Steel frames optimization considering beam-column joint stiffness and geometric constraints*

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Abstract. Structural optimization is an active research branch in engineering, especially dealing with complex and concomitant aspects likewise in seismic design. Capacity design criteria for seismic design and detailing must be respected, e.g. according to the "strong-column weak-beam" principle. In steel structures, the choice of a specific beam-column joint typology may strongly affect its behavior under horizontal actions. In this study, the authors investigated the role of beam-column joint stiffness within an optimization paradigm related to steel structure frames. Specifically, the authors adopted simplified modeling assumptions for analysis under lateral loads in the Python environment and Computer and Structures inc. SAP2000 finite element software. Indeed, the main focus hitherto is oriented toward the problem definition accounting for geometric constraints and beam-column rotational stiffness capacity. Future investigations will adopt more realistic modeling procedures accounting for the typical non-linearities involved during strong dynamic actions.

Keywords: steel structures · geometric constraints · beam-column joints · meta-heuristic algorithms · optimization.

1 Introduction

In the civil engineering field, structural optimization procedures play a crucial and significant role nowadays with the final purpose of minimizing the various costs items involved in the design, construction, and maintenance phases [18,16,15]. Three main branches can be evidenced within the structural optimization field. Size optimization involves the definition of the optimal size of structural elements within the domain under investigation, whereas

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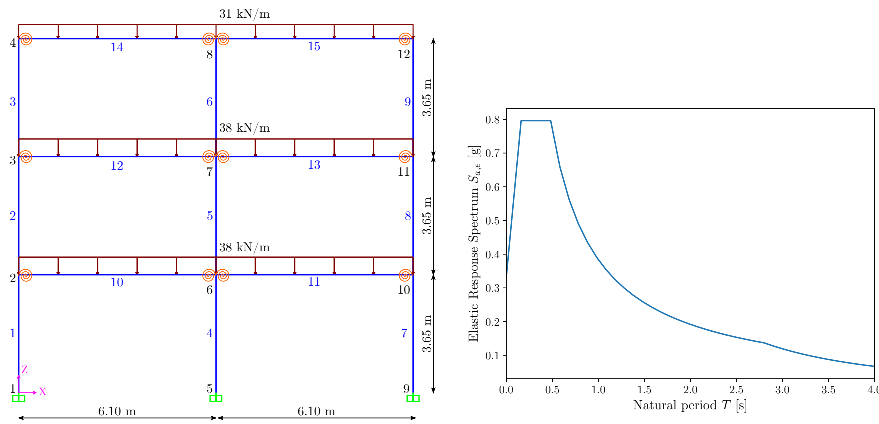
shape optimization studies the optimal shape of the structural members without varying their relative positions and structural layout [18,20]. The latter aspect is indeed investigated by the topology optimization branch [18,20].
With a growing consciousness and concerns for the environmental impact of the architecture and construction industry (AEC), in recent years life-cycle cost assessment posed the structural optimization area in a new perspective to globally account for optimizing cost-effective solutions considering the entire life-cycle of a structure [12,11]. However, this approach requires tremendous efforts to carefully evaluate many complex related aspects concurring with the definition of representative cost objective functions (OF) [1]. Nonetheless, as a first approximation, and in the absence of precise requirements and prescription, the weight minimization may still be successfully used as an indirect indicator of the construction cost, directly related to the minimum material consumption [18,6,20,14]. Furthermore, the minimum self-weight optimization has other benefits besides the material consumption criterion, e.g. it may aid to reduce the inertial effects due to a reduction of the mobilized mass during earthquake conditions [17].

Mainly focusing on steel structures, one of the main aspects to account for is fulfilling capacity design criteria for seismic design and detailing, in which, e.g., the principle of "strong-column weak-beam" must be respected [5,21]. Furthermore, the technical choice of a specific beam-column joint typology may strongly affect the steel frame's behavior under horizontal actions. Numerous beam-column connection typologies exist and they affect the beam's ultimate rotational capacity behavior evaluated on the moment-rotation plane, thus permitting the formation of plastic hinges and safer ductile global mechanisms [8,10,7,3]. The stiffest connections are represented by the welded ones, but they entail increased complexities from a constructive point of view [23], therefore the less stiff bolted ones are often preferred. The specific technical solution adopted for the connection directly affects the rotational stiffness denoted as K_θ, thus determining a semi-rigid beam-column joint. In the present study, the authors investigated in a global and simplified manner how the various limit modeling approaches of the beam-column joint [9] affect the results of a size optimization problem of a steel plane frame. Since the current focus is oriented toward global frame optimization results, linear dynamic analyses were herein conducted. Future studies should further explore the rotational stiffness effects by adopting more detailed nonlinear analyses to account for the entire moment-rotation behavior of the specific technically adopted connections.

