Earthquake-resistant buildings with steel or composite columns: Comparative assessment using structural optimization

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ABSTRACT

This work investigates and compares the cost-effectiveness of seismically designed buildings having either pure steel or steel-concrete composite columns. In order to ensure an objective comparison of these two design approaches, the assessed building designs are obtained by a structural optimization procedure. Thus, any bias that would result from a particular designer's capabilities, experience, and subjectivity is avoided. Hence, a discrete Evolution Strategies optimization algorithm is employed to minimize the total cost of materials (steel and concrete) used in a structure subject to constraints associated with: (a) Eurocode 4 provisions for safety of composite column-members, (b) Eurocode 3 provisions for safety of structural steel members, and (c) seismic system behaviour and resistance. Extensive assessments and comparisons are performed for a variety of seismic intensities, for a number of building heights and plan configurations, etc. Results obtained by conducting 154 structural design optimization runs provide insight into potential advantages attained by partially substituting steel (as a main structural material) with concrete when designing the columns of earthquake-resistant buildings.

KEYWORDS:
steel structure; composite structure; structural optimization; seismic design; Eurocode 3; Eurocode 4; pushover analysis; fundamental period

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1. Introduction

The incorporation of steel-concrete composite elements in a structure is nowadays regarded as established design and construction practice. Nevertheless, the investigations conducted on the conditions under which such practice is more cost-effective than other alternatives are rather limited. The use of composite elements is typically seen as an alternative to the use of pure steel elements. The use of each of these two types of elements is associated with certain advantages and disadvantages. Therefore, it is essential to comparatively assess structures incorporating either type of elements.

The purpose of this work is to assess multi-storey composite buildings with steel-concrete composite columns with respect to their cost effectiveness and seismic resistance capability. The assessments performed include comparisons with pure steel buildings. To ensure that all assessments and comparisons are made in an objective manner, the structures considered are designed in a way that optimal usage of the available materials and cross-sectional geometries is achieved. Thus, the designs attained do not depend on a designer's capabilities, experience and subjectivity, but are the outcome of an objective automatic design optimization procedure.

Structural optimization is a powerful computational tool which assists engineers in efficiently searching for cost-effective designs within extensive solution spaces. The existing literature includes several design optimization applications for pure steel structures (e.g. [1]-[12]). Design optimization applications for structures with steel-concrete composite columns have appeared primarily in recent years ([13]-[20]). The comparisons between pure steel and composite buildings presented in the above publications cover a narrow spectrum of design cases. Thus, although some information and optimization results are provided in the relevant available literature, additional assessments are needed for a more comprehensive comparison between the alternatives of pure steel and composite columns in optimally designed multi-storey buildings.

In the present paper, structural optimization is applied for the seismic design of composite buildings, in which the steel-concrete columns consist of steel members with standard I-shaped sections (HEB) fully encased in concrete. Moreover, buildings with pure steel columns are optimally designed using standard HEB sections. Steel beams with standard I-shaped sections (IPE) and (optional) steel bracings with standard L-shaped sections are considered for all design cases (using either composite or pure steel columns). All buildings assessed are required to satisfy the provisions of Eurocode 4 for the steel-concrete composite members and Eurocode

3 for the pure steel members. Seismic actions are taken into account through lateral deflection constraints evaluated using nonlinear static pushover analyses. Moreover, the fundamental periods of the optimally designed buildings are determined and assessed. All structural analyses required during any optimization run are performed with the software OpenSEES [21], which is automatically invoked by a discrete Evolution Strategies optimization algorithm. The seismic design optimization framework utilized in the present work is described in detail in [20].

Extensive assessments and comparisons are made herein for composite and pure steel buildings. Optimal structural designs are identified for a variety of seismic intensities, for a number of building heights and plan configurations, etc. The optimization results allow for an objective comparison of various designs in terms of required materials cost and achieved capacity to withstand earthquake actions and provide insight into the relative cost-effectiveness of the composite and pure steel design approaches.

