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On the progressive collapse resistant optimal seismic design of steel frames

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Abstract. Design of safe structures with resistance to progressive collapse is of paramount importance in structural engineering. In this paper, an efficient optimization technique is used for optimal design of steel moment frames subjected to progressive collapse. Seismic design specifications of AISC-LRFD code together with progressive collapse provisions of UFC are considered as the optimization constraints. Linear static, nonlinear static and nonlinear dynamic analysis procedures of alternate path method of UFC are considered in design process. Three design examples are solved and the results are discussed. Results show that frames, which are designed solely considering the AISC-LRFD limitations, cannot resist progressive collapse, in terms of UFC requirements. Moreover, although the linear static analysis procedure needs the least computational cost with compared to the other two procedures, is the most conservative one and results in heaviest frame designs against progressive collapse. By comparing the results of this work with those reported in literature, it is also shown that the optimization technique used in this paper significantly reduces the required computational effort for design. In addition, the effect of the use of connections with high plastic rotational capacity is investigated, whose results show that lighter designs with resistance to progressive collapse can be obtained by using Side Plate connections in steel frames.

Keywords: progressive collapse; optimal structural design; steel frame; alternate path method

1. Introduction

A chain of element failures which results in structural collapse is called “progressive collapse”. This spread of a local damage to considerable portion of a structure may be very catastrophic. Hence, design of safe and reliable structures (Azar, Hadidi *et al*. 2015) with resistance to progressive collapse is of paramount importance in structural engineering. For this aim, GSA (2013) and UFC (2009) have exclusively provided guidelines to design buildings that can resist progressive collapse. For instance, UFC has provided three design approaches: (i) The “tie force” method which improves the load redistribution capability of the building; (ii) The “enhanced local resistance” method which increases the strength and ductility of columns and/or walls at the first floor; and (iii) The “alternate path” method with which bridging of structure over the critical columns or walls removal scenarios is analytically ensured.

In the alternate path method for evaluating the potential of a building for progressive collapse,

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the possible chain of structural failures is not simulated, and instead, the structural response is checked for the UFC acceptability criteria. The structure is progressive collapse-resistant if these limitations are satisfied, which are specified based on the type of structural analysis as linear static (LS), nonlinear static (NS) or nonlinear dynamic (ND).

Up to now, many researchers have investigated the mechanisms of the progressive collapse (Ettouney, Smilowitz *et al*. 2006, Bažant and Verdure 2007, Sasani and Sagiroglu 2008, Fu 2009, Yang and Tan 2013, Song and Sezen 2013, Gerasimidis 2014). The capacity of 2D steel moment frames using alternate path method and according to UFC and GSA provisions has already investigated by Kim and Kim (2009). Khandelwal, El-Tawil *et al*. (2009) has studied braced steel frames subjected to progressive collapse. Tavakoli and Alashti (2013) have evaluated the progressive collapse potential of seismic steel moment frames using lateral load method. The optimal design of structural systems using different optimization methods can be found in literature (Rafiee, Talatahari *et al*. 2013, Nguyen and Lee 2015, Fedorik, Kala *et al*. 2015, Lee and Shin 2015); however, studies regarding the optimal design against progressive collapse are few. Grierson and Khajehpour (2002) used optimization techniques to achieve a minimum cost design which can withstand progressive collapse. More recently, Liu (2011) used structural optimization for efficient design of steel framed structures against progressive collapse, wherein, alternate path method is used for evaluating the potential for progressive collapse. The effect of vibration suppression devices on the progressive collapse deign of structures can best be found in Kim, Lee *et al*. (2014), Tavakoli, Naghavi *et al*. (2015).

On the other hand, ever increasing value of natural resources and at the same time access to powerful computer processors, urge designers to provide cost-effective structures. Although design of progressive collapse-resistant buildings using alternate path approach is systematic, complexities of this method may result in heavy and expensive structures. To this end, the use of efficient structural optimization techniques seems to be mandatory. The efficiency of optimization algorithm results in two desirable results: one is the minimization of structural cost and the other is the minimization of computational cost. Thus, many researchers have proposed efficient algorithms for structural optimization (Hadidi and Rafiee 2014, 2015, Nigdeli, Bekdas *et al*. 2015, Li and Lu 2015). One of these methods is “Iteration Particle Swarm Optimization” (IPSO) method proposed by Lee and Chen (2007). They added a new term to the displacement vector of particles in classic PSO to prevent the entrapment of search algorithm in local optima. In this way, the premature convergence is prevented and the exploration ability of PSO is improved. IPSO has recently been improved by Mohammadi-Ivatloo, Rabiee *et al*. (2012) using modified weight coefficients.

In this paper, improved IPSO is utilized for optimal sizing design of steel moment frames subjected to progressive collapse. Optimization constraints include Seismic design specifications of AISC-LRFD (2005) code together with progressive collapse provisions of UFC (2009). In order to evaluate the potential for progressive collapse, three LS, NS and ND analysis procedures of alternate path method of UFC (2009) are considered in design process and the results of different procedures are compared. Three examples of planar steel frames with different span lengths and number of stories are optimized. Results show that frames, which are designed solely considering the AISC-LRFD (2005) limitations, do not meet UFC acceptability criteria. Moreover, although the LS analysis procedure needs the least computational cost with compared to NS and ND procedures, is the most conservative one and results in heaviest frame designs against progressive collapse. By comparing the results of this work with those reported in literature and with those obtained using classical PSO algorithm, it is also shown that the IPSO significantly reduces the required computational effort with compared to PSO and genetic (GA) algorithms. Furthermore, the effect of the use of "side plate" connection (as a connection with high plastic rotational capacity) is investigated in this paper, whose results show that lighter designs with resistance to progressive collapse can be obtained by using such connection types. It should be noted that, the effects of Side Plate connections on the progressive collapse resistance of structures has recently been studied in a valuable work by Faridmehr, Osman *et al*. (2015), while, the influence of such connections in the optimal minimum weight design of steel frames has not been investigated so far.

