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Design guides to resist progressive collapse for steel structures

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Abstract. The progressive collapse phenomenon in structures has been interested by civil engineers and the building standards organizations. This is particularly true for the tall and special buildings ever since local collapse of the Ronan Point tower in UK in 1968. When initial or secondary defects of main load carrying elements, overloads or unpredicted loads occur in the structure, a local collapse may be arise that could be distributed through entire structure and cause global collapse. One is not able to prevent the reason of failure as well as the prevention of propagation of the collapse. Also, one is not able to predict the start point of collapse. Therefore we should generalize design guides to whole or the part of structure based on the risk analysis and use of load carrying elements removal scenario. There are some new guides and criteria for elements and connections to be designed to resist progressive collapse. In this paper, codes and recommendations by various researchers are presented, classified and compared for steel structures. Two current design methods are described in this paper and some retrofitting methods are summarized. Finally a steel building with special moment resistant frame is analyzed as a case study based on two standards guidelines. This includes consideration of codes recommendations. It is shown that progressive collapse potential of the building depends on the removal scenario selection and type of analysis. Different results are obtained based on two guidelines.

Keywords: progressive collapse; steel frame structures; direct and indirect design; AP method; retrofitting methods

1. Introduction

There were no useful studies and guidelines related to progressive collapse phenomenon in buildings until collapsing of the Ronan Point tower in the United Kingdom in 1968 which caused by gas explosion in kitchen in 18th floor. Also sequent events such as collapsing of the Mary Axe buildings on London bombing in 1992, the Bishopsgate building in 1993 and the Alfred P. Murrah tower in 1995 caused engineers to pay more attention to progressive collapse approach. Afterward researchers focused to derivate new design and construction methods and resistant materials numerically. A few experimentally studies have been implemented by researchers but no expected results have been obtained.

The progressive collapse can occur if no alternate path exists in the structure to redistribute vertical loads after a local failure. Therefore, considering the resistant alternate paths in the structure designing is the main philosophy to mitigate the progressive collapse. In steel moment

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resisting frames to prevent progressive collapse, structural systems require to have a well-distributed, redundant lateral load resisting system and ductile connections capable of undergoing large inelastic rotations without failing.

Since structural hardening is costly and engineers cannot reasonably design the entire building for unpredicted loads or probable overloads, so analysis and design criteria should be capable of preventing progressive collapse without the need to strengthen the whole structure.

Explicit design methods for progressive collapse resistant design can be found in the General Services Administration (GSA 2003) and the United Facilities Criteria (UFC) (DoD 2013). The GSA (2003) guidelines provide a methodology to mitigate progressive collapse potential in structures based on the alternate path method (AP). In this approach, the structure is designed such that if any one component fails, alternate paths are available and a general collapse does not occur. The resulting post damage mechanism after the component failing is a sagging beam spanning two bays which must be capable of sustaining large deflections and the generated overload by catenary action. At the failed column location, effective continuous facilities are necessary in order to form a plastic hinge and transfer axial load developed by the catenary action.

The analysis procedures recommended by the guidelines for AP method are linear static, linear dynamic, nonlinear static and nonlinear dynamic methods. Kaewkulchai and Williamson (2003) investigated the analysis procedures using a two-dimensional frame analysis, and found that linear static analysis might result in non-conservative results since it cannot reflect the dynamic effect caused by sudden exclusion of columns. As the phenomenon of progressive collapse is nonlinear in nature, it is more reasonable to carry out nonlinear analyses with nonlinear modeling of each element. Among the nonlinear analysis procedures, Marjanishvili (2004) indicated that the nonlinear static procedure may result in larger ductility because the load path moves not to surroundings but to vertical direction.

In the UFC (DoD 2013) two design methods are specified; the tie force method (TF) and the AP. The ASCE 7-02 (2006) presents general design guidelines and suggestions for improving structural integrity, which includes a catenary action of the floor slab among others. The Best Practice for Reducing the Potential for Progressive Collapse in Buildings (NIST 2006) recommends the catenary action as one of means for upgrading existing buildings. The National Building Code of Canada (1995) specified requirements for design of major elements, establishment of connection elements, and ways of providing load transfer paths. Crawford (2002) proposed new connection details developed for earthquake such as Side-Plate^{TM} and the use of mega-truss in high-rise buildings.

2. Progressive collapse approach

The progressive collapse is defined as “*the spread of an initial local failure from element to element, eventually resulting in the collapse of an entire structure, or of a disproportionally large part of it.*”, (ASCE 2006, SCE 2006).

Leyendecker and Burnett (1976) estimated that at least 15-20% of building damages are due to progressive collapse. This kind of collapse has been occurred and reported in framed structures such as buildings, trusses and bridges.

Over stressed critical members due to abnormal unpredicted loading or up floor debris impact loads, can be damaged. Moreover, the initial or secondary defects in construction procedure can cause inefficient operation of a critical member or connection of a structure. Neither codes nor engineers consider these defects while designing the structure. Local damages can spread the entire structure and cause total collapse of the building. Hence, limiting the local collapse is a major concern of these researches. More ductility, connectivity and indeterminacy are a number of provided solutions. Progressive collapse phenomenon frequently is considered for important and tall buildings, thus experimental testing of these buildings is costly. Furthermore, numerical modeling of connection behavior and rigidity based on the tests results are recommended.

3. Analysis methods

Based on required accuracy, importance of the buildings and acceptable risk level, the analysis methodologies range from linear to nonlinear with static and dynamic approaches.

3.1 Linear Static (*LS*)

In this method dead and live loads are applied statically. Therefore, it is the easiest and simplest method for progressive collapse analysis. The Unified Facilities Criteria (UFC) in the UFC 4-023-03: design of buildings to resist progressive collapse, recommends following increased gravity load combination in the static analysis equation (linear and nonlinear) for those bays immediately adjacent to the removed element and at all floors above it. Factor of 2 is dynamic amplification factor (DAF) to simulate dynamic response and account nonlinear effects. For the rest of the structure, load combination without the DAF is applied.

$$2.0\left[(0.9\,\mathrm{or}\,1.2)\,\mathrm{Dead} + (0.5\,\mathrm{Live}\,\mathrm{or}\,0.2\,\mathrm{Snow})\right] \tag{1}$$

Beside, the UFC 4-023-03 offers tables as acceptance criteria for elements forces and connections rotations. According to this standard, the analysis and design will be complete if none of the elements and connections violates the acceptability criteria mentioned in those tables and all damage limits are satisfied. Modifying the geometry or material properties of the model are a number of provided solutions.

The General Services Administration (GSA) standard for progressive collapse analysis and design purposes the following vertical load combination to be applied downward to the structure under investigation.

$$2.0(\mathrm{Dead} + 0.25\,\mathrm{Live}) \tag{2}$$

This method is used for simple structures if nonlinear and dynamic response effects are predicted easily (Marjanishvili and Agnew 2006).

3.2 Nonlinear Static (*NLS*)

Similar to “pushover analysis”, in the nonlinear static analysis, applied loads on the structure increase gradually until a collapse of structural element occurs. When any element violates criteria, it will be removed. Re-analysis and re-design will be done until no more violation reported. In this type of analysis, structural elements experience inelastic behavior (Marjanishvili and Agnew 2006).

