of destroyed beams has been checked in different positions.

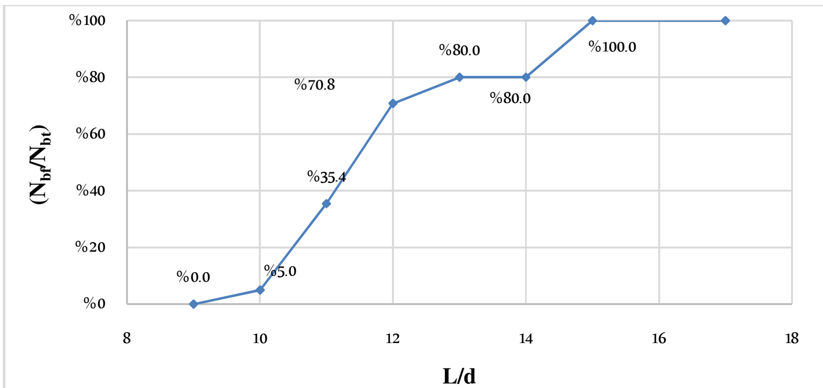

Figure6: Situation of beam's destruction to the total number of beams in the removed column area in proportion of length to beam's height

5.2. Situation of columns

After checking the models in different destruction modes and type of analysis, it got clear that in all the controlled areas (MC,EC and CC) with nonlinear dynamic (ND), nonlinear static (NS) methods the columns have the capability of transferring loads over other elements. In linear static method it was made clear that as the proportion of L/d increases, in particular more than 15 (L/d>15), the upper column in the destruction place in the corner of building is not capable of transferring load. Also, as the proportion of L/d exceeds 16, in the edge columns of the structure the DCR of the upper column of the destroyed place exceeds mLIF while in all the interior columns of the building DCR is less than mLIF. Table 4 illustrates the comparison of column resistance against progressive collapse regarding the destruction position and the performed analysis.

5.3. Comparison of subsidence in different destruction position in various analyses

After analyzing the structures with nonlinear static and linear static methods in different positions (MC, EC and CC) the subsidence of upper node of each structure was extracted. In Figures 7 to 9, subsidence variance of LS to ND and also subsidence variance of NS to ND in proportion to both length and height of the beam (L/d) was drawn for different positions. Checking the graphs, the following conclusions can be drawn:

The Vertical Displacement of upper node in removed column place in the studied structures is in LS, ND and NS, respectively. In other words: $\Delta LS > \Delta ND > \Delta NS$.

In regular steel structures, the lateral load response ($0.002\sum P$) in various directions is almost the same and the difference is marginal.

After checking the node subsidence of the destruction place, it was made clear that the displacement resulting from linear static analysis is 40% more than nonlinear dynamic analysis. Needless to say, this amount declines as the indeterminacy degree increases. Besides, the amount of node subsidence in the destruction place in NS method is less than ND method and it equals 30%.

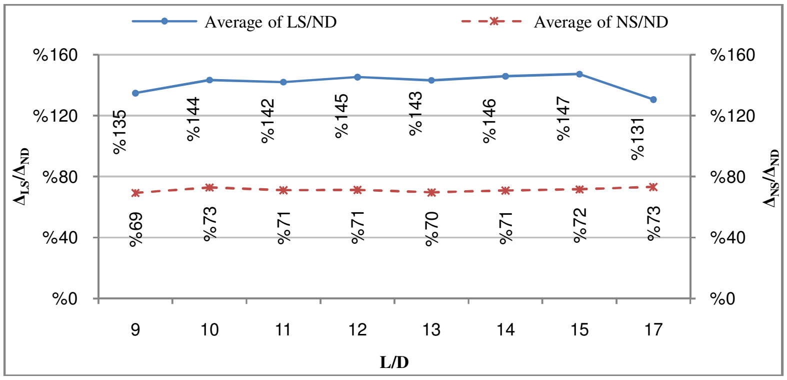

Figure 7: Comparison of proportion of node vertical displacement in removed column place in NS and LS modes to ND in CC position

