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Discrete optimization of semi-rigid steel frames in second-order analysis

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Traditionally, analysis and design of steel structures consist on simplifying assumptions that connections behave as ideally rigid or pinned. However, these simplifications alter significantly frame behavior, as in the case of beam-to-column connections that, by introducing local effects and imperfections, tend to modify transmitted bending moments and rotations. In this context, the study of a semi-rigid approach is relevant to more realistic modeling and a more economic design. As follows, the present work seeks identification and evaluation of influences from application of semi-rigid joints in first- and second-order analysis on optimal designs of selected structures. To do so, a discrete Particle Swarm Optimization algorithm is implemented, considering in linear and geometric nonlinear analysis the beam-to-column connection semi-rigid effects of four different joint types, following AISC-LRFD specification on structural sizing of steel-framed buildings. Based on analyzed examples, the results obtained demonstrate a distinguished change in structural strength, mainly when semi-rigid connections are assessed in second-order analysis, resulting in significantly stiffer optimal solutions.
Keywords: discrete optimization, geometric nonlinearity, semi-rigid connections, metaheuristic algorithm

INTRODUCTION

Analysis and design of frames seek to numerically simulate the actual behavior of the structure using mathematical models and/or procedures provided in specification standards, in which several simplifications and idealizations of the real model are considered. Such assumptions are necessary to make the equations less complex and enable the solution of these systems. However, some of them significantly alter the structural behavior, as in the case of connections, that would behave as ideally rigid or pinned. The academic community and regulatory agencies showed great interest in studies that revealed a semi-rigid approach as a more pertinent alternative for connection modeling over the last decades.

In light of the rise in use of such structural systems, numerical models are required to reproduce semi-rigid behavior more adequately. Ramires (2010) argues that there is an increasing need to use computational resources as an important tool in engineering, not replacing the engineer's experience, but as a complement to the execution of repetitive and expensive tasks, allowing several structural alternatives to be tested in order to minimize the costs of the structure.

In this sense, the research in metallic structures have presented advances in formulations used in structural analysis. Simões (1996) stated that by treating the connection as semi-rigid a more reliable prediction of frame behavior is obtained, achieving additional economy to design results. Lemes (2018) said that connection members also have resistant capacity and stiffness, which must be studied in order to accurately measure the load capacity of the structure as a whole. Pinheiro and Silveira (2005) tested a semi-rigid connection modeled as a zero length spring element inserted in the intersection point between the beam and the column.

Despite constant advances in the structural optimization area involving frames and trusses, the numerical complexity involved potentiates the development of new contributions, since the addition of new hypotheses in the analysis formulation makes it possible to obtain fresh results, and therefore, allows further exploration of the effects involved, as well as structural configurations closer to real behavior.

Thus, we propose in this work the implementation of a Particle Swarm Algorithm, subject to structural analysis capable of taking into account geometric nonlinearity and semi-ridid connections, which allow for development of discrete optimization, following Load and Resistance Factor Design (AISC-LRFD, 2010) specification on strucural sizing of steel-framed buildings in order to identify and evaluate influences from semi-rigid connections in linear and nonlinear analyses on optimal design of structures widely available in the literature.

The developed computational procedures, obtained results and discussion are presented in the following sections. Computational procedure is divided into finite element formulation for analysis of steel frames considering semi-rigid connections, introduction to Particle Swarm Optimization algorithm and the objective function of a discrete variable optimization problem. Next, three steel planar frames are subject to analysis accounting for geometric nonlinearity and semi-rigid connections, followed by the optimum designs obtained for the last two and discussion.

Analysis of plane frames and connection stiffness

Structural analysis consists in the commonly used calculation of structural stiffness, being determined by the relationship between the nodes joining the beam and column members, as in Eq. (1):

$$\{F\} = [K]\{d\} \tag{1}$$

where the applied forces, F, and resulting displacements, d, are related to the structural global stiffness matrix, K, which is assembled from the combined elements stiffness.

According to Geschwindner (2000), when the deformed configuration of the structure is determined, the resulting analysis is a second-order elastic analysis. When the influence of member curvature is included, it is referred as the P- δ effects and when the sidesway effects are included it is referred as P-Δ effects.

As presented by Cook *et al* (1989), when elastic (first-order) and geometric (second-order) stiffnesses are both taken into account, the total or effective stiffness matrix accounts for the stiffening or weakening effect of axial load on bending stiffness. Allied to this, as stated by Porteous and Kermani (2007), if the connections exhibit semi-rigid rather than fully fixed rotational behavior, there will be a reduction in the stiffness of the structure. In order to model this, a rotational spring with zero length element is attached to each end of the beams, being added to the structural analysis in this work by the fixity factor. Cunningham (1990) proposes that the fixity factor, in Eq. (2),

$$\alpha_j = \frac{\phi_1}{\phi_2} = \frac{1}{1+\frac{3EI}{S_j L}}, j = 1,2 \tag{2}$$

is a ratio of rotations of the end of the beam (ϕ_1) with a unit end moment divided by that of the beam plus the connection (ϕ_1) for the same moment, shown in Fig. 1, with S_j being the end-connection rotational stiffness and *EI/L* the flexural stiffness of the adjacent beam-column member.