It is clear that using factor of 2.0 for both linear and nonlinear analysis is an inconsistency of these methods. So a dynamic increase factor (DIF) that is smaller should be used instead of the DAF to account just the inertial effects.

3.3 Linear Dynamic (*LD*)

The dynamic analysis for progressive collapse approach is carried out using an 'initial conditions' methodology. By taking the effects of missing load-bearing element into account, more accuracy is possible just in dynamic analyzing procedure. This missing causes a sudden geometric change in the structure, load distribution ways and further has dynamic effects on the vicinity elements. These effects are resulted in a release of the potential energy and rapid variation of the internal dynamic forces, including inertia forces (Marjanishvili 2004). In this method, structural properties (stiffness, damping, etc.) are constant during the analysis. In spite of its accuracy, it is usually avoided due to its complexity.

Since the dynamic effects are already considered in the analysis, the UFC 4-023-03 recommends following load combination without amplification load factor for both linear and nonlinear dynamic analysis.

$$(0.9 \text{ or } 1.2)\text{Dead} + (0.5\text{Live or } 0.2\text{Snow}) \tag{3}$$

The load combination (4) has been purposed by the GSA standard.

$$\text{Dead} + 0.25\text{Live} \tag{4}$$

3.4 Nonlinear Dynamic (*NLD*)

The most accuracy for progressive collapse analysis can be accomplished if both dynamic effects and nonlinearity of geometry and materials are taken into account. These include the main differences between linear static and nonlinear dynamic analysis. In other words, dynamic amplification factor A and strength reduction factor β are applied in the LS method to consider the dynamic effect and the nonlinear behavior of materials, respectively. Therefore, the nonlinear dynamic analysis (NLD) consumes more time and effort and can trigger formal peer review requirements set by building codes.

In the NLD method the stiffness, damping and loads may depend on the displacements, velocities and time. This requires an iterative solution to the equations of motion.

4. Design methods

Two approaches are used for providing resistance to progressive collapse. The ASCE 7-02 as well as other researchers defines two general approaches to mitigate damage due to local failure but without quantitative requirements: Direct and Indirect Design.

Designing methods use following three strategies to reduce the risk of progressive collapse in the event of the loss of structural elements.

- Redundancy: After missing elements, redundancy ensures that alternate load paths in the vertical-load-carrying system are available.
- Ties: Vertical and horizontal continuous ties system along the principal lines of structural framing can serve to redistribute loads.
- Ductility: to resist under large deformation and rotation of connections, appropriate ductility should be provide for elements and connections

4.1 Direct design

The Direct design approach includes “explicit consideration of resistance to progressive collapse during the design process through one of two methods.” (1) Alternate Path method (AP); and (2) Specific Local Resistance method (SLR).

4.1.1 Alternate Path method (*AP*)

This direct method requires that the structure has capability of bridging over a missing element with the resulting extent of damage being localized. This work will be done with the removal scenario of specific vertical load bearing elements. The missing element is any element that cannot provide an adequate vertical tie force. In case of inability of bridging over, the element must be designed as a key element and should have enough strength to resist extreme loads.

This method follows the LRFD philosophy by employing the ASCE 7 load factor combination for extraordinary events and resistance factors to define design strengths. It is used in two situations: (1) when a vertical structural element cannot provide the required tie strength; and (2) for structures that require Medium or High Levels of Protection.

According to the UFC 4-023-03 (DoD 2013) standard, in this method with any of the design methodologies (Linear Static to Nonlinear Dynamic), the limitation of considered damage during and at the end of the analysis must be quantified.

Damage limits for removal of external column or load-bearing wall require that the collapsed area of the floor directly above the removed element must be less than the smaller of 70 m^2 (750 ft^2) or 15% of the total area of that floor . The floor directly beneath the removed element should not fail, as well as no collapse extension allows beyond the structure tributary to the removed element.

Damage limits for removal of internal column or load-bearing wall require that the collapsed area of the floor directly above the removed element must be less than the smaller of 140 m^2 (1500 ft^2) or 30% of the total area of that floor. The floor directly beneath the removed element should not fail, as well as any collapse extension not allowed beyond the bays immediately adjacent to the removed element.

The linear static procedure of AP method concludes three general steps (Lu *et al*. 2008).

(1) Conduct linear static analysis for each removing target load carrying elements respectively to get the static design internal force S_{static}.

(2) Use dynamic amplification factor A to estimate the dynamic design internal force $S_{dynamic}$:

$$S_{dynamic} = A.S_{static} \tag{5}$$

(3) Residual structural elements should have elasto-plastic strength R.

$$R \geq \beta.S_{dynamic} \tag{6}$$

in which β is the strength reduction factor that represents the energy dissipation capacity in plastic deformation of real structures.

A and β will be determined by comparing the difference between nonlinear dynamic and linear static results.

4.1.2 Specific Local Resistance method (SLR)

The SLR method requires that the building or parts of it provide sufficient strength to resist a specific load or threat for the specific protection level. This method is costly and not applicable for any buildings.

4.2 Indirect design

In the Indirect Design method, resistance to progressive collapse is considered implicitly through the provision of the minimum levels of strength, continuity and ductility. The commentary in the ASCE 7-02 goes on to present general design guidelines and suggestions for improving structural integrity. These include: (1) good plan layout; (2) integrated system of ties; (3) returns on walls; (4) changing span directions of floor slabs; (5) load-bearing interior partitions; (6) catenary action of the floor slab; (7) beam action of the walls; (8) redundant structural systems; (9) ductile detailing; and (10) compartmentalized construction.

This method is considered the simplest method with general design guidelines. Although it decreases the probability of progressive collapse, but simulation of post-failure behavior of structures is not readily possible (Sasani and Sagiroglu 2008). Conceptual method and Tie Force method (TF) are two indirect design methods.

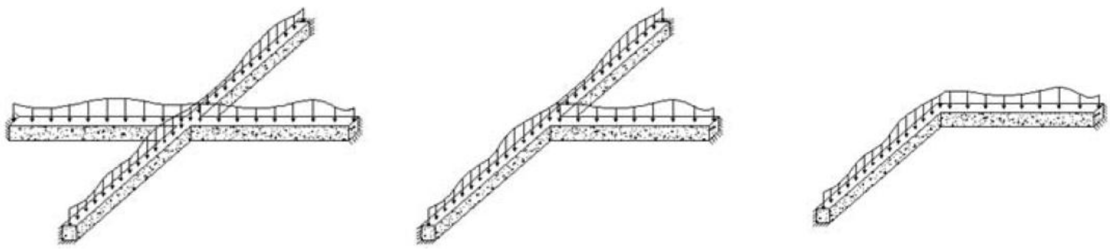
4.2.1 Conceptual method

This method emphasizes on integrity, ductility and redundancy of structures by rationally arranging structural members and strengthening weak members and joints. Usefulness of this method depends on engineer's experiences.

4.2.2 Tie Forces (TF)

This indirect design approach enhances continuity, ductility, and structural redundancy by requiring "ties" to guarantee the integrity of structures and reserved load path. Unlike the AP method, this method does not need to calculate the response of the entire structure. Therefore it is easy to apply but has lower precision. In this method, designer can divide the structure into substructures with own loading patterns (Fig. 1). It is assumed that the progressive collapse will be prevented if every substructure has proper strength and enough deformation capacity. Ultimate state of each substructure is analyzed in the TF method directly. Therefore, each substructure is a static determinate problem. By the fact that many assumptions have been used in this method, the experimental factors should be implemented carefully to guarantee the accuracy of the TF method.