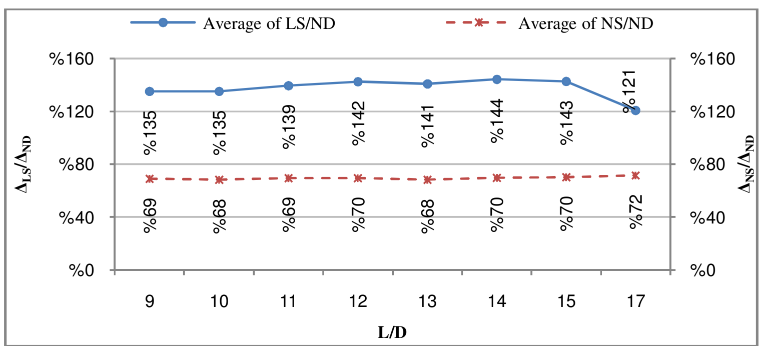

Figure 8: Comparison of proportion of node Vertical Displacement in removed column place in NS and LS modes to ND in EC position

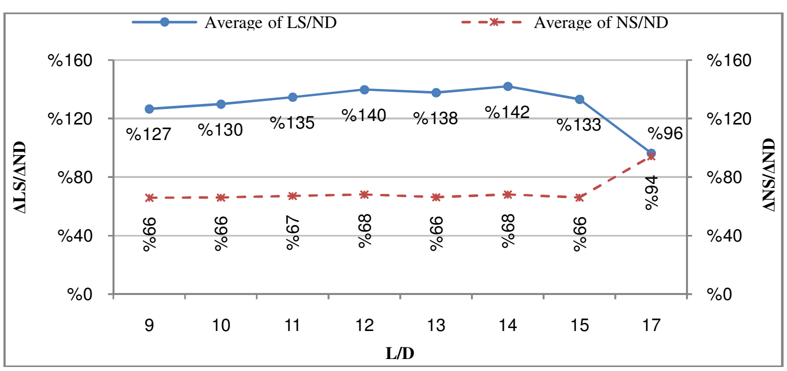

Figure 9: Comparison of proportion of node Vertical Displacement in removed column place in NS and LS modes to ND in MF position

6. CONCLUSION

Regarding the carried out research on steel structures designed based on Iranian codes as well as their evaluations against progressive collapse with alternative path approach suggested by DoD, the following results were achieved:

Concerning the performed studies, it was made clear that in SMRF buildings designed based on IBC code, the last floor's beam are not capable of bridging over other members and are prone to progressive collapse.

Except for the roof beams, all the controlled places – EC, CC and MC – in ordinary moment resistance frame, if the proportion of evaluated area to the ratio of beam's length to height is less than 17 (L/d<17), it is resistant against progressive collapse in nonlinear dynamic and nonlinear static analyses. It is necessary to mention that in linear static approach the response of the structure has been like this: as the proportion of length to beam's bay (L/d) declines, the elements, destruction decreases; in other words, in areas where L/d<9, the destruction potential tends to be zero.

In nonlinear dynamic and nonlinear static analyses, the upper column and the columns around the destruction do not have the potential of progressive collapse; as a result, in linear static analysis, the upper column of the destruction in CC area has a more suitable situation compared to EC and MC.

The introduced linear static approach in DoD code is approximately 40% more conservative than nonlinear dynamic method.

Inanalyzing the structures designed based on linear static approach the amount of ΔLS is approximately 40% more ΔND. It obvious that, as the proportion of length to beam's bay exceeds 15 (L/d>15) or as indeterminacy degrees increase, the mentioned amount goes up.

In analyzing the structures designed based on nonlinear static approach the amount of ΔNS is approximately 30% more ΔND. It is clear that, as the proportion of length to beam's bay exceeds 15 (L/d>15) or as indeterminacy degrees increase, the mentioned amount goes up.

In regular steel structures, the lateralload response ($0.002\sum P$) in various directions is almost the same and their difference is marginal. This issue is clear in all various models.

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