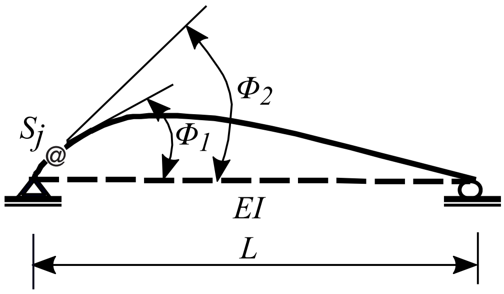


Figure 1 – Connection stiffness and fixity factor relationship.

Consequently, the elastic matrix and the geometric matrix are modified by the respective semi-rigid correction matrices, resulting in the one in Eq. (3), allowing for a member with semi-rigid connections with rotational stiffness S_1 and S_2, where K_E, K_G, K_{ce} and K_{cg} are the corresponding elastic, geometric, and semi-rigid correction matrices, respectifully, with K_i^{SR} being the total stiffness matrix, as proposed by Xu (2001):

$$[K_i^{SR}] = [K_E][K_{ce}] + [K_G][K_{cg}] \tag{3}$$

Discrete Variables Formulation for Design Optimization

The discrete optimization technique consists of the optimum design problem mathematical formulation, where the discrete variables can be mathematically stated as in Eq. (4):

$$\begin{aligned}
&\text{find}: \ \mathbf{X} = \left[x_1, x_2, \cdots, x_{nc}\right] \\
&\text{which minimizes}: \ f_{obj}(\mathbf{X}) = \sum_{i}^{\mathrm{N_m}} A_i L_i \rho_i \quad i = 1, \cdots, \mathrm{N_m} \text{ for all members} \\
&\text{subject to the constraints}: \quad C_k^{\sigma} \leq 0 \quad k = 1, \cdots n_c \\
&\qquad\qquad\qquad\qquad\qquad\quad C_r^{\delta} \leq 0 \quad r = 1, \cdots n_s \\
&\text{where}: \ 1 \leq A_i \leq m_s \quad i = 1, \cdots n_g
\end{aligned} \tag{4}$$

in which $\sum$ is the summation of all structural members N_m, each with their cross-sectional area A_i, length L_i and material density ρ. The vector $\mathbf{X}$ correspond to the discrete design variable set selected from the standard W-section list that contains the number of discrete values for these variables. *ns* and *nc* are the number of stories and the number of beam-columns, respectively. A_i is selected from the standard steel sections table (W-shaped sections given in Steel Sections Database by AISC, 2016) and *ms* shows the total number of W-shaped sections considered in the design for group *i*. The constraints are formulated by the mechanical resistance C_k^σ, and displacement requirements C_r^δ, where serviceability and strength constraints are designed following LRFD specification (AISC, 2010), showed in Eq. (5).

$$C_k^\sigma = \begin{cases} \dfrac{P_u}{2\phi_c P_n} + \dfrac{M_{ux}}{\phi_b M_{nx}} - 1 & \text{if } \dfrac{P_u}{\phi_c P_n} < 0.2 \\ \dfrac{P_u}{\phi_c P_n} + \dfrac{8}{9}\dfrac{M_{ux}}{\phi_b M_{nx}} - 1 & \text{if } \dfrac{P_u}{\phi_c P_n} \geq 0.2 \end{cases}$$
$$C_r^\delta = \frac{\delta^*}{\delta_{ru}} - 1 \qquad \text{where } \delta^* = (\delta_r - \delta_{r-1}) \tag{5}$$

where the strength constraints, $C_k^\sigma \leq 0$, for members subjected to axial force and bending, are expressed in terms of P_u, the required strength (tension or compression); P_n, the nominal axial strength (tension or compression); ϕ_c, the resistance factor ($\phi_c = 0.9$ for tension, $\phi_c = 0.85$ for compression); M_{ux}, the required flexural strength in the x direction; M_{nx}, the nominal flexural strength in the x direction and ϕ_b, the flexural resistance reduction factor ($\phi_b = 0.90$). The displacement constraints, $C_r^\delta \leq 0$, representing the inter-story drift of a multi-story frame are expressed in terms of δ_r and δ_{r-1}, the lateral deflection of two adjacent story levels; δ_{ru}, the allowable lateral displacement.