As shown in Fig. 2 and based on construction type, there are several horizontal ties that must be



(a) Interior (b) Side (c) Corner

Fig. 1 Substructures positions in TF method

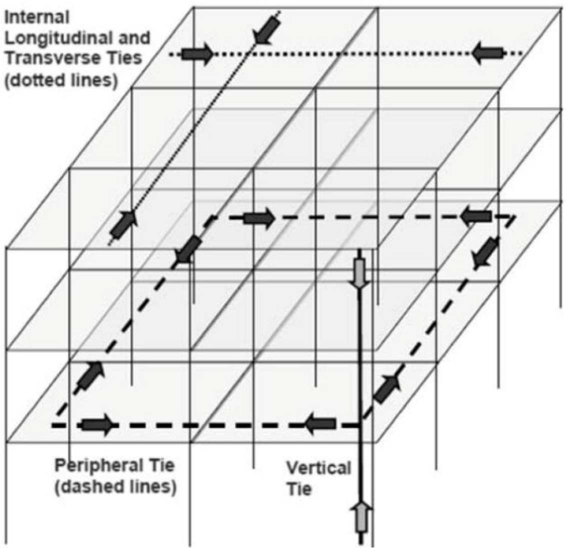


Fig. 2 Schematic of tie forces in a frame structure (GSA)

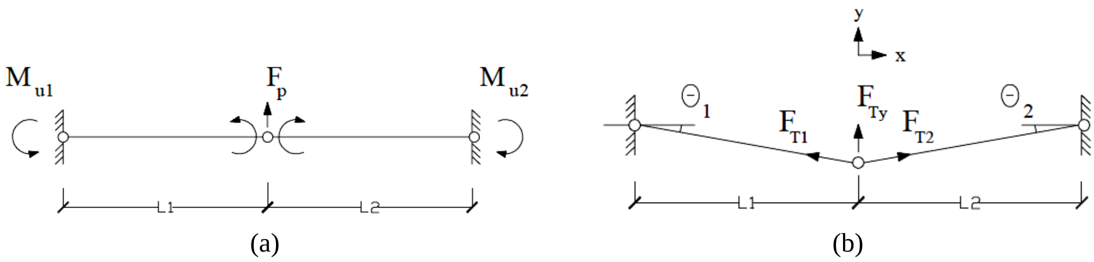


Fig. 3 Mechanism in beam tied elements

provided: internal, peripheral, and ties to edge columns, corner columns, and walls. Vertical ties are required in columns and load-bearing walls.

Basic assumption for this method is that two mechanisms may be occurred in beam after tied elements bridging over a vertical load carrying element, (1) beam mechanism that considers only bending capacity at the fixed end of the beams (Fig. 3(a)); and (2) catenary mechanism that is considered only if the beams are continuous. In this mechanism, tensile strength of beam is calculated (Fig. 3(b)) (Lu *et al*. 2008).

Steel members acting as internal ties and their end connections must be resisting under following load combination (UFC 4-023-03, DoD 2013)

$$0.5(1.2D+1.6L)s_t L_l > 75\,\mathrm{KN} \tag{7}$$

Where L_l is span and s_t is mean transverse spacing of the ties adjacent to the ties being checked. Peripheral ties must be capable of resisting (UFC 4-023-03, DoD 2013)

$$0.25(1.2D+1.6L)s_t L_l > 75\,\mathrm{KN} \tag{8}$$

Main steps of design procedure are beginning from calculating equivalent nodal load of substructures include bending moment of each beam. Total equivalent load (N) shall be resist by beam and catenary mechanisms shown in Fig. 3. (Lu *et al*. 2008)

$$N = \sum_{i=1}^{n} (F_p^i + F_{Ty}^i) \tag{9}$$

$$\sum_{i=1}^{n} F_p^i = \sum_{i=1}^{n} \frac{M_u^i}{L_i} \tag{10}$$

Where n is the total number of beams in the substructure, F_p^i and F_{Ty}^i are the resistances of i-th beam with beam mechanism and catenary mechanism, respectively, L_i is the length of i-th beam and M_u^i is the negative bending capacity at the fixed end of i-th beam.

The tie force does not need to be considered if N can be borne just by beam mechanism. The tie force in each direction shall be obtained considering the bending stiffness of beams if there are continuous beams in joints (Lu *et al*. 2008)

$$F_{Ty}^i = \frac{\dfrac{EI_j}{L_j^3}}{\displaystyle\sum_{j=1}^{m} \frac{EI_j}{L_j^3}} \times \sum_{i=1}^{n} F_{Ty}^i \tag{11}$$

Where I_j is the rotational inertia and F_{Ty}^i is the tie force of beam in j-th direction.

5. Material properties

Steel is a ductile material and has shown good resistance in large deformation and rotation occurs in progressive collapse phenomenon. In spite of high strength, it is lighter than concrete which is advantageous for progressive collapse resistance. According to Table 1 appropriate over-strength factors shall be apply to calculating design strengths for both Tie Forces and Alternate Path methods.

Table 1 Over-Strength Factors for structural Steel (UFC 4-023-03)

Structural steel	Ω_u	Ω_y
Hot-rolled structural shapes and bars		
ASTM A36/A36M	1.05	1.5
ASTM A573/A572M Grade 42	1.05	1.3
ASTM A992/A992M	1.05	1.1
All other grades	1.05	1.1
Plates		
All other products	1.05	1.1

Ω_u: Ultimate Over-Strength Factor; Ω_y: Yield Over-Strength Factor

6. Progressive collapse in guidelines

Three major methods for analyzing and designing structures to prevent progressive collapse are pointed in most foreign codes include: Conceptual method, TF method and AP method. Here, some of them are summarized.

6.1 Department of Defense Unified Facilities Criteria (*DoD*)

The DoD classified designing for resistance to progressive collapse to the "level of protection" assigned to the building. It recommends indirect design method for structure with lower levels of protection by providing minimum tie forces and alternate path method for higher levels of protection and for the structures systems that sufficient ties cannot be provided.

6.2 General Services Administration

The General Services Administration (GSA) mentioned guidelines about the alternate load path method and removal of vertical load carrying members. It directs the structural engineer to design buildings for the loss of a column along the perimeter for the first two floors above grade to prevent progressive collapse.

6.3 ASCE 7

As mentioned before, the ASCE 7-05 is one of the good codes that points to the direct design approaches (alternate path method and specific load resistance method) and the indirect design approach. It provides design guidelines for general structural integrity, such as good plan layout and use of structural ties. It recommends load combinations including extraordinary loads, and explains the underlying probabilities as well.

6.4 British Standards

The British Standards points three design approaches for resisting progressive collapse including Tie Forces, Alternate Load Path and Specific Local Resistance methods.

6.5 Eurocode

The Eurocode recommends tying the structure elements together and defines parameters of tie forces. It also provides basic design guidelines to prevent progressive collapse. This code classified buildings to four safety classes that only the two highest classes require consideration of progressive collapse design guidelines.