2. Structural design optimization

Standardized steel sections are used for all structural elements (composite or pure steel) in this work. Hence the search space consists only of discrete design options, which renders the investigation performed a discrete optimization problem. The procedure developed in [20] is adjusted and applied herein. In particular, an Evolution Strategies algorithm is employed, which is a population-based evolutionary optimization method. At each ES-generation, this algorithm uses recombination and mutation operations to manipulate a population of μ parent design vectors and produce a population of λ offspring design vectors. Then, a new parent population for the next ES-generation is formed by selecting μ vectors from the set of λ offspring vectors. This iterative procedure is terminated when no actual improvement is observed in the objective function value for a number of ES-generations. The main features of the adjusted ES implementation utilized in the present paper are described in the remainder of this section.

2.1. Objective function, design variables and constraints

The objective function employed in this work is an implicit measure of the total materials cost of the main structural elements (columns, beams and bracings) in the building frame considered. Specifically, the objective to be minimized by the optimization procedure is the total equivalent steel mass M_s^{tot} (tonnes of steel) of all structural material quantities used:

$$M_s^{tot} = M_S + CR \cdot V_C . \quad (1)$$

In the above equation, M_S and V_C are the total steel mass (t) and concrete volume (m^3), respectively, used in the structure. The Cost Ratio *CR* enables the conversion of the total concrete volume in the structure to equivalent steel mass and is defined as the total unit cost for concrete (€/m^3) over the total unit cost for steel (€/t). For Cyprus, *CR* is estimated as 0.012 t/m^3 [20], which corresponds to relatively cheap concrete (locally produced) and expensive steel (imported). To derive this *CR*-value, both material and labour costs were taken into account, as well as features and details required to install the respective structural members [20]. As *CR* is a ratio, its value is practically insensitive to uniform market price variations. The material mass needed for structural elements/parts that are not explicitly included in the analysed frame model and are typically designed separately (slabs, secondary beams, foundation, etc.) is not taken into account in the objective function (1).

The design variables of the optimization procedure are associated with the steel section geometries of the main frame elements and take values from the following 3 properly sorted discrete databases: (a) HE 100 B to HE 1000 B for columns, (b) IPE 80 to IPE 600 for beams and (c) L 90×90×7 to L 250×250×28 for bracings. For the steel-concrete columns, the same basic configuration is always used for the concrete and its reinforcement encasing the HEB-sections, therefore no additional design variables are required beyond the ones controlling the steel cores; section dimensions and details are provided in [20]. It is also mentioned that the database with L-shaped sections includes a ‘zero’ option (no bracing section), which actually offers the optimizer the choice to deactivate bracings in a structure. Thus, in general, the optimal solution identified may be a braced or unbraced frame. Note that the steel HEB-sections have a common orientation across all columns of a pure steel or composite frame: their major axes are parallel to the global horizontal *y*-axis of the building. Therefore, the bracings’ sections are determined based mainly on the building’s stiffness needs in the *y*-direction. A final issue linked with the handling of design variables is the potential incompatibility of member sections at beam-column connections. Hence, when the column web height is too small to accommodate the flanges of the beams at a connection along *y*-direction, we are forced to override the optimizer’s choice and adopt an increased column section.

As regards the implemented constraints, these are associated with the design requirements imposed by relevant standards and guidelines. Hence, with the aid of linear static analysis results, composite column members are designed according to provisions of Eurocode 4 [22], while pure steel column and beam members are designed according to provisions of Eurocode 3 [23]. Moreover, as regards seismic system resistance, the output of nonlinear pushover analyses is used to assess whether a structure: (a) can safely reach a pre-specified target displacement at the roof level (see section 3 for details) and (b) has interstorey drifts within acceptable limits (≤4% of the storey height for composite frames, ≤5% of the storey height for pure steel frames [24]). Steel bracings are not checked with respect to provisions of Eurocode 3; their sections are identified based on the lateral deflection constraints for adequate seismic performance of the structural system.

