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Dynamic Increase Factor for Progressive Collapse Analysis due to Sudden Column Removal

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Abstract
Assessment of progressive collapse potential of structures can be carried out through alternate path approach. In the alternate path approach, load bearing structure is suddenly removed and the ability of other member to withstand the added forces are examined. Linear static, nonlinear static and nonlinear dynamic analysis can be employed for determination of the structural response during progressive collapse analysis. In the dynamic analysis, the inertial forces are directly considered through the equations of motion. But in nonlinear static method, a Dynamic Increase Factor (DIF) is introduced for modifications of dead and live loads due to the unaccounted inertial effects. The Unified Facilities Criteria (UFC) and General Service Administration (GSA) guidelines initially adopted Dynamic Increase Factor of two. Latter both guidelines use Dynamic Increase Factor (DIF) based on plastic rotation capability that the damaged frame experiences. In this study, variation of dynamic increase factor with actual level of inelasticity is found out. For this purpose, three to seven storey symmetrical reinforced concrete building of varying bay sizes designed as per Indian Standard are analysed. The effect of sesmic design level on Dynamic Increase Factor (DIF) is also found out.

Keywords
Progressive collapse, Dynamic Increase Factor, RC structure, Column removal, SAP2000

1. Introduction

After the collapse of the World Trade Center towers in 2001, Progressive collapse has been an increasing concern in the structural engineering community. Progressive collapse of existing building is initiated by the sudden failure of one or more of its major load bearing elements, typically columns or walls, followed by redistribution of the loads and failure of the next elements in the vicinity in a chain-like reaction until the failure of the whole building. The cause of the phenomenon may be a result of one specific event or a combination of causes that lead to local failure like vehicular impact, earthquakes, fire, explosions as well as human error in design or construction of the structure. A typical example of this would be the intentional removal of a column by an explosion. The structural components of the floors above this column would experience a sudden increase in stress as well as large deflections. This amplification of the load may continue to cause failure in other primary members of the structure until the building stabilizes with noticeable deformations or until the complete collapse of the structure.

The General Services Administration, "Progressive Collapse Analysis and Design Guidelines"[1] and The Department of Defense Unified Facilities Criteria 4-023-03 "Design of Buildings to Resist Progressive Collapse"[2] are the available guidelines for the progressive collapse analysis. Both the guidelines follow alternate load path method where the analysis of structure is carried out to determine collapse potential after the removal of load bearing elements. Different analytical procedures like linear static, nonlinear static and nonlinear dynamic method are employed for determination of the structural response during progressive collapse. Among these method, nonlinear dynamic analysis procedure gives accurate and better results but is more complicated, tedious and time consuming[3]. Due to that reason simple, less tedious static analysis is being carried out with certain magnification factors that accounts for dynamic and nonlinear effects so that the final responses are more similar and exact to dynamic analysis. Load Increase Factor (LIF) used in linear static method accounts inertial and nonlinear effects whereas Dynamic

Increase Factor (DIF) used in nonlinear static procedure accounts only for inertial effects.

Initially the General Services Administration, “Progressive Collapse Analysis and Design Guidelines” and The Department of Defense Unified Facilities Criteria 4-023-03 “Design of Buildings to Resist Progressive Collapse” both adopted the Dynamic Increase Factor of 2 on analysis[4, 5]. Latter this value is modified in both guidelines and adopted the same formula which is based on material properties of affected structural members only. It isn’t based on the actual plastic deformation level that the a damaged frame experiences. It is possible that a frame can still remain elastic even after a certain column has been removed which is particularly true for the frames that were originally designed to withstand large lateral loads such as those from earthquakes and winds. Such frame have significantly extra capacity against gravity induced progressive collapse. Also even after the damaged frame enters in inelastic range, the actual level of inelasticity is not necessarily so high that the plastic rotation of the controlling beam reaches maximum allowable plastic hinge rotation. So the variation of DIF with gravity loading and structural capacity in terms of Mu/My is found out in this paper which can be useful for DIF formulation.

2. Analytical Models

Three-dimensional reinforced concrete buildings are modeled and analyzed using finite element program SAP2000. The table 1 shows the parameters of building taken in this study.