The in-plane effective length factor, K_x, is calculated for unbraced frames from Eq. (6), from Dumonteil (1992). The relative stiffness ratios (G_A and G_B) are modified to allow flexibly connected ends by means of the semi-rigid modification factor (α_g), available in Dhillon and O'Malley (1999).

$$K_x = \sqrt{\frac{1.6G_AG_B + 4.0(G_A + G_B) + 7.5}{G_A + G_B + 7.5}}$$
$$G_{A,B} = \frac{\sum I_c/L_c}{\sum \alpha_g I_g/L_g} \tag{6}$$
$$\alpha_b = 1/(1 + 6EI/S_jL)$$

where, the not previously defined unknowns, the sufixes *c* and *g* refer to the adjacent columns and girders (beams) of the respective end nodes *A* and *B* of a beam-column member.

Last but not least, the development of theories that aims at optimizing problems was based on mathematical programming. The basic idea shared by these methods is that the gradient of the function to be optimized has important information to quickly find an optimum solution for a specific problem. However, when dealing with highly nonlinear, non-convex, non-differentiable, (i.e., problems where the mass reduction conflicts with stress/displacements constraints and especially discrete design variables), these methods may present some convergence difficulties. Like so, we use Particle Swarm Optimization (PSO) algorithm to deal with the discrete optimization formulation.

Particle Swarm Optimization

In the words of Hare, Nutini and Tesfamarian (2013), Particle Swarm Optimization algorithms mimic animal flocking behaviors. These algorithms, originally accredited to Eberhart and Kennedy (1995), Shi and Eberhart (1998), have a similar stochastic nature to Genectic Algorithms (GAs) and like GAs, work with a set of potential solutions and the concept of "fitness". Essentially, particles (candidate solutions) move around the search space, iteratively improving their fitness value according to a given quality measure. Each particle is influenced by its neighbor. Simple mathematical formulas for position and velocity are used to move the particles through the d-dimensional hyperspace, accelerating towards "better" solutions.

The original PSO algorithm was developed for continuous design variables. Dogan and Saka (2012) proposed that to be able to use the method for discrete design variables some adjustments are required to be carried out. Firstly the discrete values among which the design variables x_i are to be selected in set $\mathbf{X}$ are arranged in ascending sequence. The sequence number of these values is then treated as the design variable instead of x_i itself. For example in a design set which consists of 272 values, the sequence numbers from 1 to 272 are the main design variables. At any stage of design cycle, once a sequence number is generated by the algorithm, then the real value of the design variable which corresponds to this sequence number is easily taken from the discrete set.

NUMERICAL RESULTS

Firstly, a structural framework commonly found in the literature for benchmarking purposes is presented in this section, allowing for the developed analysis routines validation. Secondly, structural analysis of two different planes are made for further investigation of the combined second-order effects and semi-rigid connections on the structural response. Finally, the discrete optimization of these frames is developed.

Structural Analysis

The two-bay, four-story frame, in Fig. 2, was analyzed by Bhatti and Hingtgen (1995), with results provided from De Mello (1999), in second-order elastic theory. The studied connection types are typically rigid and extended end-plate, for a semi-rigid one, with the latter assumed to maintain initial connection stiffnes, $S_j = 88,884,194.6$ N.m/rad.

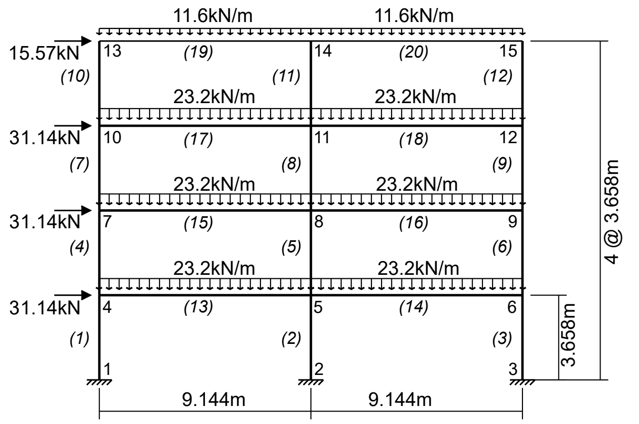


Figure 2 – Two-bay, four-story frame.

By means of Eq. (2), the associated fixity factors to the beam-to-column spring models are calculated, resulting in the ones in Tab. 1 below, along with the section profiles and properties for the beams and the columns, respectively. The Young's Modulus of steel used here is $E = 199.955$ GPa (29,000ksi), with yielding stress $\sigma_y = 248.2$ MPa (36 ksi).