A structural design that violates any of the aforementioned constraints on structural member capacities and system resistance under seismic action is deemed infeasible. Such a design is penalized by adding to the objective function (1) a penalty term, which is equal to the total mass of the heaviest design possible for the database options available. Thus, infeasible designs are not immediately eliminated by the optimizer, but are exploited during the new design generation process to potentially contribute any favourable design feature they possess.

2.2. Structural modelling and analyses

The composite and pure steel frame structures assessed in the present work are numerically modelled and analysed using OpenSEES [21]. To properly simulate the stress-strain behaviour of structural materials, the following models in OpenSEES are employed:

- ‘Steel01’. This material type is utilized to model all structural steel members, *i.e.* the steel cores of composite columns, the pure steel columns, the beams and the bracings. It is implemented with a yield stress of 235MPa and an elasticity modulus of 210GPa. As regards hardening, the post-yielding stiffness is 5‰ of the initial one.
- ‘Concrete01’. This material type is employed for all concrete regions of the composite columns. It is implemented with cracking and crushing strains of 2‰ and 3.5‰, respectively. For the confined concrete area surrounded by the reinforcement, the compressive strength is set to 20MPa (no tensile strength is assumed). For the unconfined concrete area (external cover of 2.5cm), a reduced compressive strength is assumed, which is 20% lower than that of confined concrete.
- ‘ReinforcingSteel’. This material type is used for the longitudinal and transversal reinforcement bars of the composite columns. A yield stress of 434MPa, an ultimate stress of 521MPa, a yield strain of 2.5‰ and an ultimate strain of 20% are defined.

All structural members are represented in any building frame using fiber section elements. Columns and beams are modelled using ‘nonlinearBeamColumn’ elements of OpenSEES.

Bracings are modelled as ‘truss’ elements [21]. Each member section is divided into a sufficient number of fibers to adequately capture the development of plastic regions and hinges.

Slabs and secondary beams are not explicitly included in the structural model; their contribution is simulated by including their weight in the structure’s dead loads and considering a rigid diaphragm at each slab level. The connections at the base of each column are modelled as fixed supports. Beam-column connections in x-direction are assumed to be moment-restrained, while beams in y-direction are simply supported.

For each of the assessed buildings, five analyses are performed: (a) a force-controlled linear static analysis under gravitational loads, (b) two eigenvalue analyses (one for each horizontal direction) and (c) two displacement-controlled non-linear static pushover analyses (one for each horizontal direction). In the linear static analysis, the combined vertical loads are gradually applied on the beams of the building. If the analysis finishes successfully, the structural members are assessed with respect to the individual capacity criteria defined in Eurocode 3 [23] for steel members and Eurocode 4 [22] for composite steel-concrete members. Although designs that fail in any of the aforementioned criteria are considered infeasible, their performance under horizontal loads is still evaluated. A penalty is added to the objective function of such designs and are not discarded from the population of designs processed by the optimizer. Designs with failures in multiple criteria receive a higher penalization.

3. Adjustment of seismic demands

The seismic structural performance is determined by displacement-controlled nonlinear pushover analyses up to a targeted top displacement Δ_{target} according to the provisions of ASCE/SEI 41-06 [24] and FEMA 440 [25]. The magnitude of the required displacement depends on various problem-related variables: type of soil, seismic hazard of the area, expected load distribution, etc. This requirement is increased for structures of high economical value or importance to the public safety. The same applies when more demanding design codes are used. Thus, to generalize the results of an investigation, a variety of targeted displacements needs to be examined for each problem case considered.

The approach followed in this work to control the seismic capacity of the building designed is by directly adjusting the targeted top displacement used to perform the pushover analysis. This is achieved by introducing the displacement modification factor δ in the calculation of the targeted top displacement:

$$\widetilde{\Delta}_{target} = \delta \cdot C_0 \cdot C_1 \cdot C_2 \cdot C_3 \cdot Sa \cdot \frac{T_e^{\,2}}{4\pi^2} \cdot g\,, \tag{2}$$

where C_0, C_1, C_2 and C_3 are coefficients defined in [25] and T_e is the effective fundamental period defined in the same document. The expression given in [25] for the targeted top displacement is Eq. (2) with δ=1.