Table 1: parameters of building

Storey of buildings	3,4,5,6,7
Height of each floor	3m
Size of bay	3m,4m,5m,6m

All together 20 symmetrical reinforced concrete building are taken. The building are seismically designed using Indian Standard code[6]. The beam and column sizes are different along with their reinforcement details.

During the analysis, beams and columns are modeled with two noded frame elements as shown in figure 1. The beam and column connection are moment resistant. The columns are stronger than beam so that the plastic hinges will form on the body of beam only. Secondary members aren’t included. The connection at the foundation are modeled as fixed connection.

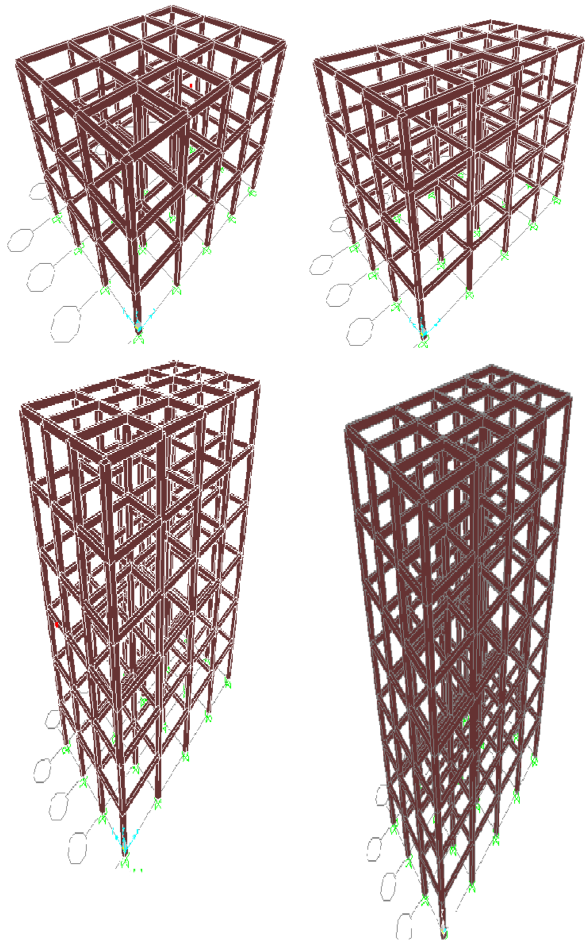

Figure 1: Three dimensional model of building

All beams are confined by shear reinforcement adequately so that beams are not shear controlled and hinges are governed by flexural stress only. The lump nonlinearity is included by assigning plastic hinges at the ends and midspan of every beam elements. The generalized force displacement curve assigned for each hinge is shown in figure 2. Point A is always origin. B represents yielding, Point C represents the ultimate capacity. Point D represents a residual strength and Point E represents total failure. The nonlinear modeling parameters a, b, c and acceptance criteria are taken from Table 4-1 of UFC 4-023-03[2] depending on the structural configuration, shear demand, and reinforcement ratios. The hinge definition is designed to allow strain hardening of 5% at the point expected to be the maximum allowed rotation which is different from 10% hardening used in ASCE 41[7]. Geometric nonlinearity such as P-delta effect is also included in analysis. Columns are assumed to have adequate strength to resist additional load redistribution after loss of the primary column.

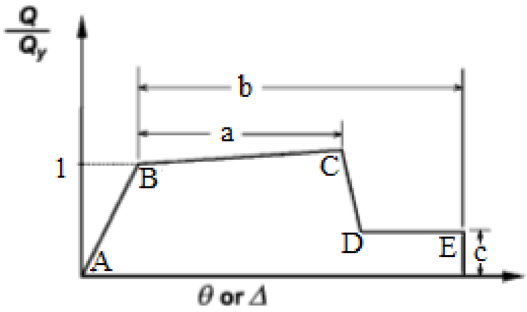

Figure 2: Force-Deformation Relation[7]

The modulus of elasticity and compressive strength of concrete is taken as 22360 Mpa and 20 Mpa respectively. The yield strength of reinforcement is taken as 500 Mpa.

The ASCE 7 extreme-event load case 1.2DL+0.5LL is used for all analysis[8]. .The structural loading applied are taken as listed below.
Live load= $3KN/m^2$
Roof load= $1.5KN/m^2$
Wall load= $10KN/m$
Slab dead load= $3.125kN/m^2$

The dead load of beam and column will be taken automatically by SAP2000. The area load such as slab dead load, floor finish, floor live load and roof live load are transferred into the corresponding beam as per tributary area.