6.6 National Building Code of Canada

The National Building Code of Canada contains general strategies for structural to support sufficient integrity. It includes recommendation for good structural layout, continuity, and structural mechanisms to mitigate progressive collapse after local damage. Tie forces or accidental loads for key structural elements are not specified.

Table 2 Recommended load combination in codes

Standards	Load combinations after notional member removal
BS	(1 ± 0.5) D + L /3 + W/3
Canada 1977	D + L/3 + W/3
ASCE 7-98, 02, 05	(0.9 or 1.2) D + (0.5 L or 0.2 S) + 0.2 W (with member removal) 1.2 D + A_k + (0.5 L or 0.2 S) (specific local resistance method) (0.9 or 1.2) D + A_k + 0.2 W (specific local resistance method) (A_k = extraordinary load)
DOD UFC 4-010-01	D + 0.5 L net floor uplift
DOD UFC 4-023-03	D +0.5 L net floor uplift (0.9 or 1.2) D + (0.5 L or 0.2 S) (nonlinear dynamic analysis) 2.0 [(0.9 or 1.2) D + (0.5 L or 0.2 S)] (static analysis)
NYC 1998, 2003	2 D + 0.25 L + 0.2 W
GSA, 2003	2 (D + 0.25 L) static analysis D + 0.25 L dynamic analysis

D, L, W, S = dead, live, wind and snow loads

Table 3 Classification design of codes for progressive collapse resistance

Eurocode 1	Protection level	1	2 (Lower risk)	2 (Upper risk)	3
	Design method	Conceptual Method	Horizontal TF method	(1) TF method (2) AP method	Risk assessment
DoD	Protection level	Very low	Low		Medium and high
	Design method	Horizontal TF method	(1) Vertical TF method (2) Horizontal TF method (AP method if failures)		(1) TF method (2) AP method (3) Ductility requirement

6.7 New York City Building Code

The 1998 New York City Building Code points to the direct design such as alternate load path and specific local resistance methods.

Tables 2-3 indicate load combinations and classifications of relevant important codes related to progressive collapse.

7. Retrofitting methods

In addition to the design guides, some researches recommend appropriate retrofitting systems to resist progressive collapse. Main retrofitting strategies for steel frame buildings are described below (Crawford 2002).

- Strengthening members, especially premier columns and beams by adding plates or encasing/filling them with concrete, such as design and details recommended in AISC 341.
- Providing redundant and alternate load path as described before.

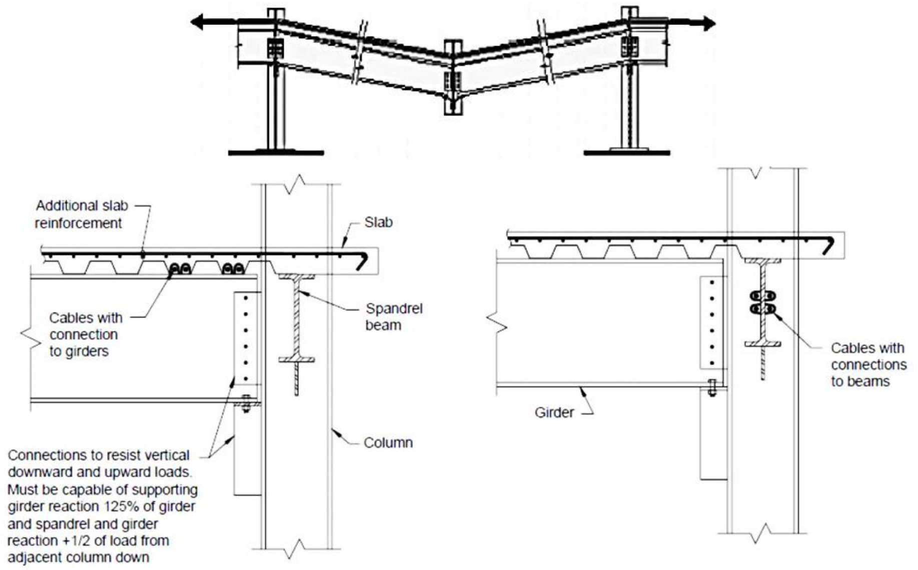
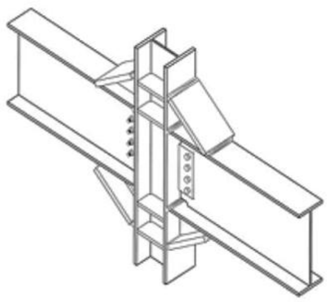
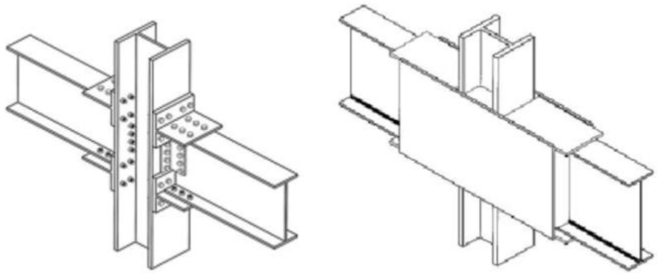


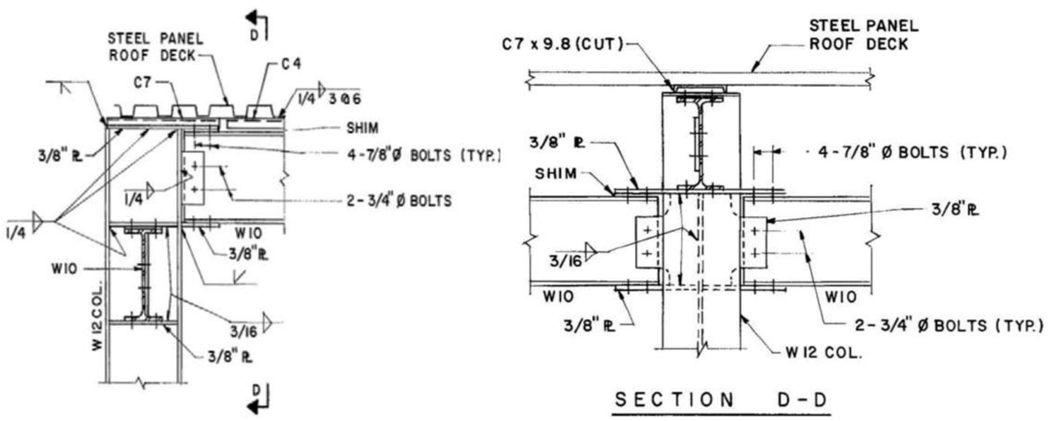
Fig. 4 Cable techniques to make horizontal tie forces (Crawford 2002)



(a) GSA recommended details



(b) Sidepalate™ system (Crawford 2002)