Table 1 – Two-bay, four-story frame member properties and fixity factors

Fixity Factor	Beams	Section profile	A (mm^2) [in^2]	Ix ($\times 10^4$ mm^4) [in^4]
0.86272	13, 14, 15, 16, 17, 18	W16 × 40	7225.8 [11.8]	21560.8 [518]
0.91795	19, 20	W14 × 30	5709.7 [8.85]	12112.3 [291]
Fixity Factor	Columns	Section profile	A (mm^2) [in^2]	Ix ($\times 10^4$ mm^4) [in^4]
—	1, 2, 3, 5, 8, 11	W12 × 79	14967.7 [23.2]	27554.5 [662]
—	4, 6, 7, 9, 10, 12	W12 × 65	12322.6 [19.1]	22185.1 [533]

The obtained results are checked against the original authors work in Tab. 2, for the lateral displacement at the windward beam-column joints, followed by those for absolute maximum moments at the columns. Table 2 agrees that the developed routines are capable of modeling the behavior of frames well accounting for the effects studied in this paper. In addition, it shows the moment redistribution that occurs when connection flexibility is considered, especially noted at the upper stories and base columns.

Structural Optimization

It is sought in this section to validate the implemented optimization algorithm, as well as to develop additional considerations of the studied frames structural response when selected beam-to-column connections are assembled. The first problem is the known benchmark two-bay, three-story frame of Togan (2012), as in Fig. 3a.

The design section profiles and total structural weight are evaluated against several authors, with the objective function being restrained by AISC-LRFD specification without displacement constraints. Beams are selected from 267 W-section profiles and columns are restricted to the W10 series only, configuring a discrete optimization problem. The out-of-plane effective length factor, K_y, is assumed equal to 1.0 for columns and to $1/6$ of span length for beams. Table 3 shows the

Table 2 – Four-story, two bay frame lateral displacements and bending moments.

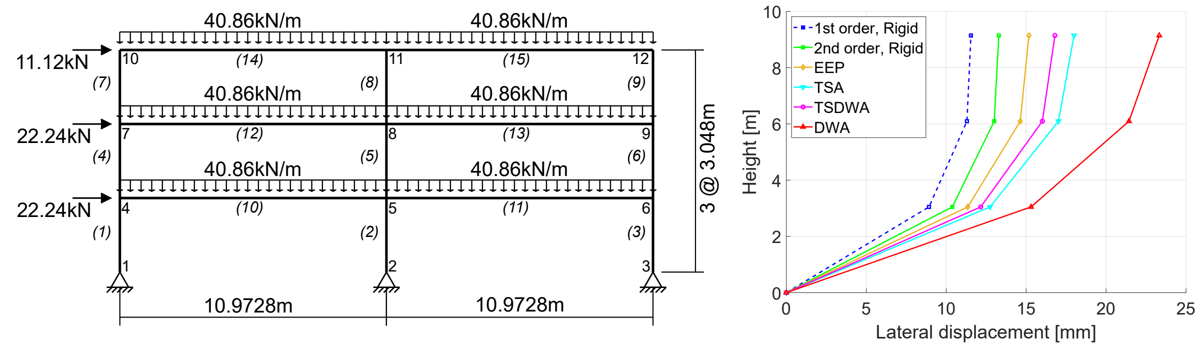
Displacements (mm) [in]						
	Rigid connections			Semi-rigid connections		
Node	Bhatti	Mello	Present work	Bhatti	Mello	Present work
4	6.83	6.83	6.84	7.72	7.72	7.72
	[0.269]	[0.269]	[0.269]	[0.304]	[0.304]	[0.304]
7	16.84	16.84	16.82	19.58	19.58	19.56
	[0.663]	[0.663]	[0.662]	[0.771]	[0.771]	[0.770]
10	23.90	23.90	23.89	28.35	28.32	28.29
	[0.941]	[0.941]	[0.941]	[1.116]	[1.115]	[1.114]
13	28.17	28.17	28.15	33.73	33.71	33.68
	[1.109]	[1.109]	[1.108]	[1.328]	[1.327]	[1.326]
Bending moments ($\times 10^{-1}$ kN.m) [in-kip]						
	Rigid connections			Semi-rigid connections		
Column	Bhatti	Mello	Present work	Bhatti	Mello	Present work
1	6.033	6.033	6.033	6.993	7.095	7.084
	[534]	[534]	[534]	[619]	[628]	[627]
2	10.812	10.812	10.801	11.479	11.479	11.468
	[957]	[957]	[956]	[1016]	[1016]	[1015]
3	13.580	13.580	13.580	14.168	14.066	14.066
	[1202]	[1202]	[1202]	[1254]	[1245]	[1245]
4	5.141	5.141	5.141	4.564	4.350	4.361
	[455]	[455]	[455]	[404]	[385]	[386]
5	7.423	7.423	7.412	7.332	7.332	7.321
	[657]	[657]	[656]	[649]	[649]	[648]
6	12.450	12.450	12.440	12.021	11.829	11.818
	[1102]	[1102]	[1101]	[1064]	[1047]	[1046]
7	6.948	6.948	6.948	6.677	6.474	6.463
	[615]	[615]	[615]	[591]	[573]	[572]
8	5.355	5.344	5.344	5.604	5.604	5.593
	[474]	[473]	[473]	[496]	[496]	[495]
9	11.626	11.626	11.626	11.354	11.140	11.140
	[1029]	[1029]	[1029]	[1005]	[986]	[986]
10	7.954	7.954	7.943	7.762	7.592	7.581
	[704]	[704]	[703]	[687]	[672]	[671]
11	2.260	2.260	2.260	2.542	2.542	2.530
	[200]	[200]	[200]	[225]	[225]	[224]
12	9.264	9.264	9.242	9.174	9.050	9.027
	[820]	[820]	[818]	[812]	[801]	[799]