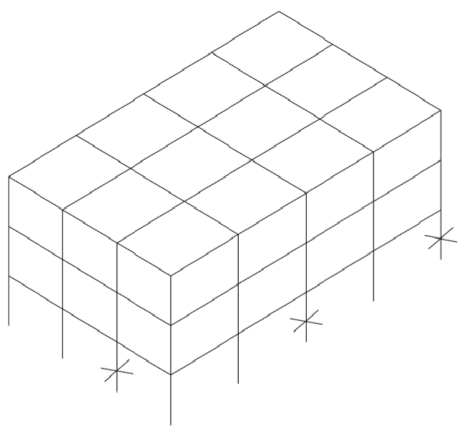

Figure 3: location of column removal

For each building, the corner column, external column near middle of long side and short side in first storey are removed one at a time as shown in figure 3. Hence all together there are 60 cases for 20 models.

In nonlinear dynamic analysis, the simulation of instantaneous removal of column is done by replacing the column with equivalent reaction[9]. For the determination of equivalent reaction, a linear static analysis is performed first using the ASCE 07[8] extreme load case 1.2DL+0.5LL and internal forces at the top joint of the column to be removed are calculated. Then, the column is removed from the model and the calculated forces are applied at the column joint in opposite direction as a reaction. After the column has been substituted with reaction forces, a new linear static analysis is again performed and the resulting flexural moments diagrams and deflections are compared with results obtained from the initial linear static analysis that included the column as shown in figure 4 . If both linear static analysis with the column and with substitute reactions resulted in identical moment diagrams and deflections, then that reaction is taken as equivalent reaction for column removal.

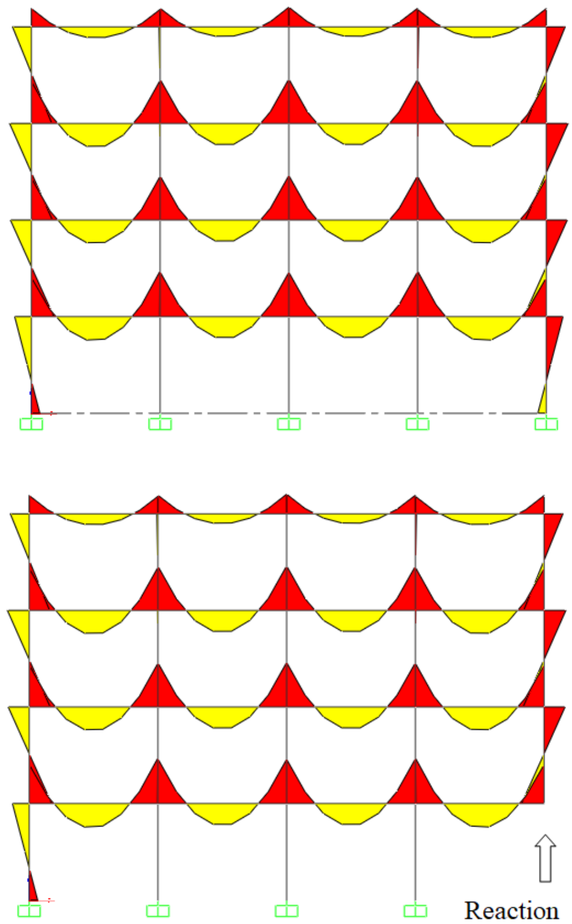

Figure 4: comparision of bending moment diagram with column and without column with substituate reactions

In this way the column is successfully replaced by equivalent superimposed reaction forces obtained from a static analysis of building using extreme event load case applied to entire structure. The reaction obtained is then removed over time as shown figure 5.

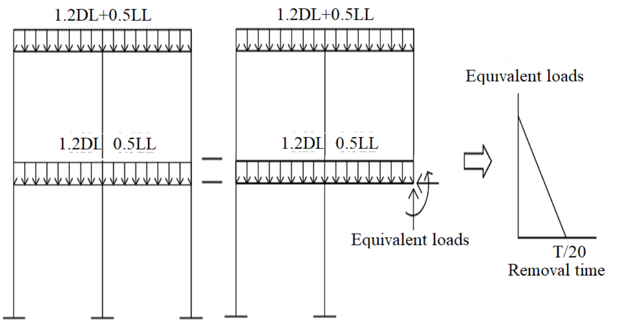

Figure 5: Transition of structural model for column removal

For dynamic analysis, following parameters are taken[9].
Damping ratio = 1%
Column removal time = T/20
Analysis Time Step = T/200
The period (T) used for the column removal duration is the period of the first mode to exhibit vertical motion at the location of the removed column after the column has been removed.