2. Iteration Particle Swarm Optimization (IPSO) algorithm

Particle swarm optimization (PSO) is a stochastic optimization method with both great applicability and simplicity at the same time. In this method, social behavior of flocking birds and fish schooling is simulated. PSO was first developed and formulated by Kennedy and Eberhart (1995), then, Eberhart and coworkers (Shi and Eberhart 1998, Eberhart and Shi 2000) added an inertia factor to the initial formulation, as follows

$$v_{k+1}^i = w\, v_k^i + c_1 r_1 \left(p_k^i - x_k^i\right) + c_2 r_2 \left(p_k^g - x_k^i\right) \tag{1}$$

$$x_{k+1}^i = x_k^i + v_{k+1}^i \tag{2}$$

where, x_k^i and v_k^i are, respectively, the position and the velocity of *i*-th particle at *k*-th iteration; p_k^i is the best ever seen position of *i*-th particle up to iteration *k*. Analogously, p_k^g is the best ever seen position of all the particles up to *k*-th iteration. w is inertia weight; c_1 and c_2 are weights associated with local and global bests, respectively. r_1 and r_2 are random numbers uniformly distributed over [0, 1].

Despite simplicity and efficiency of PSO, it may converge to a local minimum instead of the global one. To prevent this event, many researchers have improved classical PSO. In a study, Lee and Chen (2007) added a new term to the PSO velocity formulation (Eq. (1)) and achieved good results. Velocity vector given by them is

$$v_{k+1}^i = w\, v_k^i + c_1 r_1 \left(p_k^i - x_k^i\right) + c_2 r_2 \left(p_k^g - x_k^i\right) + c_3 r_3 \left(I_k^b - x_k^i\right) \tag{3}$$

where, I_k^b is the best position of particles at *k*-th iteration; w=1, c_1=c_2=0.01, $c_3 = c_1\left(1 - e^{-c_1 k}\right)$ and r_3 is a uniformly distributed random variable from the [0, 1] interval.

On the other hand, weight coefficients used in IPSO have considerable effects on the optimization results. Mohammadi-Ivatloo, Rabiee *et al*. (2012) changed weight coefficients of Eq. (3) as below and showed that these weights result in better optimums

$$w = w_{\max} - \frac{w_{\max} - w_{\min}}{K} k \tag{4}$$

$$c_1 = c_{1i} + \frac{c_{1f} - c_{1i}}{K} k, \qquad c_2 = c_{2i} + \frac{c_{2f} - c_{2i}}{K} k, \qquad c_3 = c_1\left(1 - e^{-c_2 k}\right) \tag{5}$$

where, K is the total number of iterations, whereas, inertia weight is linearly decreased from

w_{max}=0.9 to w_{min}=0.4 during optimization process. The initial and final values of c_{1i}=2.5, c_{1f}=0.5 and c_{2i}=0.5, c_{2f}=2.5 imply linear decrease and increase of c_1 and c_2, respectively. Application of these variable coefficients makes the algorithm to search for global optimum in early iterations while more focus is given on local optimums in the final iterations. Thus, the exploration and the exploitation abilities of the algorithm are improved simultaneously. Due to the high efficiency of the IPSO, improved by Mohammadi-Ivatloo, Rabiee *et al*. (2012), it is used as the optimization tool in this paper.

3. AISC-LRFD specifications

In this study, AISC-LRFD (2005) specifications are used to design structures for regular loading conditions. Load combinations are selected according to ASCE-7 (2006). If axial demand to capacity ratio (DCR) for a column is greater than 0.4, the column should be checked under special load combinations in which earthquake force is multiplied by over-strength factor (Ω_0). Other code requirements such as inter-storey drifts and stability of the structure should also satisfy the limitations defined by ASCE-7 (2006). Seismic design requirements such as shear in beams when plastic hinges are formed at beam ends and length to depth ratio of beams should be checked for the AISC (2005) requirements.

4. UFC alternate path method

Among the three approaches introduced in Introduction, alternate path method checks the ability of structure to bridge over the removed elements. According to UFC (2009), column or wall elements of the first, middle and last stories together with those of stories above the column splice locations are most likely to be removed. However, as a minimum element removal scenario consideration, removal of corner and middle elements of mentioned stories must be examined. In this paper, these minimum mandatory scenarios are considered for design of examples. As mentioned earlier, UFC (2009) defines acceptance criteria for resistance evaluation of buildings against progressive collapse. These criteria are specified for three LS, NS and ND analyses and are different for “Deformation-Controlled” (DC) and “Force-Controlled” (FC) actions. In a moment frame, for example, “bending” and “tension” are DC actions, whereas, “axial force” and “shear” are categorized as FC actions. In the following subsections loading and modeling procedures together with acceptance criteria for alternate path method of UFC are briefly described for different analysis procedures.

4.1 Linear static analysis procedure

This type of analysis used for alternate path method is the simplest one, in which, for DC and FC actions, two separate load combinations are given by UFC. Gravity load combinations for affected spans (i.e., spans above the removed element) are as follows

$$G_{LS} = \Omega_{LS}\left[\left(0.9\ or\ 1.2\right)D + \left(0.5L\ or\ 0.2S\right)\right] \tag{6}$$

where, G_{LS} is the increased gravity loads; D is Dead load including façade loads; L is Live load including live load reduction per ASCE-7; and S is snow load. In addition, in a steel moment frame, for the bays affected by column or wall removal scenario a load increase factor (Ω_{LS}) is used. This factor for DC actions is Ω_{LS}=0.9 m_{min}+1.1, wherein, m_{min} is the smallest of *m*-factors of any primary beams and connections which are directly connected to the columns or walls directly above the column or wall removal location. It should be noted that, structural elements and components that provide the capacity of the structure to resist collapse due to removal of a vertical load-bearing element are classified as primary. Moreover, the *m*-factor accounts for nonlinear deformation capacity of a structural element or component and is calculated based on ASCE-41 (2007) and UFC (2009). For a connection of a structure, however, *m* must be modified on the basis of detailing of continuity plates, the strength of panel zone, the beam span-to-depth ratio, and slenderness of the beam webs and flanges. On the other hand, Ω_{LS}=2 is used for FC actions.