absolute maximum bending moments obtained in second-order analysis considering different connection types compared to obtained in first-order analysis with rigid connections. The optimum results obtained in first- and second-order analysis, considering rigid connections, are in Tab. 4.

Displacements in Fig. 3b, subject to first-order rigid, and second-order rigid and semi-rigid analysis, show that P-Δ effects are not as present, due to the small displacements noticed. Figure 4 shows the objetive function domain relative to the discrete variables considering first-order analysis (Fig. 4a) and second-order analysis (Fig. 4b). Figure 5 exhibits the objective function domain relative to columns (Fig. 5a) and beams (Fig. 5b).

It may be noted from Fig. 4 that the discrete optimization entails in a highly nonlinear objective function, such that the fluctuation noticed reveals the presence of local minima from the discretized domain, particularly defined by 'gaps' due to variable discontinuity, potentializing the application of heuristic methods, such as PSO, for this type of problem, as is clear from Fig. 5, where the global minimum was successfully found by the optimization algorithm in both cases.

Also, in Tab. 4 and Fig. 5, the optimum solutions found for the problem with rigid connections, both from first- and second-order analysis, were the same, and are quite close to the referenced authors ones. This result was already expected,



(a) Two-bay three-story frame. Adapted from Togan, 2012.

(b) Lateral deflection of the two-bay, three-story frame.

Figure 3 – Lateral deflection of the two-bay, three-story frame.

Table 3 – Two-bay, three-story frame maximum bending moments at the column nodes.

Bending moments ($\times 10^{-1}$ kN.m)						
	First-order	Second-order analysis				
Column	Rigid	Rigid	EEP	TSA	TSDWA	DWA
1	4.401	3.443	3.166	2.799	2.548	1.201
2	6.578	7.523	7.643	7.739	7.805	8.076
3	14.771	15.549	15.404	15.152	14.941	14.016
4	15.892	15.980	15.551	15.009	14.585	12.828
5	2.984	3.233	3.442	3.611	3.729	4.205
6	16.982	16.893	16.139	15.338	14.735	12.971
7	19.981	19.977	19.762	19.236	18.764	16.487
8	1.001	1.034	1.140	1.233	1.301	1.601
9	21.228	21.266	21.173	20.751	20.354	18.410

once the bending moments did not vary considerably in any of the two formulations, as evidenced in Tab. 3, neither the displacements showed much of an increase, as Fig. 3b shows.

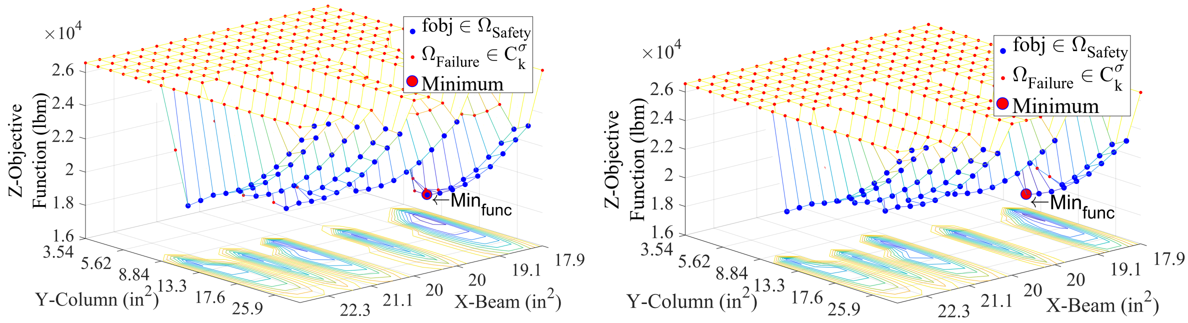
Once the optimization is properly validated, the connection flexibility is assessed in the analysis, by means of four different semi-rigid connection types, available in Chen, Kishi and Komuro (2011) database. The chosen types are assumed to retain initial connection stiffness, consisting in extended end-plate (EEP), top-and-seat angle (TSA), top-and-seat angle with double web angle (TSDWA), and double web angle (DWA), in this case, types V-9, IV-13, III-32, II-17, respectively.