For Nonlinear Static Analysis, Non-linear staged construction feature in SAP2000 is used to simulate the column removal[10]. Three stages using 100 steps per stage is used in analysis. In first stage, all the floor are loaded with ASCE extreme loading.In second stage, the additional load as per trial DIF are loaded over the bays around the column removal only. In third stage, column is finally removed .

3. Procedure for calculation of DIF

The procedure for the calculation of DIF consists of following steps as shown in figure 6.

1. Obtain maximum plastic hinge rotation and vertical displacement among all the beams of the bay affected by column removal location by conducting NLD analysis without amplified extreme event load case
2. Conduct NLS analysis with the same model with trial DIF applied to extreme event load case only on the bay around loss location. The process is repeated untill the maximum plastic hinge rotation or maximum vertical displacement at the column location matched with NLD analysis response.

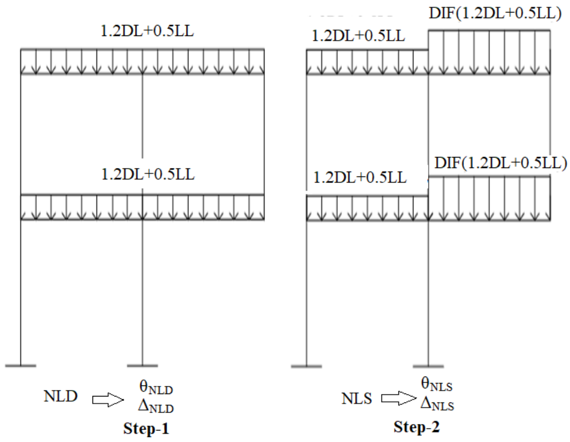

Figure 6: Procedure for determination of DIF

3. For the same model, conduct NLS analysis without amplified extreme event load case to determine maximum ratio of moment demand (Mu) to yield moment capacity (My) of beams around loss location[11]. Finally DIF value for corresponding Mu/My is obtained.

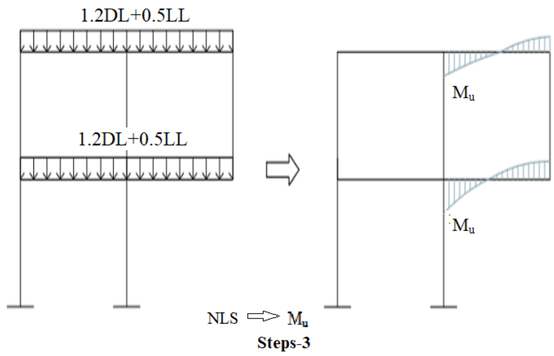

Figure 7: Procedure for determination of Mu

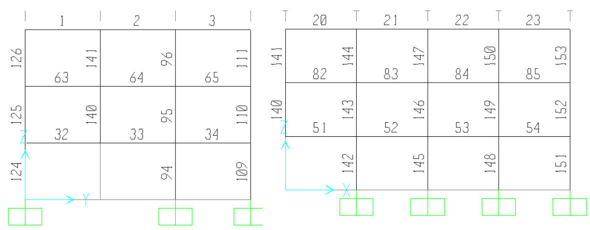

Figure 8: Beam and column notation

4. Results and discussion

The tables 2,3 below shows DIF calculation for external column removal near centre of shorter side in three storey RC building of 4m bay spacing building shown in figure 8. For each model with different location of column removal, trial amplification factors are applied to the load in static analysis to match deformation level obtained from dynamic analysis.