As $\widetilde{\Delta}_{target}$ is directly related to the design earthquake through the spectral response acceleration (Sa), a more severe seismic excitation, *i.e.* one with increased Sa, would cause larger displacements to the assessed building. Hence, δ actually corresponds to the ratio of the spectral response acceleration considered for the design over the spectral response acceleration $Sa_{10/50}$ corresponding to the Earthquake Hazard Level with probability of exceedance 10% at 50 years, which is typically used for ordinary residential buildings.

FEMA-356 [26] provides an equation to adjust the mapped response acceleration parameters to other probabilities of exceedance:

$$S_i = S_{i,10/50} \cdot \left(\frac{P_R}{475}\right)^n \tag{3}$$

where n is a site-dependent coefficient, which takes into consideration the soil type and seismic hazard of the area [26] and P_{EY} is the probability of exceedance in time Y (years) for the desired earthquake hazard level. Dividing this equation by $S_{i,10/50}$, a function to calculate the displacement modification coefficient δ for a given return period and vice versa is obtained:

$$\delta = \left(\frac{P_R}{475}\right)^n \Rightarrow P_R = 475 \cdot \sqrt[n]{\delta} \tag{4}$$

This equation includes coefficient n, so for any location there is an earthquake with some return period for which the structure is required to reach δ times the targeted top displacement that corresponds to an earthquake with 10% probability of exceedance in 50 years. Fig. 1 shows the return period for the design earthquake corresponding to different values of coefficient δ.

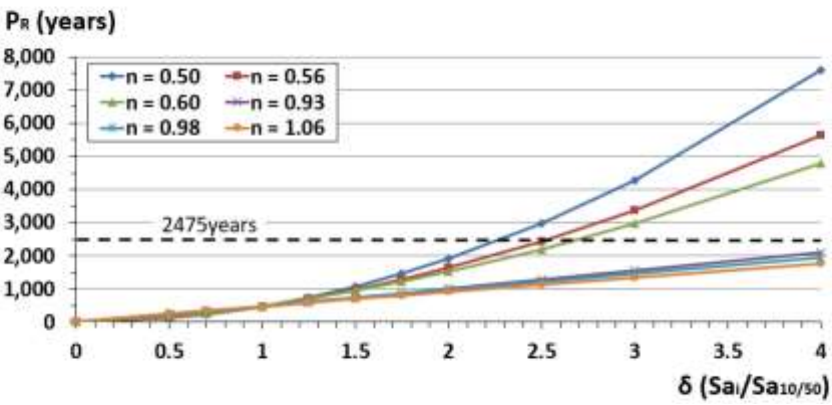

Fig. 1. Return period of the design earthquake corresponding to various values of factor δ (the dashed line defines the 2475-year limit for the 2% probability of exceedance in 50 years).

While high values of δ, such as 3.0 or 4.0, might seem excessive, only in a few cases do such values correspond to seismic excitations with a probability less than the minimum values already used in FEMA-356, *i.e.* 2% in 50 years (see shaded cells in Table 1). It is of particular interest to note that, for values up to δ = 2.0, the considered design earthquakes have probabilities of exceedance >2% for any *n*-value (Table 1).

Table 1. Probability of exceedance (P_{EY}) in 50 years.