Comparing the optimum designs for several connection types in second-order analysis, in Tab. 5, it is noticeable the weight reduction that outcomes from the more flexible joints. Surely, bending moments redistribution and P-δ effect are dominant in this problem, and thus responsible for the beam cross-sectional area downsizing and column stiffening. Nevertheless, it is noticed that distinct optimum solutions were obtained for all connections, but TSA, associated to the repercussion of different stiffnesses in the structural configuration, arising as a consequence from the moment resistance capacity of each type of connection (see Tab. 3) and the available solutions found in the discrete set.

Table 4 – Benchmark designs for two-bay, three-story frame.

		Benchmarks			Present work	
Elem. group	Members	GA Pezeshk	TLBO Togan 2012	HS Degertekin	PSO 1^{st}-order	PSO 2^{nd}-order
1 (Beam)	10-15	W24 × 62	W24 × 62	W21 × 62	W24 × 62	W24 × 62
2 (Column)	1-9	W10 × 60	W10 × 49	W10 × 54	W10 × 45	W10 × 45
Weight (kg)		8,524	8,069	8,297	7,916	7,916
Weight (lbm)		18,792	17,789	18,292	17,453	17,453

Another consideration for the semi-rigid effects is the total structural deflection magnified by connection flexibility. In a recent study, Borges, Vetorazzi and Esposito (2018) measured the influence in the objective function of a semi-rigid frame in first-order analysis. To further investigate this phenomenon, the second frame optimized is a one-bay, ten-story planar frame subject to single-load case as shown in Fig. 6. This frame consists of 30 members and they are organized

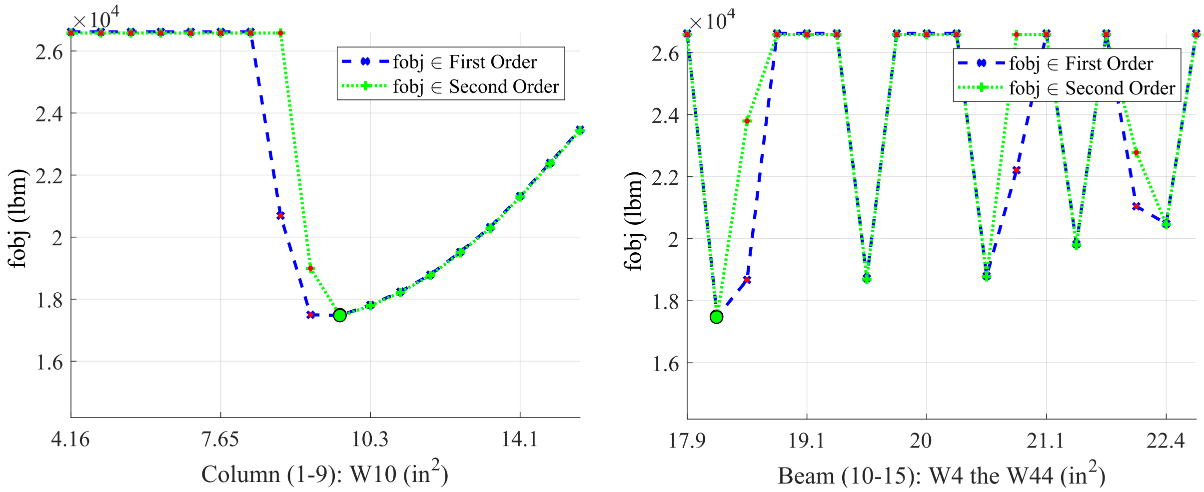


(a) Objective function by column/beam using first-order analysis. (b) Objective function by column/beam using second-order analysis.

Figure 4 – Objetive function of the two-bay three-story frame.

Table 5 – Comparison of optimum designs for different connection types in second-order analysis for two-bay, three-story frame.

	Semi-rigid connections			
Elem. group	DWA	TSDWA	TSA	EEP
1. Beams	W21 × 48	W24 × 55	W24 × 62	W24 × 55
2. Columns	W10 × 49	W10 × 45	W10 × 45	W10 × 49
Weight (kg)	6,674	7,252	7,916	7,404
Weight (lbm)	14,714	15,988	17,453	16,324



(a) Objective function × column. (b) Objective function × beam.