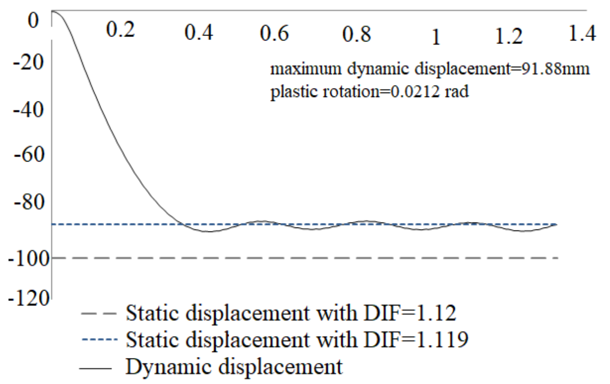

Figure 9: Displacement at column removal location

Table 2: Response from dynamic analysis

storey	Beam	dis(mm)	rot(rad)
1	32	91.88	0.0201
1	33	91.88	0.0212
1	51	91.88	0.0167
2	63	91.88	0.0187
2	64	91.88	0.0197
2	82	91.88	0.0181
3	1	91.88	0.0125
3	2	91.88	0.0141
3	20	91.88	0.0199

Table 3: Response from static analysis

Beam	dis(mm) DIF = 1.12	rot(rad) DIF = 1.12	dis(mm) DIF = 1.119	rot(rad) DIF = 1.119
32	103	0023	89.12	0.0193
33	103	0.0239	89.12	0.0197
51	103	0.0197	89.12	0.0166
63	103	0.0216	89.12	0.0179
64	103	0.0214	89.12	0.0185
82	103	0.0219	89.12	0183
1	103	0.0129	89.12	0.0123
2	103	0.0162	89.12	0.0135
20	103	0.0234	89.12	0.0195

Table 4: calculation of M_u/M_y

	M_u^+	M_u^-	M_y^+	M_y^-	$\frac{M_u}{M_y}^+$	$\frac{M_u}{M_y}^-$
32	78.46	126.06	81.86	142.40	0.96	0.89
33	80.77	135.83	81.86	142.40	0.99	0.95
51	42.80	130.68	81.86	142.40	0.52	0.92
63	78.27	129.98	81.86	142.40	0.96	0.91
64	80.59	138.34	81.86	142.40	0.98	0.97
82	49.68	130.84	81.86	142.40	0.61	0.92
1	56.37	82.38	57.16	97.35	0.99	0.85
2	56.71	92.99	57.16	97.35	0.99	0.96
20	46.18	95.09	57.16	97.35	0.81	0.98

From figure 9, it is observed that the maximum displacement above the column removal location is 91.88 mm and corresponding maximum plastic rotation of beams nearer or above the column removal location is 0.212 radian which are obtained from nonlinear dynamic analysis.To match response of static analysis with the dynamic analayis, the load on static analysis are amplified with trial factor starting from 1 and found out that the response are more similar when the amplification factor is 1.119 with only 2% error. Hence DIF value of 1.119 at that column removal location estimates the dynamic effects due to instantaneous removal of column at that location. Finally the DIF value 1.119 is plotted in figure 10 for max Mu/My ratio of 0.99 which is obtained in table 4 .

In the similar way, building with variable bay spacing, column removal location and seismic design level are analysed and corresponding DIF with Mu/My are plotted as shown in figure 10.

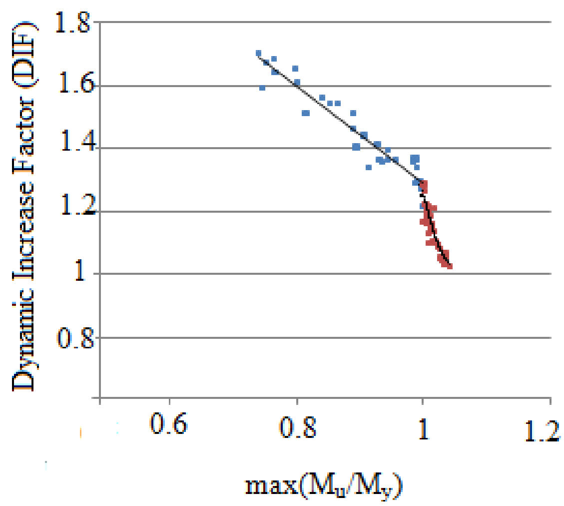

Figure 10: Variation of DIF with max(Mu/My)