(c) TM5-1300 sample recommended details

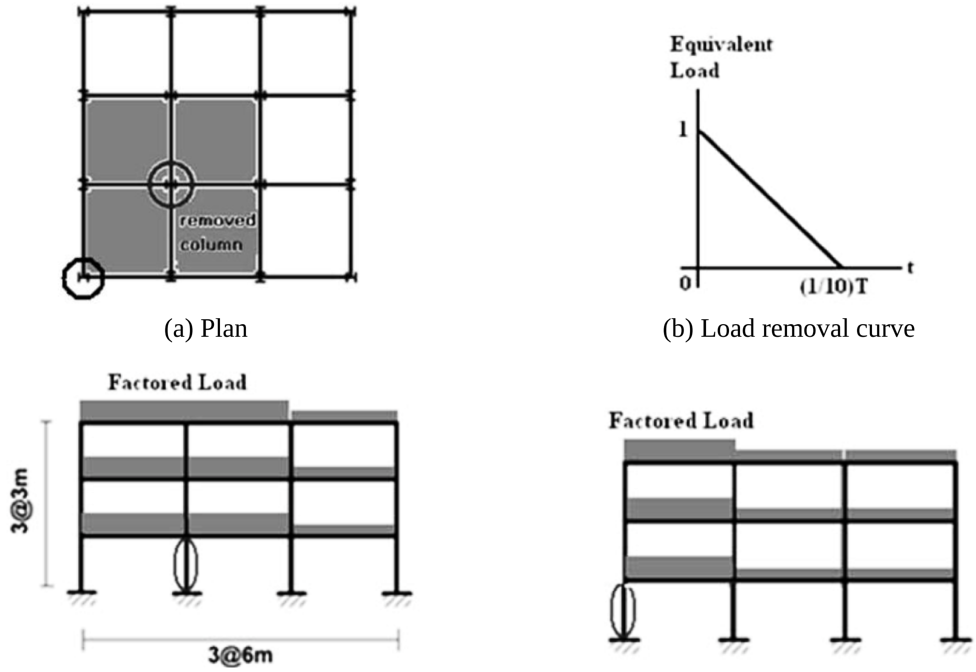
Fig. 5 some details for strengthening steel connections

- Cabling: These cables cause load transformation from missing elements to another, retain slabs and prevent progressive collapse (Fig. 4).
- Megatruss: megatruss usually used to create a strong floor at intervals in tall buildings. It can suspend/support damaged portions of the structure and provide an alternate load path around the damage zone. Adding columns or cables on the building's exterior supports the trusses.
- Composite shear walls: combination of steel elements and concreting.
- Steel connection: Connections should be designed to developing: (1) beam plastic moment; and (2) beam axial tension capacity. There are some suggested details for strengthening steel connections in Fig. 5.

In Side-Plate™ system, separation between the face of the column flange and the end of the beam mitigates the triaxial stress concentrations and reduces local buckling.

Some beneficial features that have been suggested by codes and researchers to mitigate progressive collapse for different structural systems are listed below.

- Closely spaced beams and columns may improve load redistribution.
- Provide resistance to collapse in both directions; do not assume "plane frame" behavior.
- Decrease load concentrations by eliminating discontinuities.
- Design regular and symmetric building plans for load sharing and redundancy as it possible.
- Continuous beams/girders cause less deflection and increase load redistribution after column loss.
- Avoid eccentricities that may create large moment under excessive load.
- Shear walls, elevator shafts and stairwells may provide alternate load paths.
- Composite slab systems can be used to provide full lateral support to beams. But floor slab missing, changes support conditions and the unbraced length of the beam. Therefore, appropriate or additional lateral supports such as stud, prevent lateral-torsional-buckling of beams. Else, beam should be laterally braced to reach plastic moment capacity in both positive and negative moments
- Use seismically compact sections according to AISC 341 (AISC 2002) to prevent local buckling.
- High-strength bolted connections prevent brittle failure from concentrated stresses at weld locations.
- Using notch-tough weld in connections and built-up elements are recommended for seismic design in AISC 341 (AISC 2002).
- Use the AISC-341 pre-qualified fully restrained (FR) connections.
- Provide strong column-weak girder strategy to ensure plastic hinging of beams.
- Check critical and premier columns stability for two story unbraced length due to loss of adjacent beams.
- Provide column continuous stiffener plates to prevent prying of column flanges under catenary tension action of beams. Stiffeners must be capable of transferring catenary tension from beam to beam across the column web.
- Add double plates to column web.
- Provide lateral flange bracing in narrow columns to reduce unbraced length.
- Consider reinforcement mesh in slab center or use two layers of continuous bars.



(a) Plan (b) Load removal curve

(c) Case-C1 (d) Case-C2

Fig. 6 (a) structure plan; (b) load removal curve; (c) and (d) missed columns scenarios

8. Case study

Three-story and three-bay square building with special moment resisting steel frame was used as a three dimensional numerical case study (Fig. 6). This structure has been studied by Kim and Park (2008) and has been designed seismically based on the IBC 2006.

Tensile yield strengths for columns and beams were Fy = 32.4 kN/cm^2 and Fy =23.5 kN/cm^2, respectively. Yield over strength factor Ωy equal to 1.5 was applied to the yield strengths due to high strain rate according to the Table 1. Profiles H250×255×14×14, H304×301×11×17 and H300×150×6.5×9 were assigned as external, internal columns and beams in three stories, respectively. All columns were fixed to the foundation. The design dead and live loads were equal to 5.0 kN/m^2 and 3.0 kN/m^2. Concrete slabs with 10 cm membrane thickness formed composite floors. These slabs were simulated with shell elements by membrane acting. Rigid diaphragms were assigned to these slabs in each story separately.

This structure was analyzed based on two standards guidelines; the UFC 4-023-03 (DoD 2013) and the GSA (2003). Results of analyses are implemented and compared in following parts.

8.1 Analysis based on the UFC 4-023-03 (*DoD 2013*)

The AP method was applied to evaluate resistant of the code-designed structure against progressive collapse. Assumed removed columns in ground floor are indicated in Fig. 6(c) and (d). According to the UFC 4-023-03 guidelines, there were no Demand-Capacity-Ratio or geometric irregularity limitations, so nonlinear static and nonlinear dynamic analysis methods considering *P*-

Δ effects were performed to demonstrate progressive collapse potential of the building.

In the nonlinear static analysis dead and live load were applied simultaneously according to the load combination (1). The beams were considered to be failed when their rotations exceed the limit state of 0.035 rad as specified in the guidelines.

Before removing the assumed columns in each case, maximum displacement and rotation of top joint of them obtained equal to $\delta = 1.0$ mm and θ = 7.64e-6 radian for Case-C1 and δ = 0.7 mm and θ = 8.4e-6 radian for Case-C2, respectively. It seems true that without applying earthquake loading for seismically code-designed structure, deformations of statically loaded building are very little. After this static step, assumed columns removed suddenly and separately. In Case-C1 deformation obtained equal to δ = 172 mm and θ = 6.8e-4 radian under load combination (1) with DAF equal to 2 for vicinity spans of removed column and stories above (Fig. 6(c)). All 12 beams above missed element were over-strength. In Case-C2 deformation obtained equal to δ = 118 mm and θ = 0.012 radian and 6 beams above missed element were over-strength. No column failed in Case-C2 in comparison of four vicinity columns failures in Case-C1. Comparing results of these two cases shows that missing interior columns are more critical than exterior one. More span's loads applied to the interior columns (4 spans) and generated beams with double spans after column removal might be the reasons.