Site-dependent coefficient *n*	**Displacement modification factor (δ)**									
	0.50	**0.70**	**1.00**	**1.25**	**1.50**	**1.75**	**2.00**	**2.50**	**3.00**	**4.00**
0.50	34.37%	19.34%	10.00%	6.52%	4.58%	3.38%	2.60%	1.68%	1.17%	0.66%
0.56	30.44%	18.05%	10.00%	6.83%	4.98%	3.81%	3.01%	2.03%	1.47%	0.89%
0.60	28.41%	17.37%	10.00%	7.00%	5.22%	4.06%	3.27%	2.26%	1.68%	1.04%
0.93	19.90%	14.32%	10.00%	7.95%	6.59%	5.61%	4.88%	3.86%	3.18%	2.35%
0.98	19.23%	14.06%	10.00%	8.05%	6.73%	5.78%	5.06%	4.05%	3.38%	2.53%
1.06	18.33%	13.71%	10.00%	8.18%	6.93%	6.02%	5.33%	4.34%	3.67%	2.81%

The disadvantage of using the probability of exceedance or the return period of an earthquake is that these two properties depend on the characteristics of the site. Using δ, one can assess structures constructed in areas with different site characteristics on the basis of their performance at their own locations. According to FEMA-440, structures designed for δ=1 possess the required ductility to sustain an earthquake, the magnitude of which has probability 10% of being exceeded in a time period of 50 years. In the analyses performed in this work, the site characteristics of a single location only have been used and, therefore, the used values of δ correspond to various intensities of an excitation from the same source in all assessed problem cases. Therefore, values δ>1 imply higher ductility demands or higher structural significance. By introducing factor δ, the comparison of structural designs with different characteristics is possible, as it is performed on the basis of an adjusted targeted top displacement, which serves as an indicator of their capacity.

4. Fundamental period formulas

ASCE/SEI 41-06 [24] and Eurocode 8 [27] provide approximate formulas to calculate the fundamental period of steel moment resisting frames using only their total height. In particular, ASCE/SEI 41-06 suggests its calculation through:

$$T = 0.035 \cdot H^{0.8}, \tag{5}$$

where H is the building height in feet, while Eurocode 8 defines it as:

$$T = 0.085 \cdot H^{3/4}. \tag{6}$$

The alternative formula for buildings up to 12 storeys and storey height at least 10ft is:

$$T = 0.1 \cdot n, \quad (7)$$

where n is the number of storeys of the building.

Goel and Chopra [28] performed an investigation of approximate formulas for various types of buildings and found that the code formulas significantly underestimate the fundamental periods of steel MRFs. Hence, they proposed alternative formulas to calculate empirically the fundamental period for this building type corresponding to a lower limit:

$$T = 0.028 \cdot H^{0.8}, \quad (8)$$

where H is the building height in feet, and to an upper limit:

$$T = 0.045 \cdot H^{0.8}. \quad (9)$$

The above approximate formulas are frequently applied to estimate the fundamental period of steel buildings in the design phase. Similar recommendations for steel-concrete composite buildings are not proposed; however, high-period designs need to be avoided. To investigate the effect approximate fundamental period calculations may have on building designs, all aforementioned formulas were used to set fundamental period equality constraints. Hence, the intended fundamental period for each assessed building height was calculated separately using Eqs (5)-(9). Then, the maximum attainable δ-value was determined for the building having fundamental period equal to each of the approximations from Eqs (5)-(9) and the corresponding cost-outcome was illustrated on the graphs presenting optimization results in the next section.

5. Optimization Results

5.1. Reference Building (6-storeys, 5×5 bays)

The building selected as a reference for the numerical investigation of the present work is a 6-storey space frame of square floor plan with 5 bays in each horizontal direction (Fig. 2(a)). Bracings are (optionally) installed at the middle bay of each of the 4 external sides of the building. The bay width along both x- and y-directions is 5.5m, while the height of each storey is 3.5m, resulting in a total building height of 21m. This is assumed to be a residential building, which implies a characteristic live load value of q=2kN/m^2 at each floor. The building is optimized separately with pure steel columns and composite steel-concrete columns, in order to assess the cost-effectiveness of each of these configurations.