From figure 10, it is seen that DIF generally decreases with increase in level of nonlinearity. Two distinct trendlines are clearly noticed. The trendline's slope is gentle when the value of max(Mu/My) is less than 1 and the slope is steep after max(Mu/My) is more than 1. The value of max(Mu/My) less than 1 indicates that the damaged structure is in elastic stage before applying the DIF. However the response of dynamic analysis will situate in inelastic stage. To drive the structure from elastic to certain point in inelastic stage from lower value of max(Mu/My), more gravity load is required in static analysis leading to the higher value of DIF. Hence DIF decreases when max(Mu/My) increases. Similarly max(Mu/My) more than 1 represents the damaged structure is in post yield stage before the application of DIF. So small load is enough for the structure to reach in final inelastic stage response of dynamic analysis. Due to that reasons the value of DIF is smaller for latter case than the earlier. The steep slope is due to the presence of high level of nonlinearity and ductility of structure in post-yield stage which is consistent with the belief that structures with large deformation capacity will withstand less dynamic effect. To study the effect of seismic design level on dynamic Increase factor, structural model 20 is redesigned considering the importance factor of 1.2. The dynamic increase factor is calculated for model and compared with initial model 20 having importance factor 1. The comparision of dynamic increase factor on these two case is shown in figure 11 below.

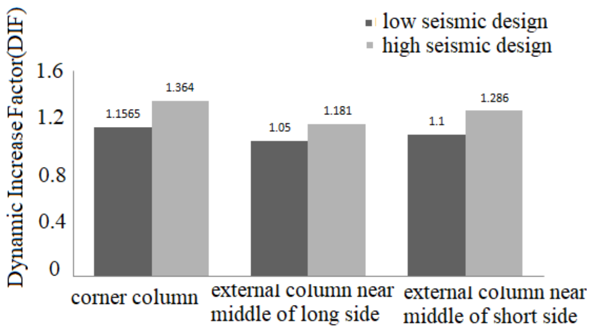

Figure 11: Variation of DIF with seismic design level

It is seen that in all column removal case, the value of dynamic increase factor increases with seismic design level. Also the value of dynamic increase factor is more in case of corner column removal case than others.

5. Conclusions

- It is observed that the nonlinear static analysis with suitable DIF can estimate the structural dynamic responses of beams within the affected bays with accuracy.
- Numerical results from analysis shows that the dynamic amplification factor decreases as the nonlinearity level increases where max(Mu/My) represents nonlinearity level. The slope of trendline of DIF with max(Mu/My) is gentle when max(Mu/My) is less than 1 and the slope is steep after max(Mu/My) is greater than 1.
- The DIF value in all cases are less than 2 which indicates that the use of amplification factor 2 in earlier version of guidelines was conservative.
- The latest version of guideline (UFC2009 and GSA 2013) showed the dependency of dynamic Increase Factor with ultimate level of inelasticity only but from results, it is seen that it is dependent on actual level of plastic deformation level.
- It is also observed that when the seismic design level of structure increases, the DIF value increases but nonlinearity level decreases. Also higher is the seismic design level, the building is less vulnerable to progressive collapse.

References

[1] General Service Administration. Alternate path analysis & design guidelines for progressive collapse resistance (GSA 2013). washington (DC).

[2] Department of Defence. Design of buildings to resist progressive collapse (UFC 4-023-03). washington (DC).

[3] S. M. Marjanishvili. Progressive analysis procedure for progressive collapse. 18(2):79–85.

[4] General Service Administration. Progressive collapse analysis and design guidelines for new federal office buildings and major modernization projects (GSA 2003). washington (DC).

[5] Department of Defence. Design of buildings to resist progressive collapse (UFC 4-023-03). washington (DC).

[6] Indian Standard Code. IS 1893 (part 1), criteria for earthquake resistant design of structures.

[7] American Society of Civil Engineers. Seismic rehabilitation of existing buildings (ASCE 41-13). new york (NY).

[8] American Society of Civil Engineers. Minimum design loads for buildings and other structures (ASCE 7-10). new york (NY).

[9] K. Marchand, A. McKay, and D. J. Stevens. Development and application of linear and non-linear static approaches in UFC 4-023-03. In *Proceedings of the 2009 Structures Congress*, pages 1–10.

[10] Milan Bandyopadhyay and Atul Krishna Banik. Progressive collapse of rigid and semi-rigid jointed steel frames according to GSA 2013 and GSA 2003 guidelines. 6(3):211–223.

[11] American Concrete Institute. Building code requirements for structural concrete (ACI 318-08) and commentary.