In nonlinear dynamic analysis (NLD), important modeling parameters are damping ratio, time step, column removal time and plastic hinge definitions. The load combination (3) was applied to the structure. For this study damping ratio and column removal time (T_c) were considered equal to 5% of the critical damping and (1/10)T, respectively. Where, T is the natural period of undamaged structure first mode. Time step of 0.001 second was used for time-history analysis. Nonlinearity was applied to the models by plastic hinges approach at elements considering interaction of two direction moments and axial force for both ends and middle of beams and interaction of buckling and axial force for both ends of columns. Dynamic analysis was carried out using Hilber-Hughes-Taylor method with α = 0.0, β = 0.25 and γ = 0.5 and the Newton-Raphson solution algorithm. The building was analyzed with and without considering effect of slabs stiffness. In this method, the column is deleted in the structural model and the internal forces (F_{eq}) which determined from undamaged equilibrium model are applied to the structure as a load case to the joint of column's

Table 4 Nonlinear modeling parameters and acceptance criteria of steel frame connections (UFC)

Connection type	Nonliear modeling parameters			Nonlinear acceptance criteria	
	Plastic rotation angle, radians		Residual strength ratio	Plastic rotation angle, radians	
	a	b	c	Primary	Secondary
Full restrained moment connections					
Improved WUF with bolted web	0.021-0.0003d	0.050-0.0006d	0.2	0.021-0.0003d	0.050-0.0006d
Reduced Beam Section (RBS)	0.050-0.0003d	0.070-0.0003d	0.2	0.050-0.0003d	0.070-0.0003d
WUF	0.0284-0.0004d	0.043-0.0006d	0.2	0.0284-0.0004d	0.043-0.0006d
SidePlate*	0.089-0.0005d(3)	0.169-0.0001d	0.6	0.089-0.0005d	0.169-0.0001d

d: depth of beam (inch)

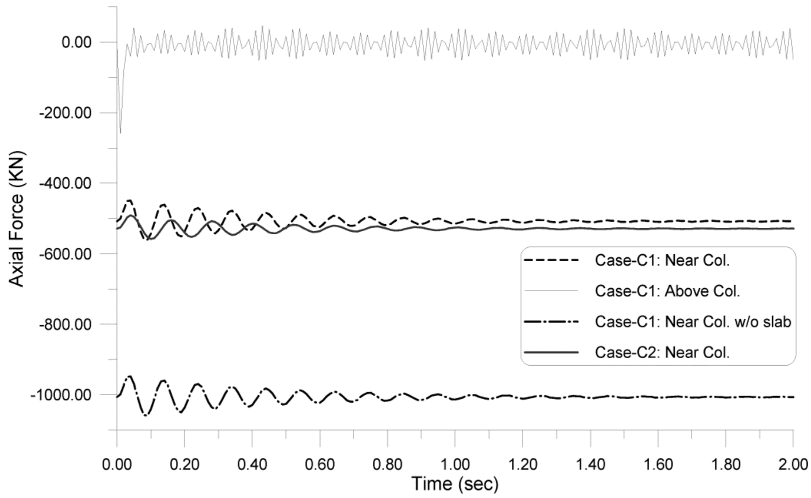


Fig. 7 Axial force time-history curves of near and above columns of the removed columns

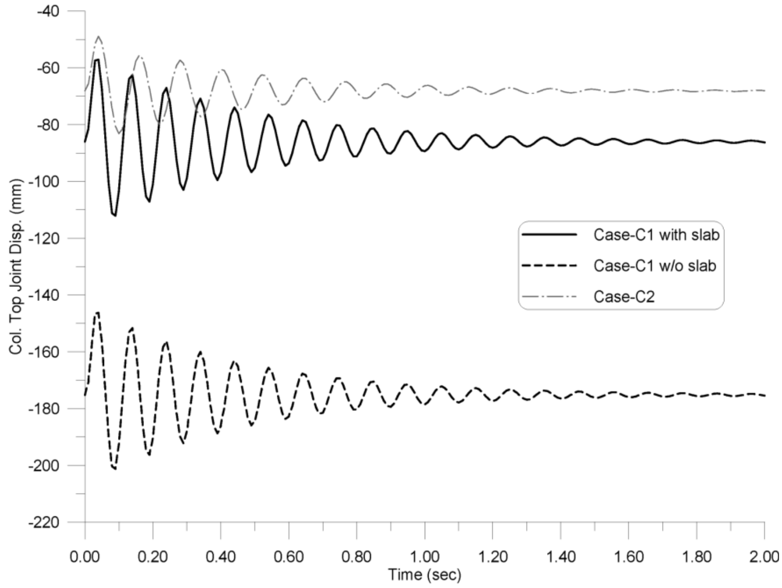


Fig. 8 Time history of vertical deflection at the lost column-beam connection (both cases)

end. Static nonlinear analysis results are used as the initial condition for the column removal. The applied equivalent loads (F_{eq}) are ramping down under duration of T_c.

Nonlinear acceptance criteria and component definitions for plastic hinges of fully restrained connections are shown in Table 4 according to Tables 5-2 of the UFC for the Life Safety condition.

Axial forces of monitored columns were indicated in Fig. 7 with and without considering effect of slab stiffness. Mean value of near column's axial load in NLD-case-C1 analyses was 508 KN which is approximately half of the NLS method (1035 KN). Additionally as mentioned in Fig. 7 resultant axial force of above column might be neglected. It is notable that axial force variations of near columns in both cases are similar. Peer looking at Fig. 7, shows that axial force variations of near column of missed one were decreased approximately 50% due to taking into account the slab stiffness.

Fig. 8 indicates vertical displacement time-history of removed column top joint. In the NLD-case-C1, displacement values ranged from 57 mm to 112 mm. In equilibrium state final deformation was equal to 86 mm which was half of the NLS method (172 mm). The largest rotation of critical joint (corner of third floor) was equal to 0.0047 rad and it was acceptable according to the Table 4.

$$0.021-0.0003d=0.021-0.0003(12.5'')=0.01/25\,\text{rad}$$

It was shown that considering the slab stiffness effects, decreased 50% of the observed deformation. Also deformations obtained in the NLD-case-C2 were approximately 20% lower than the NLD-case-C1 results due to missed column position.

Internal moment of girder with new generated boundary conditions has been shown in Fig. 9. In the NLD-case-C1 mean value of positive or negative moment was equal to 183 KN.m and it was half of the NLS method's result (370 KN.m). Besides, it is expected that beam moment at the middle of the span be negligible (4 KN.m). We can derive following results from indicated curves in Fig. 9.

- Considering effects of slab stiffness decrease 63% of internal forces variations.
- Internal moments of generated cantilever beam in the NLD-case-C2 are at least 35% lower than 2-span generated beam in the NLD-case-C1 over the removed columns.

The last step of designing the structure to mitigate progressive collapse is to recognize plastic hinges forms in the structure and its stability. Following calculations show that the structure is stable based on the NLD method.

The FEMA-356, Table 5-6 has defined acceptance criteria for rotation of elements based on yield rotation θ_y, that is calculated by following relations (FEMA-356 2000): Eqs. (5-1) and (5-2)).