Gravity load combination for the other spans of the building (which are not affected by column removal) is not increased (i.e., Ω_{LS}=1). According to UFC (2009), lateral loads must be applied to the structure in addition to gravity loads. The lateral load applied to a floor/roof level is equal to 0.002×Sum of gravity loads (Dead and Live) acting at that level ignoring load increase factors. Then, a structure is considered to be progressive collapse resistant by LS analysis if

(i) for DC actions in all the components $\phi m Q_{CE} \geq Q_{UD}$ holds; and,

(ii) for FC actions in all the components we have $\phi Q_{CL} \geq Q_{UF}$;

wherein, ϕ is Strength reduction factor; Q_{UD} and Q_{UF} are, respectively, DC and FC actions from LS model; Q_{CE} is Expected strength of the component or element for DC actions; and, Q_{CL} is Lower-bound strength of a component or element for FC actions.

4.2 Nonlinear static analysis procedure

In this analysis method, the effects of nonlinear behavior of structural material are taken into account in modeling; thus, it gives more accurate results compared to LS analysis. Despite its high accuracy, the high computational effort required in this method makes it expensive-to-use compared to LS analysis. In this procedure, to calculate the DC and FC actions, gravity and lateral loads are applied simultaneously. The following increased gravity load combination is applied to those bays immediately adjacent to the removed column and at all the floors above the removed element (i.e., the affected spans) as

$$G_{NS} = \Omega_{NS}\left[\left(0.9\ or\ 1.2\right)D + \left(0.5L\ or\ 0.2S\right)\right] \tag{7}$$

wherein, *D*, *L* and *S* are similar to Eq. (6). Also, to consider dynamic behavior induced by element removal, dynamic increase factor (Ω_{NS}) is applied to gravity loads. This factor is as follows

$$\Omega_{NS} = 1.08 + 0.76 \big/ \left[\min\left(\theta_{acc}/\theta_y\right) + 0.83\right] \tag{8}$$

where, θ_{acc} is acceptable plastic rotation angle and θ_y is yield rotation angle of the primary beams and connections. The values of these angles are given by ASCE-41 (2007) and UFC (2009) codes for the spans affected by element removal. As an example, plastic rotation angle of a WUF (welded unreinforced flange, Fig. 1) moment connection equals 0.0284−0.0004*d*, in which *d* is the height of the beam section, measured in inches. Like *m*-factors, θ_{acc} must be modified for connections based on detailing of continuity plates, the strength of panel zone, the beam span-to-depth ratio, and slenderness of the beam webs and flanges.

Analogous to LS approach, in NS procedure, lateral loads (equal to 0.002×Sum of gravity loads) are applied together with gravity loads. Finally, a structure is considered to be progressive collapse

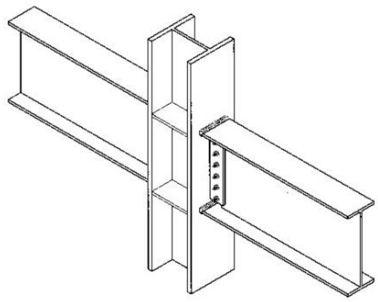


Fig. 1 WUF connection

resistant using NS analysis when DC demands (e.g., plastic rotation of beams) meet acceptance criteria defined by ASCE-41 (2007) and UFC (2009) and FC demands are smaller than the yield strength of sections.

4.3 Nonlinear dynamic analysis procedure

In comparison with LS and NS analyses, this analysis procedure gives the most accurate results. This is because in this approach, dynamic effects of element removal are taken into account in addition to material nonlinearity effects. In the ND procedure, following gravity load combination is applied to the entire structure as

$$G_{ND} = \left(0.9 \text{ } or \text{ } 1.2\right)D + \left(0.5L \text{ } or \text{ } 0.2S\right) \tag{9}$$

wherein, D, L and S are similar to Eqs. (6) and (7).

Similar to previous procedures, lateral loads are applied simultaneously in addition to gravity loads (as 0.002×Sum of gravity loads). To analyze a structure using ND method, first gravity and lateral loads are applied to the intact frame; then, after reaching static equilibrium condition, the critical element is suddenly removed and time history analysis is carried out for a full cycle of vertical motion of span under consideration. Acceptance criteria used for ND procedure are exactly the same as those imposed using NS analysis; except that, in ND procedure dynamic effects of column or wall removal is also modeled.

5. Numerical examples

Three examples of 2D steel moment frames with fixed support at base are optimized in this paper. In order to impose the fabrication conditions on the construction of the frames, beam and column sections are grouped. Doing so, the number of variables is also reduced. All the connection types are WUF. However, in third example, Side Plate connection is used to investigate the effects of the use of connections with high plastic rotational capacity on the optimum design of a frame. The ASTM A992 Steel with lower bound yield strength of 345 Mpa, tensile strength of 450 Mpa, and over-strength factor of 1.1 is used as the material for design purpose. These frame examples are all assumed to belong to seismic design category C with S_{DS}=0.232 (ASCE-7, 2006). The frames are loaded analogous to the example solved by Liu (2011). Moreover, these building examples are all supposed to be of occupancy category (OC) II, which means that alternate path requirements are enough for design of these frames against progressive collapse.

To be concise and for comfort, following abbreviations are used hereafter for optimal designs:

MWD (Minimum Weight Design): optimal frame design obtained when just AISC-LRFD provisions are used as constraints. MWD is used as a base line design which does not take into account the progressive collapse action.

PCLS (Progressive Collapse resistant using LS analysis procedure): optimal frame design obtained when AISC-LRFD provisions and alternate path approach of UFC are applied, while linear static analysis is carried out.

PCNS (Progressive Collapse resistant using NS analysis procedure): optimal frame design obtained when AISC-LRFD provisions and alternate path approach of UFC are applied, while nonlinear static analysis is carried out.

PCND (Progressive Collapse resistant using ND analysis procedure): optimal frame design obtained when AISC-LRFD provisions and alternate path approach of UFC are applied, while nonlinear dynamic analysis is carried out.

In addition, in the optimization problem involved herein, member sections of the frame are discrete design variables which are chosen among American wide flange (W) standard steel sections. Although it cannot not completely quantify the actual expenses associated with construction of a steel building, to provide a minimum cost structural design, total weight of structural elements is considered as the objective function of the optimization problem. The IPSO with 30 particles and 150 iterations is used as optimization algorithm. During optimization, the fitness function is evaluated through structural analysis which is accomplished using OPENSEES.