2 Steel frame case study description

The two-bay three-story plane steel frame analyzed in the current document is inspired by the steel frame example presented in [19], and it is illustrated in Fig.1 (a). The black numbers represent the nodes' numbering system whereas the blue ones are referred to the structural finite element (FE) beams and columns members. The positions of semi-rigid joints are indicated with orange spirals, simulating the partial bending moment release due to the actual connection typology. Using a steel S275 material grade, the FE model has been implemented



(a) **(b)**

Fig. 1. View of the steel frame case study (a) and the adopted elastic response spectrum (b).

in the Computer and Structures inc. SAP2000 commercial software [2]. Specifically, the dead loads are automatically computed by the software after choosing the specific cross-section profile for the beams and for the columns. These latter are selected directly from the available cross sections in SAP2000 according to the Europe area commercially available standard steel profiles. Specifically, 70 different profiles for the HE steel profiles are available for the columns (HE100A, HE100B, etc., until HE1000M), and 50 different IPE cross sections are available for the beams (IPE100, IPE120, etc., until IPE600V). In order to account for seismic conditions for the optimal design of the size of each structural element, the authors hypothetically located the steel frame in the Abruzzi region in Italy, precisely in L'Aquila city. The seismic analyses have been conducted with the dynamic linear modal analysis with elastic response spectrum for the life-safety limit state (SLV), which is depicted in Fig.1 (b). The depicted response spectrum has been determined based on the geographical location of the structure, accounting for a ground typology B (typical of L'Aquila city), considering a commercial building class, with a nominal life of 50 years, an usage class III, thus a usage coefficient of $c_U = 1.5$, a topographical typology T1 for flat land, and considering a standard damping ratio of $\xi = 5\%$ [21]. The complete quadratic combination (CQC) has been performed to define the action effects envelope coming from the modal superposition of the first natural modes mobilizing a modal mass ratio greater than 85%. The mass considered in the modal analysis was accounted for quasi-permanent vertical load conditions, i.e. considering the dead load and the live load multiplied for a combinatorial factor $\psi_{2,1} = 0.6$ [21].

3 Discrete size optimization strategy with genetic algorithm

The authors implemented a Python script to deal with the *SAP2000 OAPI* module [2] in order to parametrize and automatically run the FE model. Specifically, the size optimization of the steel frame can be stated as a constrained problem in which the OF $f(\boldsymbol{x})$ is represented by the dead load total base vertical reactions. The constraints are thus represented by the unitless ratios associated with the structural checks according to the NTC18 Italian structural design code regulations, which incorporate the EN1993 Eurocode 3 for steel structures. The assessment of the constraints has been performed with the *SAP2000 Steel Design* module [2]. The design vector $\boldsymbol{x}$ is a discrete-integer-value array whose components referred to the index of the steel profiles associated with the various structural members, thus spanning between 0 and 70 for selecting a specific HE profile, and varying between 0 and 50 for identifying a specific IPE profile.

$$\begin{cases} \min & f(\boldsymbol{x}) \\ s.t. & g_1(\boldsymbol{x}) = \sum_{i=1}^{15} \max\{r_{NMM,i}, 0\} \leq 1 \\ & g_2(\boldsymbol{x}) = \sum_{i=1}^{15} \max\{r_{V,i}, 0\} \leq 1 \\ & g_3(\boldsymbol{x}) = \delta_{max} \leq L/250 \\ & g_4(\boldsymbol{x}) = \delta_2 \leq L/300 \end{cases} \tag{1}$$

The constraint g_1 in Eq.(1) accounts for the combined bending and axial force cross-section checks ratios ($r_{NMM,i}$) according to Eq.(C4.2.37) of Method B of the NTC18 commentary [22]. This ratio also accounts for compression buckling and lateral torsional buckling checks. g_2 consider the shear design checks ratios ($r_{V,i}$) according to NTC18 Eq.(4.2.16) [21]. On the other hand, g_3 and g_4 account for deformability checks respectively related to the maximum deflection δ_{max} and to the deflection under live loads only δ_2. SAP2000 steel design module automatically accounts for special seismic provisions such as the "strong-column weak-beam" principle by defining an over-strength check of the beam-column joint, see NTC18 Eq.(7.5.11) [21].