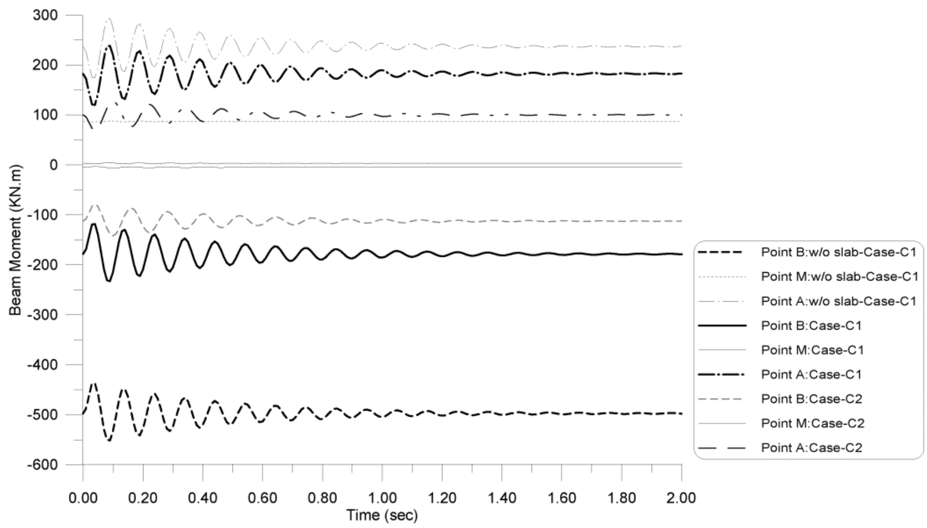


Fig. 9 Time-history of major moments of beam above the removed column

$$\theta_y = \frac{ZF_{ye}I_b}{6EI_b} \tag{12}$$

$$\theta_y = \frac{ZF_{ye}I_c}{6EI_c}\left(1-\frac{P}{P_{ye}}\right), \qquad P_{ye} = A_g F_{ye} \tag{13}$$

According to Table 5 and the Eqs. (12)-(13) we have:
For beams

$$\frac{b_f}{2t_f} = \frac{150}{2\times 9} = 8.33 < \frac{52}{\sqrt{F_{yc}}} = \frac{52}{\sqrt{23.5}} = 10.73, \quad \frac{h}{t_w} = \frac{300}{6.5} = 45.2 \le \frac{418}{\sqrt{F_{yc}}} = \frac{418}{\sqrt{23.5}} = 86.23$$

$$\theta_y = \frac{ZF_{ye}I_b}{6EI_b} = \frac{31.86\times 23.5\times 236.22}{6\times 29e3\times 166.55} = 0.0061 \quad \rightarrow \quad (LS): 6\theta_y = 0.0366$$

For exterior columns

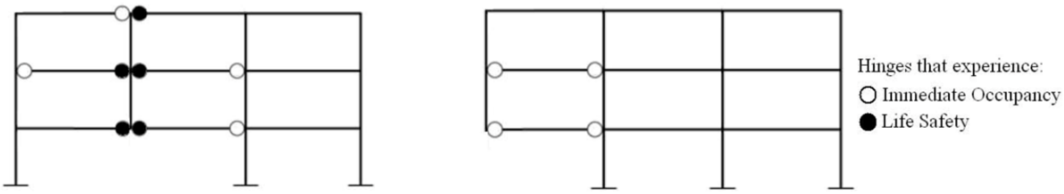
$$\frac{b_f}{2t_f} = \frac{250}{2\times 14} = 8.93 < \frac{52}{\sqrt{F_{yc}}} = \frac{52}{\sqrt{32.4}} = 9.14, \quad \frac{h}{t_w} = \frac{283.5}{14} = 20.25 \le \frac{300}{\sqrt{F_{yc}}} = \frac{300}{\sqrt{32.4}} = 52.7$$

$$\theta_y = \frac{ZF_{ye}I_c}{6EI_c}\left(1-\frac{P}{P_{ye}}\right) = \frac{61.94\times 32.4\times 118.1}{6\times 29e3\times 269.8}\left(1-\frac{508}{15.88\times 32.4}\right) = 0.0033 \quad \rightarrow \quad (LS): 6\theta_y = 0.02$$

Table 5 Acceptance Criteria for Nonlinear Procedures-Structural Steel Components (FEMA 356 (2000))

Component/ action	Modeling parameters			Acceptable criteria				
	Plastic rotation angle, radians		Residual strength ratio	Plastic rotation angle, radians				
					Primary		Secondary	
	a	b	c	IO	LS	CP	LS	CP
Beams−flexure								
$\frac{b_f}{2t_f} \le \frac{52}{\sqrt{F_{yc}}}$ and $\frac{h}{t_w} \le \frac{418}{\sqrt{F_{ye}}}$	$9\theta_y$	$11\theta_y$	0.6	$1\theta_y$	$6\theta_y$	$8\theta_y$	$9\theta_y$	$11\theta_y$
Columns−flexure								
For $P/P_{CL} > 0.20$								
$\frac{b_f}{2t_f} \le \frac{52}{\sqrt{F_{ye}}}$ and $\frac{h}{t_w} \le \frac{300}{\sqrt{F_{ye}}}$	$9\theta_y$	$11\theta_y$	0.6	$1\theta_y$	$6\theta_y$	$8\theta_y$	$9\theta_y$	$11\theta_y$

* IO, LS and CP are the acceptance criteria for deformation corresponding to the target Building Performance Levels based on the FEMA356 (2000) (Collapse Prevention (CP), Life Safety (LS), and Immediate Occupancy (IO)).



(a) Case-C1 (b) Case-C2

Fig. 10 Plastic hinges formation of the beams (both cases)

Fig. 10 indicates formation of plastic hinges in both cases based on implemented calculations. We can see no LS hinge has been formed in the case-C2 that shows more safety state rather than the case-C1. So, engineers should consider and design two-span generated beam above missed column in removal scenario. There was no CP hinge in both cases, so the structure remained safe enough for occupants to come out and repair the building.

8.2 Analysis based on the GSA 2003

In this section, load combination (2) that has been recommended by the GSA (2003) was applied to the structure in nonlinear static analyze method.

Before removing the assumed columns in each case, maximum displacement and rotation of top joint of them obtained equal to $\delta = 165.5$ mm and θ = 6.5e-4 radian for the Case-C1 and δ = 114 mm and θ = 0.011 radian for the Case-C2, respectively. All 12 beams above missed element in the Case-C1 and 6 beams above missed element in the Case-C2 were over-strength. No column failed in the Case-C2 in comparison of four vicinity columns failures in the Case-C1.

Comparison between these results and the results obtained based on the UFC recommendations in the NLS method shows that there is no notable difference between them.

Based on this guideline, in nonlinear dynamic analyze method the load combination (4) was applied to the structure. All considerations used in analyzing based on the UFC guideline were applied in this part (Section 8.1). Fig. 11 shows that the results obtained according to the GSA guidelines have more similarity to those obtained based on the UFC recommendations.

Nonlinear acceptance criteria are shown in Table 6 according to Table 2.1 of the GSA. No element violates these criteria after formation of plastic hinges based on the NLD method.