Figure 5 – Objetive function by column/beam of the two-bay, three-story frame.

into 9 groups due to fabrication conditions (explicit in Tab. 6). Originally designed following AISC-LRFD specification, the frame has also a displacement constraint, with allowable inter-story drift no greater than 1/300 of the story height. The out-of-plane effective length factor, K_y, is specified to be $1/5$ of span length for beams and to be unity for columns.

The graphic in Fig. 7 relates the deflections obtained in first-order and second-order analysis, for rigid and semi-rigid connections, to story heights. The P-Δ effect combined with semi-rigid connection unstiffening have a considerable influence in structure response. It is noteworthy to account for these effects on the structural design, ensuring stability.

Table 6 presents the optimum designs obtained for the ten-story frame, compared to a number of authors who previously had studied it, considering rigid connections, while Tab. 7 brings a comparison of optimal solutions, when three different connection types, available in Hayalioglu and Degertekin (2005), are assembled in first- and second-order analyses. The semi-rigid connections are assumed to maintain initial stiffness, and to model extended end-plate (EEP), top-and-seat angle with double-web angle (TSDWA), top-and-seat angle without web-angle (TSA) and double-web angle (DWA), with

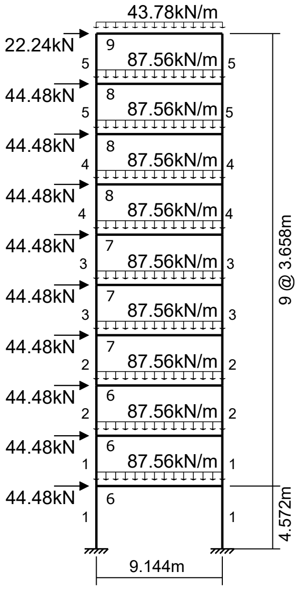


Figure 6 – Ten-story frame. Adapted from Togan, 2012.

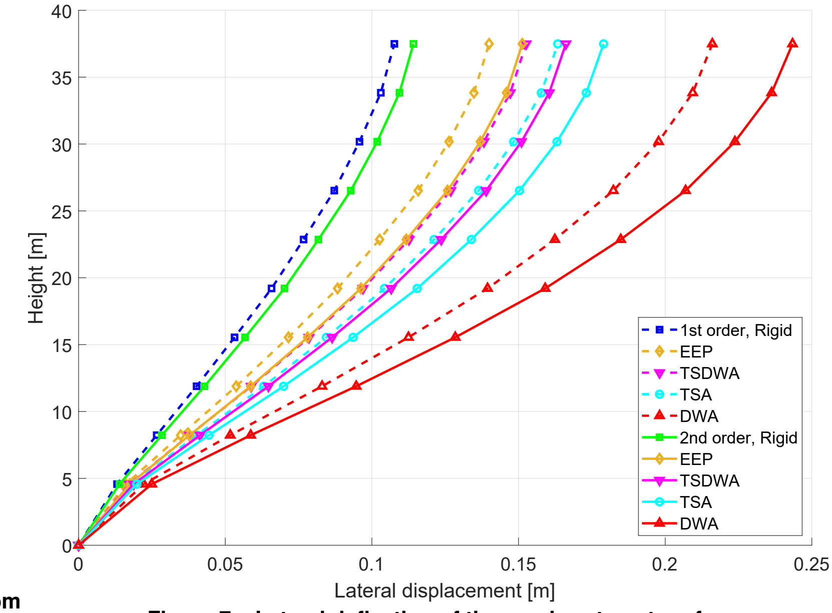


Figure 7 – Lateral deflection of the one-bay, ten-story frame.

Table 6 – Benchmark designs for one-bay, ten-story frame.

	Benchmarks			Present work	
Elem. group	GA Pezeshk	TLBO Togan 2012	EHS Maheri 2014	PSO 1^{st}-order	PSO 2^{st}-order
1 (column 1-2S)	W14 × 233	W14 × 233	W14 × 159	W14 × 176	W14 × 159
2 (column 3-4S)	W14 × 176	W14 × 176	W14 × 730	W14 × 120	W14 × 120
3 (column 5-6S)	W14 × 159	W14 × 145	W14 × 61	W14 × 90	W14 × 109
4 (column 7-8S)	W14 × 99	W14 × 99	W12 × 87	W14 × 61	W14 × 68
5 (column 9-10S)	W12 × 79	W12 × 65	W14 × 283	W14 × 48	W14 × 48
6 (beam 1-3S)	W33 × 118	W30 × 108	W24 × 68	W36 × 135	W40 × 149
7 (beam 4-6S)	W30 × 90	W30 × 90	W19 × 99	W33 × 130	W33 × 118
8 (beam 7-9S)	W27 × 84	W27 × 84	W21 × 111	W30 × 99	W27 × 102
9 (beam 10S)	W24 × 55	W21 × 44	W33 × 201	W21 × 44	W21 × 44
Weight (kg)	29,545	28,038	26,995	26,787	27,120
Weight (lbm)	65,136	61,813	59,514	59,056	59,789

respective rotational stiffness, $S_{EEP} = 395 \times 10^6$ N.m/rad, $S_{TSDWA} = 282 \times 10^6$ N.m/rad, $S_{TSA} = 226 \times 10^6$ N.m/rad and $S_{DWA} = 113 \times 10^6$ N.m/rad. The DWA connection is truly flexible, and would return extremely expensive solutions, thus it was judged to be infeasible for this type of problem. In addition, Fig. 8 shows the solutions considering all connection types tested, comparing the results in weight for linear and nonlinear analyses.