To address the current size optimization problem, the authors adopted the meta-heuristic well-established population-based *genetic algorithm* (GA) method [13]. Specifically, the available Python library *pymoo* [4] has been successfully employed within the current framework. Specifically, a population of 50 individuals (various samples of design vectors) has been considered, and the GA termination criterion was set to 50 generations, thus determining 2500 OF evaluation calls. The implemented Python script has been made freely available at the following GitHub repository: `https://github.com/marco-rosso-m/SAP2000-python-for-structural-optimization`.

4 Results and discussion

In order to explore the effects of various beam-column joint modeling strategies on the size optimization optimal solutions, the authors analyzed three structural configurations. The first configuration is related to a limit condition with perfect hinges between beams and columns, the second one is another limit condition with perfect fixity constraints between beams and columns. The third condition accounts for a more proper modeling condition referring to a realistic value of rotational stiffness K_θ. Based on the quite extensive experimental tests provided in the existing literature [8,10,7,3], the authors have identified that a reasonable range of K_θ values broadly spans between 10×10^3 (bolted connections, e.g. web cleats to column flanges) and slightly lower than 100×10^3 kNm/rad (welded connections, e.g. flush end plate to column web). Based on this quite wide range, the authors adopted a discrete range strategy to contain the search space complexity, i.e. by assuming only integer multiples of 10^3, e.g. 0×10^3, 10×10^3 20×10^3, etc.

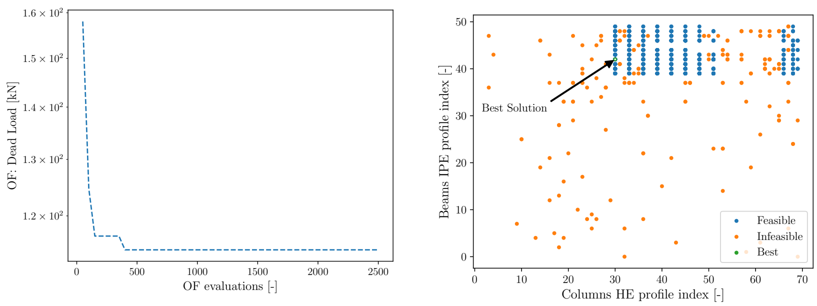
The above-mentioned three modeling approaches have been analyzed into two main cases. In the first main case, a simplified optimization strategy has been performed, thus considering the same beams' and the same columns' profiles for every floor level. Therefore, in general, the design vector appears as a three-component array $\boldsymbol{x} = [x(1), x(2), x(3)]^T$. The first two components indicate respectively the steel profiles for the columns and for the beams for the whole elevation, whereas the third one considers a discrete possible value for K_θ for every beam-column joint. In the second main case, instead, a more complete optimization strategy has been implemented with an extended search space, i.e. based on adopting different profiles for both columns and beams at every floor level. In that case, in general, the design vector appears in general as a nine-components array. The first three components account for different columns' HE profiles for every level, the next three ones designated the different beams' IPE profiles for every level, whereas the latter three components identify the K_θ values for every single floor. Furthermore, two additional constraints have been introduced in order to account for geometric reasons. Indeed, when the column section varies with the elevation, it is typical that it tapers from the bottom to the top of the building. Therefore:

$$g_5(\boldsymbol{x}) = x(1) - x[0] \leq 0 \qquad \text{and} \qquad g_6(\boldsymbol{x}) = x(2) - x(1) \leq 0 \tag{2}$$

The summary of optimal results for all the analyzed cases is reported in Tab.1.

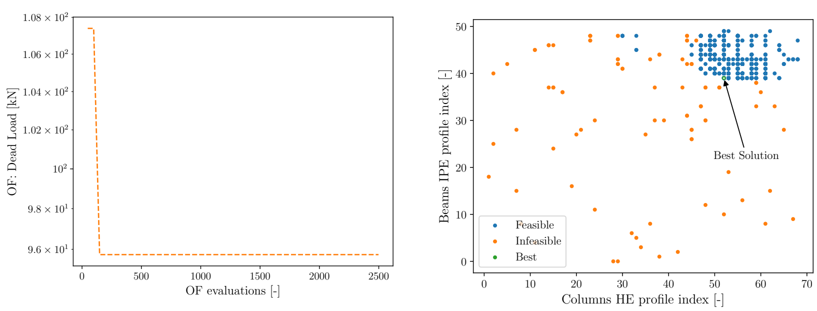
4.1 Same beams' and columns' profiles for every floor case

In this section, the results of the first main case are presented, i.e. the ones with the same beams' and columns' profiles for every floor. In particular, the above-mentioned three modeling strategies sub-cases have been explored, as depicted in Figs.2-4 respectively. In all three sub-cases, the best optimal solution was found within the first 500 OF evaluations. Since the search space is composed of two design variables only, except when considering the



(a) **(b)**

Fig. 2. Perfectly-hinged case with the same columns' and beams' profiles at every floor: OF history (a) and analysis of the GA's explored search space (b).