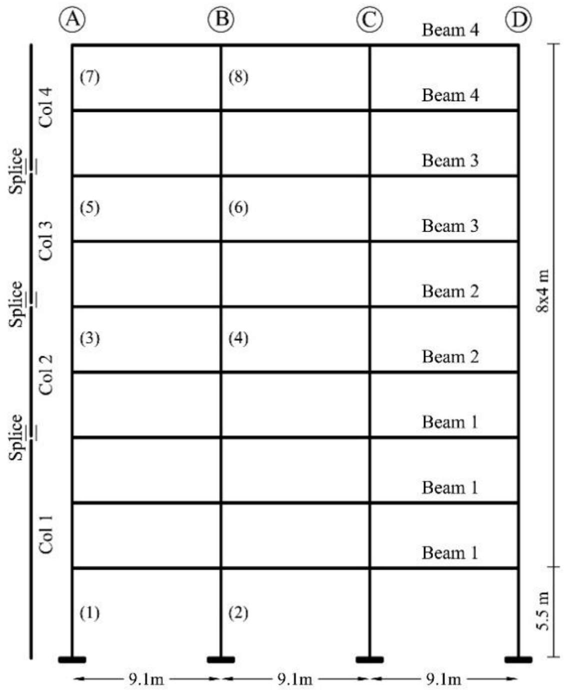


Fig. 2 Nine-storey, three-bay steel frame (Example 1)

Table 1 Optimal designs obtained for nine-storey frame (Example 1) using IPSO algorithm

	Weight (kN)		Groups	1	2	3	4
	GA (Liu 2011)	IPSO					
MWD	440.6	429.5	Col	W18×97	W18×76	W18×76	W18×46
			Beam	W27×94	W24×84	W21×57	W18×46
PCLS	611.6	639.1	Col	W18×211	W18×119	W18×97	W18×76
			Beam	W24×103	W24×76	W24×76	W30×116
PCNS	498.9	487.6	Col	W18×143	W18×106	W18×46	W18×46
			Beam	W24×76	W24×76	W24×62	W24×68
PCND	476.8	472.4	Col	W18×158	W18×97	W18×76	W18×50
			Beam	W24×76	W24×76	W24×55	W24×62

Table 2 Optimal designs obtained for nine-storey frame (Example 1) using classical PSO

	Weight (kN)		Groups	1	2	3	4
	IPSO (Table 1)	PSO					
MWD	429.5	448	Col	W18×106	W18×97	W18×86	W18×40
			Beam	W24×103	W24×76	W24×62	W18×35
PCLS	639.1	656.6	Col	W18×192	W18×130	W18×106	W18×106
			Beam	W21×93	W27×94	W21×93	W30×108
PCNS	487.6	524.7	Col	W18×143	W18×130	W18×130	W18×76
			Beam	W24×76	W24×68	W24×55	W24×84
PCND	472.4	514	Col	W18×158	W18×106	W18×86	W18×86
			Beam	W24×76	W27×84	W24×55	W21×73

5.1 Example 1: a nine-storey, three-bay frame

The geometry, member grouping (eight design variables) and the column splice locations for this example are illustrated in Fig. 2. In this Figure, different column removal scenarios are numbered beside corresponding columns within parentheses. Since the frame is symmetric about vertical centerline, half of the removal scenarios are only shown in the figure. This example has recently been solved by Liu (2011) using genetic algorithm (GA). In this example, column sections are chosen among W18 sections (with total number of 23 sections). Base shear calculated for the frame due to earthquake load is 650 kN and is distributed along the frame height according to ASCE-7 (2006).

This frame is optimized following four different approaches which are described at the beginning of this Section (i.e., MWD, PCLS, PCNS and PCND). Table 1 presents the optimum weight results obtained by using IPSO and GA (Liu, 2011) algorithms for this frame. In this Table, the optimum member sections chosen for beams and columns using IPSO are listed. Results show that in most of cases, IPSO gives lighter frames with compared to GA (Liu 2011); while, the number of structural analyses required for IPSO and GA are 4500 and 6000, respectively. This shows the high efficiency of IPSO in comparison with GA.

Table 2, also, compares the results of classical PSO with that of IPSO. It is seen from this Table

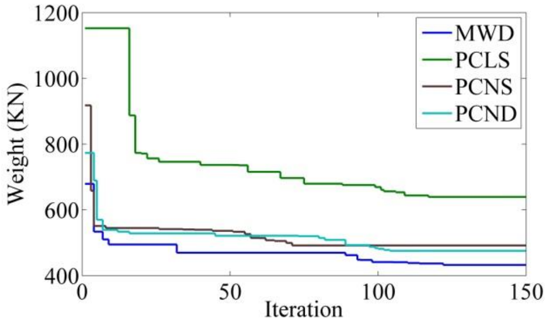


Fig. 3 The convergence histories of optimization using IPSO for nine-storey frame (Example 1)

Table 3 Largest DCR of member groups by LS requirements and corresponding column removal scenarios (Example 1)

		Beam 1	Beam 2	Beam 3	Beam 4	Col1	Col2	Col 3	Col 4
MWD	Scen.	(1)	(3)	(5)	(5)	(1)	(3)	(4)	(5)
	DCR	**1.34**	**1.62**	**2.09**	**2.3**	**1.95**	**1.86**	**1.72**	**1.04**
PCLS	Scen.	(1)	(3)	(3)	(3)	(1)	(2)	(4)	(4)
	DCR	**1.0**	**0.97**	**0.95**	**0.88**	**0.96**	**0.97**	**0.99**	**0.66**
PCNS	Scen.	(1)	(3)	(3)	(3)	(1)	(3)	(4)	(6)
	DCR	**1.27**	**1.21**	**1.3**	**1.51**	**1.37**	**1.24**	**1.38**	**0.55**
PCND	Scen.	(1)	(3)	(3)	(5)	(1)	(3)	(4)	(5)
	DCR	**1.49**	**1.48**	**1.68**	**1.55**	**1.28**	**1.4**	**1.46**	**1.02**

that the classical PSO (with c_1=2, c_2=2 and c_3=0) gives heavier frame layouts with compared to IPSO; whereas, the number of particles and iterations are, respectively, 30 and 150 (i.e., 4500 frame analyses) for both of these algorithms. This shows the high efficiency of IPSO in comparison with classical PSO. The convergence histories of frame optimization procedure using IPSO are depicted in Fig. 3.