Fig. 2 illustrates the 6 member groups defined for the reference building. Columns are organized into 4 groups according to their position in the floor plan: (1) corner, (2) peripheral in x-direction, (3) peripheral in y-direction and (4) internal. A single group (5) contains all beams, while the last group (6) contains all bracings of the structure. One discrete design variable is assigned to each member group taking values from the respective database (HEB for columns, IPE for beams and L for bracings). For medium-rise and high-rise buildings, increasing the number of column-groups to allow for different sections along the building height could lead to more cost-effective designs (see e.g. [20]). However, such a finer design variable configuration is not applied herein, because this would substantially enlarge the available design space and increase the computational time required for its effective search.

To facilitate the discussion in the sequel of the present work, structural designs are codified using the format: *'Column type - Number of storeys - Number of bays in each direction - Value of δ (Optional additional characteristics)'*. The options for *Column type* are 'C' for composite or 'S' for pure steel columns. For example, the reference building with composite columns designed for δ=2 is designated as C-6-5-2. If this is assumed to be an office building with pure steel columns, then it is designated as S-6-5-2(q=5kN/m^2).

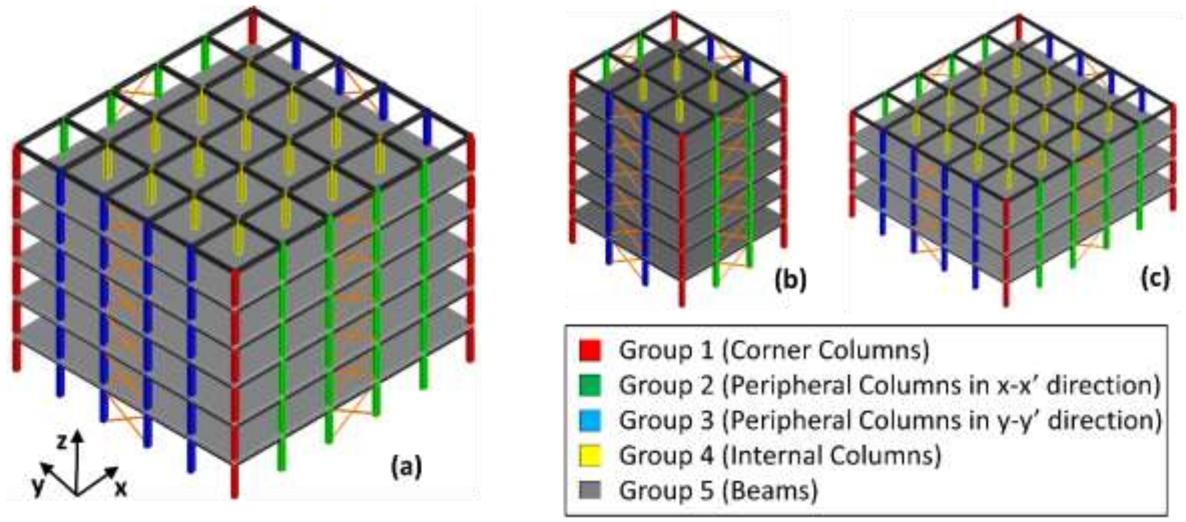

Fig. 2. Section grouping applied to: (a) the reference building C-6-5-δ, (b) C-6-3-δ and (c) C-4-5-δ (top slabs removed for visualization purposes).

A total of 22 optimization runs were carried out for the reference building: 10 runs were performed for each of column types 'C' and 'S' with δ-values ranging from 0.5 to 4, while one more run for each column type was conducted for gravitational loads only (δ=0). Fig. 3 presents the total equivalent steel mass and the fundamental period of the optimized designs identified as a function of the δ-value specified. Clearly, higher δ-values (*i.e.* increased seismic resistance demands) induce the need for larger amounts of structural material in the building, therefore monotonic increase in both curves C-6-5-δ and S-6-5-δ (designs of reference building for various δ-values) is observed.