Table 6 Acceptance criteria for nonlinear analysis

Component	Ductility* (μ)	Rotation degrees (θ)	Rotation %radians (θ)
Steel beams	20	12	21
Steel columns (tension controls)	20	12	21
Steel columns (compression controls)	1		
Steel frames		2	3.5
Steel frame connections; Fully restrained			
Welded beam flange or coverplated		1.5	2.5
Reduced beam section		2	3.5

*Ductility is defined as the ratio of ultimate deflection to elastic deflection

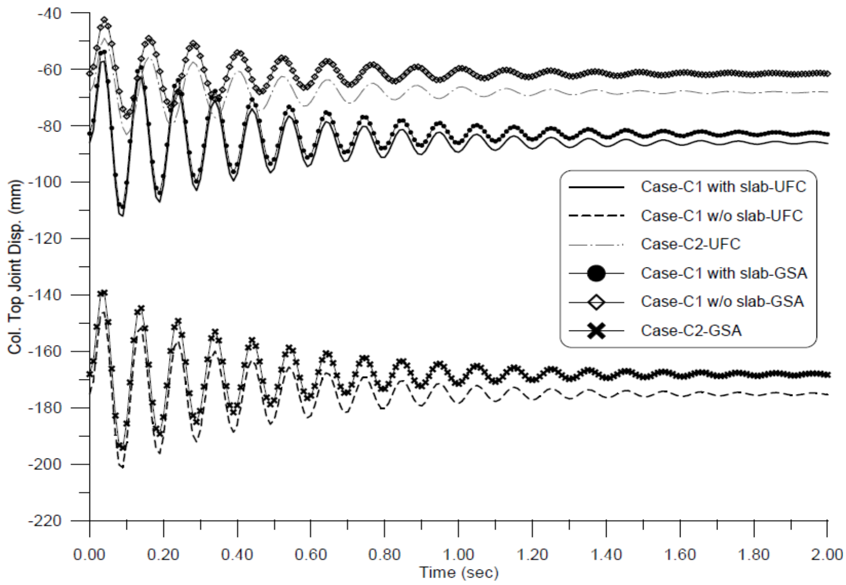
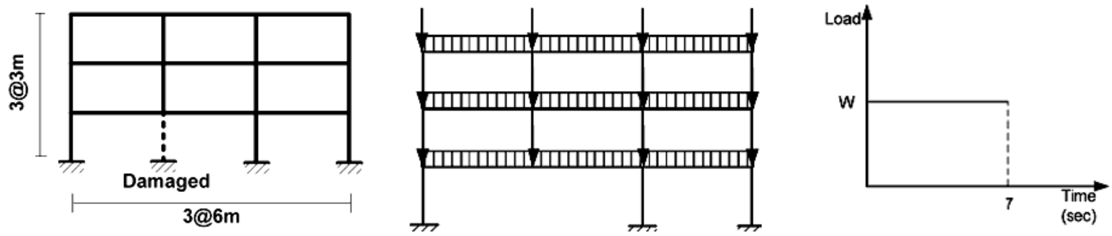


Fig. 11 Comparison of time history of vertical deflection based on the guidelines



(a) Elevation (b) Vertical Load (c) Time history of Vertical Load

Fig. 12 Analysis model structures (Kim and Park 2008)

Regarding to the similarity of the results based on two guidelines, formation of the plastic hinges is similar to Fig. 10.

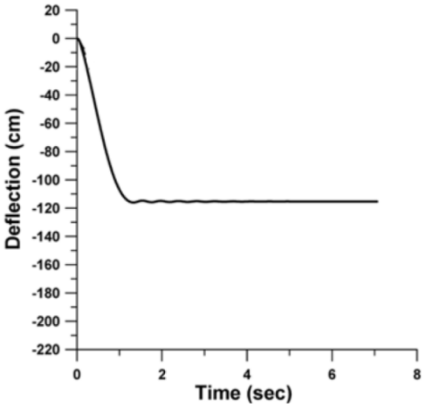
These comparisons show that both guidelines have been suggested fairly similar load factor in load combinations. Although, the NLD analyses show that these load factor are conservative.

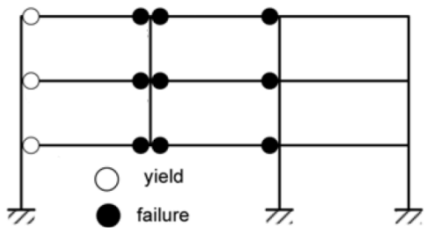
8.3 Model verifying

As mentioned in the Section (8) of this paper, the structure with three-story special steel moment frames that has been studied by Kim and Park (2008) was selected as a case study (Fig. 12).

They designed the structure based on the IBC-2006 guidelines. Potential of the structure for progressive collapse has been studied according to the GSA recommendations by them. Other assumptions used by the authors are listed below:

- Two-dimensional analysis was carried out.
- Effect of slabs was neglected.





(a) Time history of deflection at the lost column top joint (b) Plastic hinge formation

Fig. 13 Some concluded results by Kim and Park (2008)

- Performance of the structures subjected to sudden removal of a column was investigated by nonlinear dynamic and nonlinear static (pushover) analyses.
- Damping ratio was assumed to be 5% of the critical damping.

They observed that the modeled structure has high potential for progressive collapse when a first story column was suddenly removed. On the other hand, they concluded that the structures designed without considering progressive collapse did not satisfy the failure criterion required by the GSA guidelines.

Fig. 13 indicates some results that have been obtained by the authors. Comparing these figures and the mentioned results in this paper shows that neglecting effects of slabs and interaction of third dimension of the structure causes more vertical deflection at the lost column top joint rather than these study conclusions (About 120 cm comparing to 9 cm). Also Fig. 13(b) shows that the structure was unstable because of number and status of formation of plastic hinges. But we concluded in this study that the structure remained stable in the Life Safety status based on Fig. 10. Therefore, two-dimensional static analyses with neglecting effects of slabs stiffening reflect more unrealistic results.

9. Conclusions

Progressive collapse in the buildings occurs when one or more critical load bearing elements fail due to unconsidered abnormal loads or misconstruction. Codes and researchers define linear/nonlinear static/dynamic analyses and direct and indirect design methods to mitigate progressive collapse. One of the famous direct design methods is alternate path method (AP) that is threat independent and easy to use. In this method, designer should consider missing columns scenarios and design the building with new generated boundary condition. Beside, the Tie Forces (TF) is such an applicable indirect design method that guarantees the integrity of the structures by analyzing and designing the divided substructures with own loading patterns. Philosophy of all methods approaches is enhancing continuity, ductility and reserved alternate load path in the structure.

In this study the numerical results based on two guidelines named GSA (2003) and UFC 4-023-03 showed that the nonlinear dynamic analysis based on the AP approach allowed that seismically code-designed structure be capable to resist progressive collapse. Regular and symmetric plan of the considered structure, closely spaced beams and columns, high capacity of beam plastic hinges, ductile materials, providing strong column-weak girder strategy and etc are some of reasons for this result. Besides, considering effects of slab stiffness decreases at least 50% of structure deformation response and internal forces of elements. This result is repeated for structure if designer uses nonlinear dynamic analysis instead of nonlinear static.

It is shown that location of missed column has perceptible effects on the structure response. It depends on quantity of beams and slabs which support on the missed columns. Increasing number of them increases deformation and internal forces of elements. Finally, number of formed plastic hinges in structure recognizes stability of the structures. It is mentioned that removing the interior column scenario is more critical than exterior one.

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