On the other hand, results show that frame weight obtained for MWD case is the lightest compared to the other three cases wherein progressive collapse provisions are taken into account. Also, PCLS includes heaviest member sizing so that its weigh is 48.8% heavier than the base line design (MWD). PCNS and PCND designs are also 13.5% and 10% heavier than MWD. Thus, although nonlinear analysis procedures are more time consuming compared to linear one, nonlinear procedures give much lighter layouts with sufficient strength against progressive collapse.

By comparing the obtained designs, it can be seen that MWD does not meet the requirements of any alternate path analysis procedures (LS, NS, and ND), so, AISC-LRFD provisions does not guarantee a progressive collapse resistant design, especially when the frame is designed using optimization methods where the minimum use of material is aimed. PCLS has sufficient strength to meet the nonlinear alternate path requirements and no plastic hinge is formed in the frame under NS and ND analyses, hence, it is the most conservative design. PCNS passes ND analysis requirements while it does not provide enough strength to satisfy LS analysis requirements. At last, PCND does not pass NS analysis requirements and some connections exceed allowable plastic rotation limit by a small percentage; it also doesn't meet the LS analysis requirements. This also should be noted that if a plastic hinge is formed at the beams of these optimal designs, it belongs to the connections and no other plastic hinge is developed along the beam itself.

Table 3 shows the largest DCR ratio of beam and column member groups by examining LS analysis requirements for all optimal designs. The results of this Table indicate that MWD has the largest DCR in the beams of upper stories; hence, these beams are the weakest against progressive collapse events in this specific frame when it is optimized only by regular steel provisions.

In upcoming subsections, two other 2D frames with different geometric conditions will be discussed.

5.2 Example 2: A nine-storey, six-bay frame

In the second example, a steel moment frame with 9 stories and 6 bays is studied. Fig. 4 shows the geometry, member grouping and the column splice locations for this example. As shown in this Figure, the frame is similar to the frame of Example 1 except the number of bays is increased from 3 to 6 whereas the length of each span has been reduced from 9.1 m to 6.5 m. Gravity loads are also similar to the previous problem, but total base shear of earthquake loads equals 930 kN for this example. Columns are all selected among W18 section shapes as it was the case for previous example. Column removal scenarios are very similar to the previous example and there are eight scenarios detected in the frame, considering symmetry.

The results of optimization procedures for this frame are given in Table 4. In this case, PCLS, PCNS, and PCND designs are, respectively, 22.6%, 0.9%, and 6.1% heavier than the MWD base line optimal design. It is seen that weight differences between progressive collapse resistant layouts and the base line design layout (MWD) is very smaller than corresponding values for the previous example. The cause of this reduction may be both the increase in number of bays and decrease in span length (i.e., the increase in density of frame members which results in increased resistance against element removal).

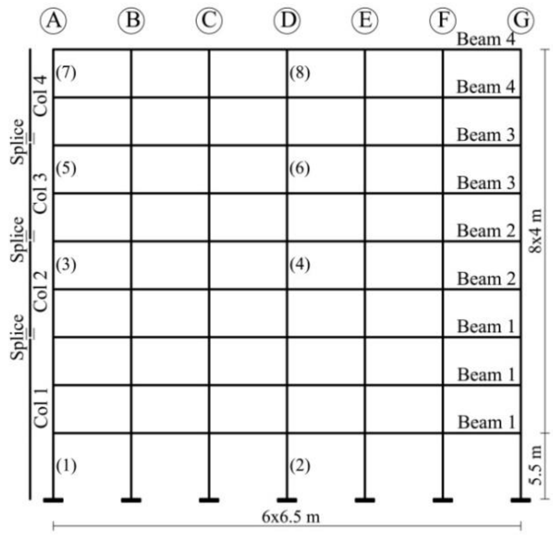


Fig. 4 Nine-storey, six-bay steel frame (Example 2)

Table 4 Optimal designs obtained for nine-storey frame (Example 2) using IPSO algorithm

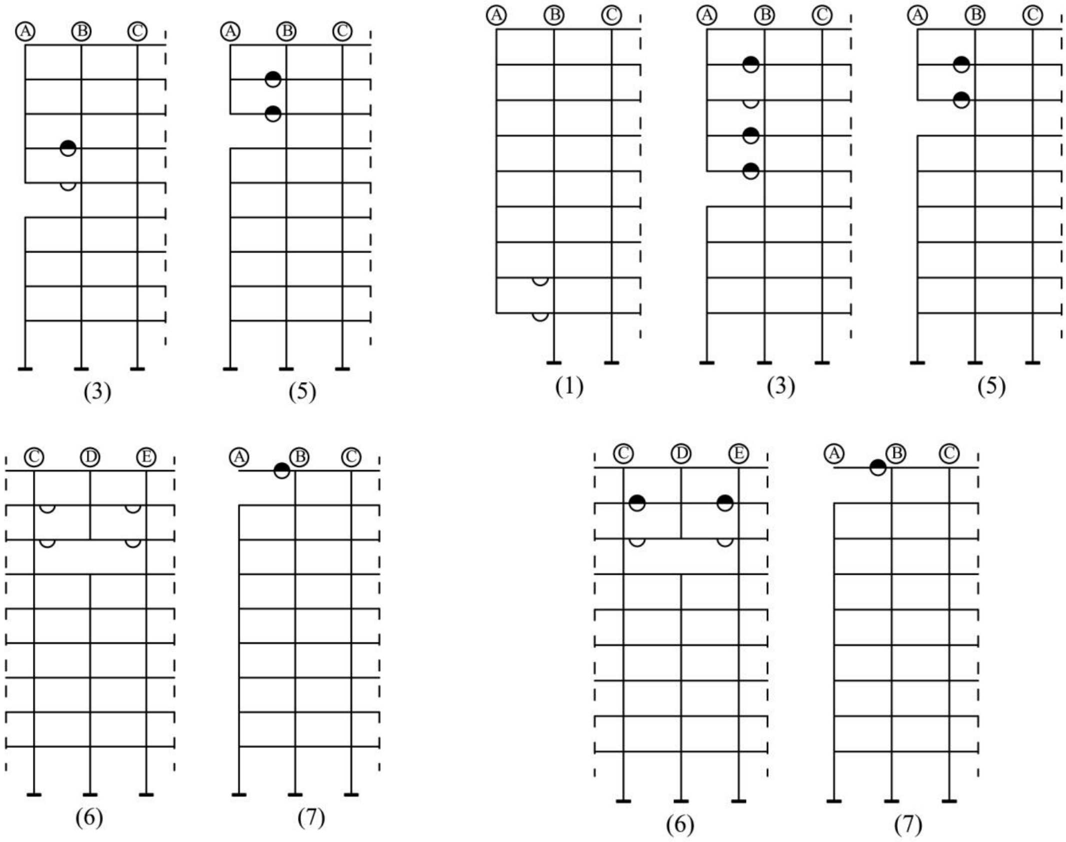
	W (kN)	Groups	1	2	3	4
MWD	489.3	Col	W18×76	W18×65	W18×50	W18×35
		Beam	W24×62	W21×57	W21×50	W16×31
PCLS	599.9	Col	W18×119	W18×86	W18×60	W18×50
		Beam	W21×57	W24×62	W18×46	W21×50
PCNS	493.9	Col	W18×86	W18×65	W18×50	W18×35
		Beam	W24×55	W24×55	W21×44	W18×40
PCND	519.1	Col	W18×97	W18×65	W18×55	W18×40
		Beam	W24×55	W24×55	W18×46	W18×40