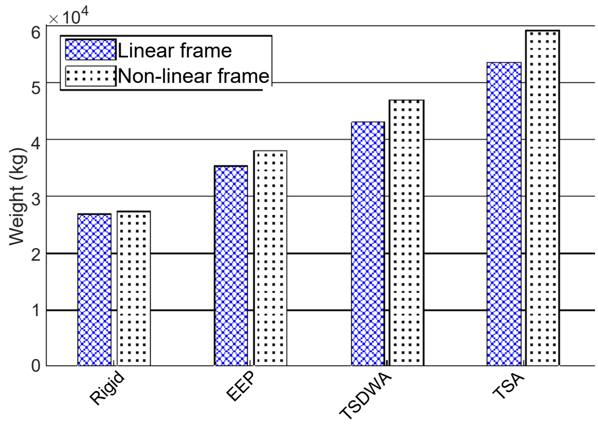


Figure 8 – Comparison of weight results for one-bay, ten-story frame.

Table 7 – Comparison of optimum designs for different connection types in first- and second-order analysis for one bay, ten-story frame.

	First-order analysis			Second-order analysis		
Elem. group	TSA	TSDWA	EEP	TSA	TSDWA	EEP
1 (column 1-2S)	W14 × 455	W14 × 342	W14 × 257	W14 × 665	W14 × 398	W14 × 257
2 (column 3-4S)	W14 × 426	W14 × 311	W14 × 211	W14 × 455	W14 × 370	W14 × 233
3 (column 5-6S)	W14 × 176	W14 × 159	W14 × 132	W14 × 257	W14 × 193	W14 × 145
4 (column 7-8S)	W14 × 90	W14 × 90	W14 × 74	W14 × 109	W14 × 82	W14 × 99
5 (column 9-10S)	W14 × 43	W14 × 48	W14 × 48	W14 × 43	W14 × 43	W14 × 53
6 (beam 1-3S)	W44 × 290	W44 × 230	W40 × 199	W44 × 262	W44 × 230	W44 × 230
7 (beam 4-6S)	W44 × 230	W36 × 194	W40 × 149	W40 × 199	W40 × 167	W36 × 160
8 (beam 7-9S)	W27 × 102	W27 × 84	W27 × 94	W30 × 90	W30 × 124	W24 × 84
9 (beam 10S)	W21 × 48	W21 × 44	W21 × 44	W21 × 83	W21 × 44	W21 × 44
Weight (kg)	53,389	43,093	35,143	58,902	46,729	37,854

When it comes to optimum solutions obtained for one-bay, ten-story frame, the results obtained by the implemented formulation are fairly close to those from the referred authors, further validating the developed routines. Being a multi-variable, highly nonlinear problem, heuristic methods once again become a competitive alternative when searching for optimized designs, even more so, considering the computational cost of deterministic or brute-force methods.

Unlike what occurred in the previous problem, the second-order effects resulted in a different optimum solution when compared to the one obtained through linear analysis, even in the case of ideally rigid connections, which strenghtens the importance of taking those effects into account for problems with this characteristic, i.e., tall and slender frames subject to high lateral loading.

Figure 8 and Tab. 7 further denounce the semi-rigidity influence on structural response. Regardless of the analysis performed, remarkable variations in weight are noticed when compared to ideally rigid connections for all three semi-rigid types tested. Besides, the more flexible the connection is, the stiffer the frame must be to meet the imposed constraints, particularly the displacement one, as Fig. 7 makes clearly visible, with the lateral deflection showing considerable increment from the most flexible connections, which requires larger cross sections especially for columns, in order to remain within what is established in design phase.

Summary

The results presented in this work demonstrate that, in adopting a typically rigid connection, a less realistic representation of the overall frame response is obtained, while the most appropriate connection type should be chosen upon strict analysis. Yet, by introducing local effects and imperfections, semi-rigid connections did amplify the effects of geometrical nonlinearity. The four different connection types tested had distinct repercussion on analysis results, and when combined with second-order effects, altered significantly the objective function behavior of the studied examples, impacting directly in the frames weight.

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