(a) **(b)**

Fig. 3. Perfectly-fixed case with the same columns' and beams' profiles at every floor: OF history (a) and analysis of the GA's explored search space (b).

optimization of the rotational stiffness, it is worthy to visualize how the GA explored the search space of possible combinations of steel profiles, evidencing the existence of a feasible region (blue) and an unfeasible one (blue). The feasible region appears noisy in Fig.2 (b) and Fig.4 (b), probably because of the imposed respectfulness of the special seismic provisions. Between the two limit case, the fixed one better exploited the load-carrying capacities redistribution, thus ensuring the minimum OF value. A similar finding was obtained in the K_θ sub-case, achieving an optimal remarkably large value of 90×10^3 kNm/rad.

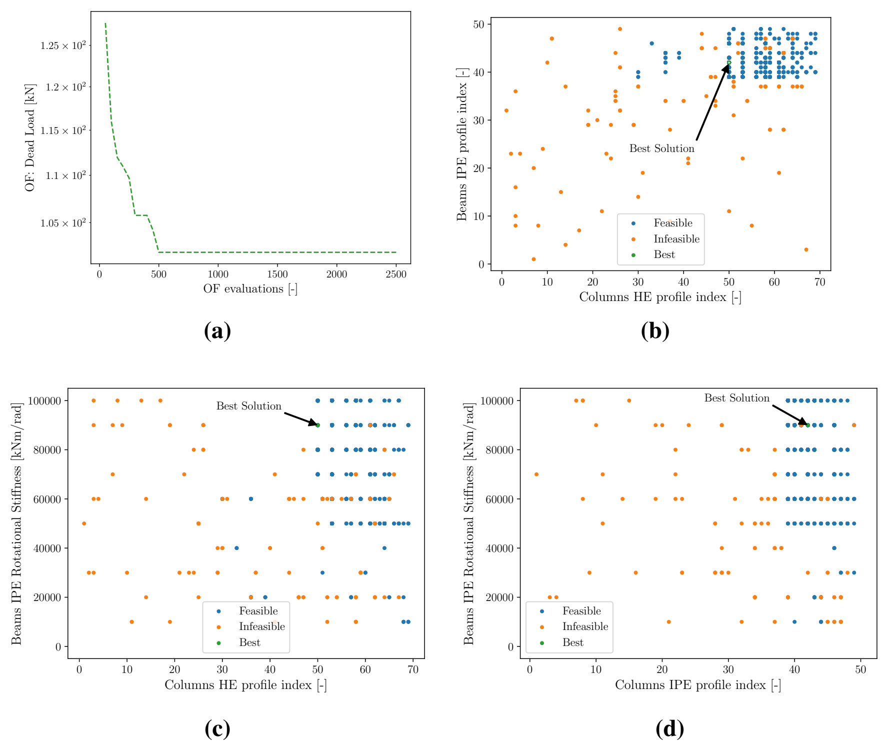


Fig. 4. Partially-released case with the same columns' and beams' profiles at every floor: OF history (a) and analysis of the GA's explored search space in terms of profiles (b), columns' profiles vs rotational stiffness (c), and beams' profiles vs rotational stiffness(d)

4.2 Different beams' and columns' profiles for every floor case

The results of the second main case are herein discussed, i.e. the one adopting different beams' profiles and different columns' profiles for every floor level. Specifically, the previously-mentioned three modeling strategies sub-cases have been analyzed, and their individual OF history during the optimization process have been gathered in Fig.5. Specifically, sub-figures (a) and (b) revealed that, during the initial 200 function evaluations, the entire population was unfeasible, and only from that point forward the GA founded the right feasible region and effectively started the actual optimization process. On the other hand, the semi-rigid connection K_{θ} sub-case revealed that, probably the optimization procedure should have been prolonged much more than 50 iterations, since the still decreasing trend of the OF. Finally,