Table 5 Largest DCR of member groups by LS requirements and corresponding column removal scenarios (Example 2)

		Beam1	Beam 2	Beam 3	Beam 4	Col 1	Col 2	Co 3	Col 4
MWD	Scen.	(1)	(3)	(5)	(7)	(1)	(3)	(4)	(6)
	DCR	**1.01**	**1.04**	**1.24**	**1.65**	**1.47**	**1.22**	**1.36**	**0.64**
PCLS	Scen.	(1)	(3)	(3)	(5)	(1)	(3)	(3)	(3)
	DCR	**0.96**	**0.93**	**0.98**	**0.98**	**0.96**	**0.96**	**0.91**	**0.42**
PCNS	Scen.	(1)	(3)	(3)	(5)	(1)	(3)	(4)	(6)
	DCR	**1.06**	**1.03**	**1.19**	**1.18**	**1.31**	**1.23**	**1.32**	**0.7**
PCND	Scen.	(1)	(3)	(5)	(5)	(1)	(3)	(4)	(6)
	DCR	**1.09**	**1.08**	**1.14**	**1.26**	**1.18**	**1.24**	**1.23**	**0.51**

To examine the LS criteria for all optimal designs, the largest DCR of member groups are provided in Table 5. Comparison of DCR values of MWD in Table 3 with those in Table 5 shows that the 9-storey frame with 6 bays has DCR values lower than 9-storey frame with 3 bays; for instance, the maximum DCR of the 6-bay frame equals 1.65, whereas, maximum DCR is 2.3 for the 3-bay frame. Similar to the case for Example 1, upper beams have the largest DCR among member groups, which indicates the weakness of these beams against progressive collapse. It is also observed that PCNS and PCND designs do not meet LS analysis requirements. By checking optimal designs for NS and ND requirements, it can also be observed that no plastic hinge is formed along the beam and only connections develop plastic rotations (as it was the case for previous example).

Moreover, the results of nonlinear analyses for this frame are as follows: (i) PCLS is the most conservative layout so that no plastic hinge is formed at the connections or the columns during nonlinear analyses. In this case, the maximum DCR occurs in column removal scenario (1) at column B1 (column of the first storey in axis B) with DCR of 0.7. (ii) MWD does not meet NS and ND requirements. In this design, column removal scenarios (5) and (7) develop plastic rotations at the connections of upper stories which are very larger than acceptance limit defined by the code. Formation of these plastic hinges with large rotations was expected since large DCR of upper storey beams are evident from Table 5. In this case, the maximum DCR of columns (which is observed in column B1) under nonlinear analysis is 1.14 and 1.09 for NS and ND procedures, respectively. As results imply, although the weight of MWD design is close to PCNS and PCND



(a) PCNS design (b) PCND design

Fig. 5 Plastic hinges developed at connections for different column removal scenarios (Example 2)

Table 6 Largest DCR values for beam groups under different column removal scenarios (Example 2)

Analysis Type			Beam Groups			
			1	2	3	4
PCNS	NL Static	Scen.	-	–	(5)	(7)
		DCR	**-**	**–**	**0.22**	**0.5**
	NL Dynamic	Scen.	-	(3)	(5)	(7)
		DCR	**-**	**0.02**	**0.31**	**0.67**
PCND	NL Static	Scen.	-	(3)	(5)	(7)
		DCR	**-**	**0.02**	**0.12**	**0.48**
	NL Dynamic	Scen.	(1)	(3)	(5)	(7)
		DCR	**0.02**	**0.07**	**0.22**	**0.65**

designs, it has not enough capacity to resist progressive collapse. This is because arrangement of structural members and their cross sectional properties in a frame is the most important factor which makes a structure to be resistant against progressive collapse. (iii) PCNS and PCND

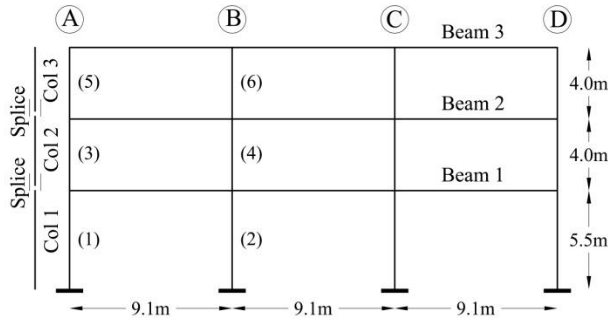


Fig. 6 Three-storey, three-bay steel frame (Example 3)

Table 7 Optimal designs obtained for three-storey frame (Example 3) using IPSO algorithm

	W (kN)	Groups	1	2	3
MWD	77.7	Col	W14×53	W14×48	W14×30
		Beam	W18×40	W16×40	W16×26
PCLS	159.8	Col	W14×74	W14×61	W14×61
		Beam	W24×84	W27×102	W24×84
PCNS	126.8	Col	W14×61	W14×61	W14×61
		Beam	W24×68	W24×62	W24×68
PCND	122.8	Col	W14×61	W14×61	W14×48
		Beam	W21×57	W24×76	W24×62

designs, which are optimized by nonlinear analysis procedures using alternate path requirements, both have been reanalyzed by NS and ND procedures and the results are compared. DCR values of plastic hinge rotations for connections of different beam groups are listed in Table 6. In addition, column scenarios under which plastic hinges have been developed are shown in Fig. 5 (in this Figure, plastic hinges formed under NS analysis are shown by filled semi-circles; whereas, plastic hinges formed under ND analysis are shown by empty semi-circles). It can be seen from the results of Table 6 that in both optimal designs, ND analysis plastic rotations are generally larger than those of NS analysis. Nonetheless, PCNS design meets ND requirements. Analogous to first example, PCND satisfies NS requirements.

5.3 Example 3: A three-storey, three-bay frame

In the third and last example, a three-storey steel moment frame with 3 bays is studied. By eliminating the stories 4 to 9 of the Example 1 this frame is achieved, but the splice locations are different and member is separately grouped in each story (Fig. 6 shows the six design variables of this frame). Gravity loads are same as the first example; while, base shear force is considered to be 194 kN for this frame. Due to the low height of the frame and relatively small axial forces produced in columns, column sections are limited to be chosen among W14 sections (total number of 36 sections). Considering frame symmetry, there are six column removal scenarios which are also shown in Fig. 6 beside columns.

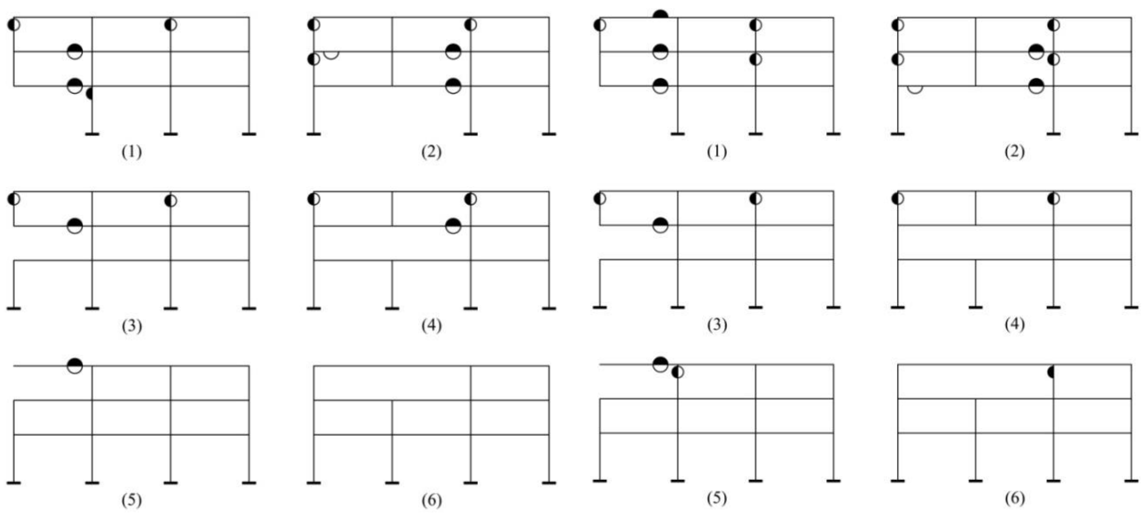
The results of optimization of the frame subjected to different constraints are given in Table 7.

It can be seen from the results that PCLS, PCNS, and PCND optimal designs are respectively %105.6, 63.2%, and 58% heavier than MWD base line design. By comparing these percentages with those of Example 1 it may be inferred that the reduction of frame stories makes progressive collapse resistant optimal designs to weigh much more than MWD frame, which only meets AISC-LRFD provisions.

Table 8 shows largest DCR values under LS analysis for the beam and column member groups. As the results indicate, the beams of top storey of the MWD frame are the weakest (with DCR of 6 in column removal scenario (5)) against progressive collapse. Comparison of mentioned DCR with the corresponding values in Example 1 (with DCR of 2.3) and in Example 2 (with DCR of 1.65) may imply that the resistance of MWD designs against progressive collapse is decreased when the number of stories of the frame is reduced; or in better words, the importance of accounting for progressive collapse effects in optimal design of low-rise buildings is much more significant. PCNS and PCND designs do not meet LS analysis requirements again but like previous examples,

Table 8 Largest DCR of member groups by LS requirements and corresponding column removal scenarios (Example 3)

		Beam 1	Beam 2	Beam 3	Column 1	Column 2	Column 3
MWD	Scen.	(1)	(4)	(5)	(1)	(1)	(2)
	DCR	**2.78**	**3.14**	**6**	**1.82**	**1.3**	**3.23**
PCLS	Scen.	(1)	(1)	(5)	(1)	(2)	(2)
	DCR	**0.95**	**0.97**	**0.96**	**0.96**	**0.86**	**0.43**
PCNS	Scen.	(1)	(1)	(5)	(1)	(2)	(2)
	DCR	**1.51**	**1.39**	**1.32**	**1.25**	**0.78**	**0.45**
PCND	Scen.	(1)	(1)	(5)	(1)	(2)	(2)
	DCR	**1.52**	**1.42**	**1.38**	**1.19**	**0.89**	**0.45**



(a) PCNS design (b) PCND design

Fig. 7 Plastic hinges developed at connections for different column removal scenarios (Example 3)

Table 9 Largest DCR values for beam groups under different column removal scenarios (Example 3)

Analysis Type			Beam Groups		
			1	2	3
PCNS	NL Static	Scen.	(1)	(1)	(5)
		DCR	**0.6**	**0.84**	**0.36**
	NL Dynamic	Scen.	(1)	(1)	(5)
		DCR	**0.52**	**0.61**	**0.34**
PCND	NL Static	Scen.	(1)	(1)	(5)
		DCR	**0.86**	**0.60**	**2.21**
	NL Dynamic	Scen.	(1)	(1)	(5)
		DCR	**0.88**	**0.41**	**1**

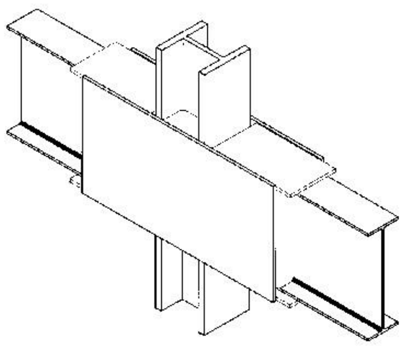


Fig. 8 Side plate connection

DCR of member groups have smaller values compared to that of MWD. As results show in Table 8, columns of second and third floors are within the acceptable limit.