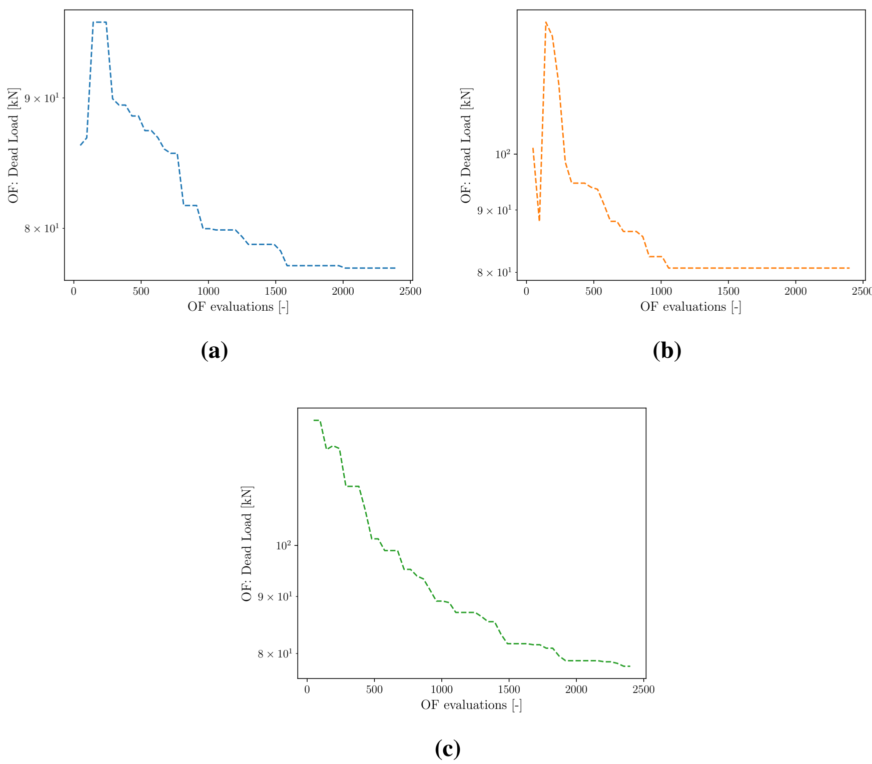


Fig. 5. Different columns' and beams' profiles at every floor case: OF history for perfectly-hinged beams (a), for perfectly-fixed beams (b), and for partially-released beams (c).

comparing the final results in Tab.1, this latter second main case with a more refined and comprehensive size optimization problem delivered in virtually all the cases a considerable reduction of the OF. In this occasion, the three optimal solutions are quite comparable (around 80 kN), and it is worth noting that the semi-rigid case preferred an optimal solution with almost fixed connections at the first level (80×10^3 kNm/rad) and almost fully hinged to the other two floor levels (0 and 10×10^3 kNm/rad respectively).

5 Conclusions

The current study permitted analyzing in a simplified and linear manner the effects of three beam-column joint modeling strategies and how it affects the size optimization of a steel

Table 1. Summary and comparison of the optimal results.

Elements	Same profiles for every floor				Different profiles for every floor			
	Hinged case profiles	Fixed case profiles	Partially released profiles	Rot.Stiff. K_θ [kNm/rad]	Hinged case profiles	Fixed case profiles	Partially released profiles	Rot.Stiff. K_θ [kNm/rad]
Columns:								
1-4-7					HE600A	HE600A	HE500A	
2-5-8	HE300M	HE600A	HE550B		HE400A	HE400A	HE450A	
3-6-9					HE280A	HE320A	HE280A	
Beams:								
10-11					IPE550	IPE550	IPE550	80×10^3
12-13	IPE550	IPE500O	IPE550	90×10^3	IPE550	IPE550	IPE500O	0
14-15					IPE450R	IPE550	IPE550	10×10^3
OF [kN]	114.366	95.741	101.976		77.181	80.652	77.925	

frame under seismic loading. As expected, the results evidenced that optimal semi-rigid joints at lower-level floors should tend to be stiffer likewise welded joints, whereas at higher levels they may tend toward hinged connections. Future studies should further explore the rotational stiffness effects by adopting more detailed nonlinear analyses to account for the entire moment-rotation behavior of the specific technically adopted connections.

Supplementary material

The implemented code is made freely available at the following GitHub repository: https://github.com/marco-rosso-m/SAP2000-python-for-structural-optimization.

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