As done for Example 2, all the optimal designs reanalyzed and studied using NS and ND analysis procedures. It is also seen here that all the plastic hinges of beams develop in connections and no hinge is formed along beams. Results of mentioned nonlinear analyses of these optimal designs are briefly discussed as follows: (i) Connections of MWD undergo large plastic rotations which exceed acceptance limits. DCR of column B1 (which is force-controlled because of big axial forces) exceeds the acceptable limit in column removal scenario (1); this value is 1.33 under NS analysis, for example. (ii) PCLS meets both the NS and ND requirements, such that, no plastic hinges are formed in its columns or beams. Column sections are also conservative enough so that none of them are considered force-controlled in different column removal scenarios. (iii) Plastic hinges formed at connections of PCNS and PCND designs are shown in Fig. 7. Also, the largest values of plastic rotation to capacity ratios for different beam groups are given in Table 9. Although in some deformation-controlled columns of this frame plastic hinges are formed, the plastic rotations are very smaller than allowable limits. As the results of Table 9 indicates, PCNS is acceptable base on ND analysis requirements; whereas, PCND does not satisfy NS analysis constraints for the roof beam which has DCR of 2.2 (assuming bilinear moment-rotation behavior) under column removal scenario (5). However, it is the only constraint which does not meet

Table 10 Optimal designs obtained for three-storey frame with Side Plate connections (Example 3) using IPSO algorithm

	W (knew)	Groups	1	2	3
PCLS	137	Col	W14×74	W14×68	W14×61
		Beam	W21×57	W27×84	W24×68
PCNS	97	Col	W14×53	W14×43	W14×43
		Beam	W18×40	W24×55	W24×55
PCND	103.7	Col	W14×61	W14×48	W14×48
		Beam	W14×76	W14×82	W24×55

acceptance criteria.

Up to this point, nonlinear analyses of example frames showed that plastic hinges occur at the connections and not along the beams. To investigate the effects of using connections which have higher capacity to withstand plastic rotations, Side Plate connections (Fig. 8) are used instead of WUF ones in the 3 story frame of Example 3. The *m*-factor of side plate connections is $6.7-0.039d$ and acceptable plastic rotation angle (θ_{acc}) is $0.89-0.0005d$ radians; where d is the depth of section (in inches). The *m*-factor and θ_{acc} should be modified as explained in Section 4. Results of frame optimization using side plate connections under different analysis procedures according to alternate path requirements are displayed in Table 10. Results show that for PCLS, PCNS, and PCND optimal designs, respectively, frame weight is reduced by 14.3%, 23.5%, and 15.6% compared to those with WUF connections (Table 7).

It is also noteworthy that frame weight reduction gained by the use of Side Plate connections does not guarantee that the frame design is “minimum cost”. This is because the construction cost of connections should properly be considered in frame total cost (Hadidi and Rafiee 2015).

6. Conclusions

In this paper, iteration particle swarm optimization (IPSO) algorithm was used for optimal sizing design of steel moment frames against progressive collapse. Seismic design specifications of AISC-LRFD (2005) code together with progressive collapse provisions of UFC (2009) were considered as optimization constraints. In order to evaluate the potential for progressive collapse, three linear static (LS), nonlinear static (NS) and nonlinear dynamic (ND) analysis procedures of alternate path method of UFC were utilized in design process and the results of different procedures are compared. Three examples of planar steel frames with different span lengths and number of stories and/or bays were optimized in this way. The optimization was accomplished in four cases: (i) The specifications of AISC-LRFD code were merely imposed as seismic design constraints (called MWD); (ii), (iii) and (iv) To design against progressive collapse, the provisions of alternate path method of UFC code were considered as constraints in addition to AISC-LRFD specifications, while LS, NS and ND analysis procedures were respectively followed (called PCLS, PCNS and PCND, respectively). Furthermore, the effect of the use of Side Plate connection (as a connection with high plastic rotational capacity) was investigated in this paper.

By comparing the optimization results of this work (using IPSO) with those reported in literature (using genetic algorithm (GA)),

• The superiority of IPSO over GA is demonstrated, such that, it finds lighter frames with compared to GA, while, total number of function evaluations (structural analyses) needed in IPSO is far less than which is required in GA. This is of paramount importance especially when we use nonlinear analysis procedures with high computational costs.

In addition, comparisons of optimal designs obtained under four mentioned cases, lead to following conclusions:

• Results show that frames, which are designed solely considering the AISC-LRFD limitations (i.e., MWD designs), do not meet UFC acceptability criteria for any of LS, NS and ND analysis procedures, and hence, are not resistant against progressive collapse.

• The PCLS optimal design, which is heavier than designs of the other three cases, satisfies the criteria needed in NS and ND procedures. Hence, in all the examples, LS analysis procedure gives the most conservative design. However, it needs the least computational cost with compared to NS and ND procedures.

• In all the examples, the PCNS optimal design does not meet criteria needed in LS procedure, whereas, it is acceptable based on criteria required in ND procedure.

• The PCND optimal design does not satisfy the criteria needed in LS procedure. For two of three examples, it is also not acceptable based on criteria required in NS procedure.

Furthermore, the comparisons show that the reduction in the number of stories of the frame increases the differences between the weight of MWD and that of the other cases. Also, these differences are reduced when the number of bays is increased and at the same time span length is decreased.

Finally, the investigation on the effect of Side Plate connections reveals that lighter designs with resistance to progressive collapse can be obtained by using such connection types. This is because the weak spot of a steel moment frame in dealing with progressive collapse is the formation of plastic hinges at their connections, so, the use of connections with high plastic rotational capacity may help to overcome this drawback.

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