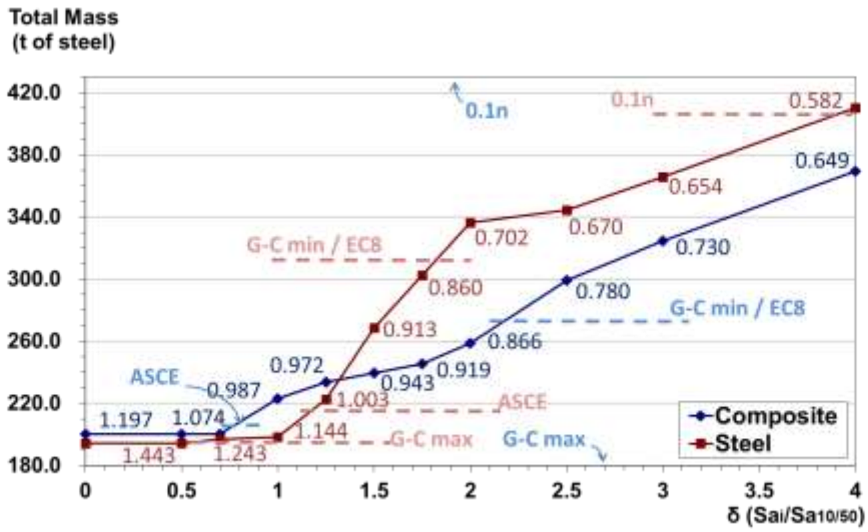

Fig. 3. Total equivalent steel mass versus δ for C/S-6-5-δ (reference building)
(the values illustrated next to each point are the corresponding maximum fundamental periods)

For each of these two curves, an upper and a lower total mass bound can be determined. The upper bound is defined as the total mass of a building designed using the largest available section for each structural member. As the same HEB-database is utilized for both composite and pure steel columns, the difference in the respective upper total mass bounds results from the presence of concrete in the case of composite columns. On the other hand, the lower total mass bound corresponds to the case of δ=0 (only gravitational loads applied to the structure). In that case, the beams' section is actually dictated by the required bending moment capacity, which is directly related to the beam length (bay width) and the applied gravitational loads. The resulting beam section specifies then the minimum acceptable column section, because, in order to be able to connect the two members, the space available between the column flanges (column web height) must exceed the width of the beam flange.

It is worth mentioning that optimized designs for δ=0 have a considerable seismic resistance capacity, which actually suffices for significant values of δ, although δ=0 means that the structure is not explicitly designed to withstand seismic loads. In particular, one can notice in Fig. 3 that the same optimized design is used for C-6-5-0, C-6-5-0.5 and C-6-5-0.7. Respectively, for pure steel columns, this applies to S-6-5-0 and S-6-5-0.5, while the cost for δ-values up to 1 is not significantly increased and is still lower than the minimum cost of the corresponding steel-concrete composite designs.

What is also of particular interest is the intersection point of the cost-versus-δ curves, as it defines the level of horizontal displacement requirement, beyond which buildings with steel-concrete composite columns are more economical than with pure steel ones. As the calculated optimal points of the curves depend on the optimization problem's configuration, when its parameters are changed (e.g. by varying the number of bays or storeys), the intersection point may shift to any direction, increasing or decreasing the benefit gained from the one philosophy over the other. Therefore, 3 extra sets of optimization runs described in the following subsections were performed. In any case, it is evident from the results in this section that buildings with pure steel columns are more economical for lower δ-values, while composite columns should be preferred for higher δ-values, with the curves' intersection point signifying the change of preference between the two column types.

Any building examined in this section is simulated using a three-dimensional frame model with the fundamental period depending on the direction of vibration. Hence, the values given next to the points of the graph of Fig. 3 (as well as in corresponding graphs in the next subsections) are the maximum fundamental periods of each optimal design, which are calculated for displacement parallel to the column sections' major axis, *i.e.* considering their stiffness about the minor axis. The dashed lines illustrate the cost-level of the designs attained when fundamental period values are imposed using equality constraints based on the approximation formulas of section 4; arrows are used to indicate cases yielding costs outside the relevant axis range. All formulas of section 4 take into account only the total height or the number of storeys of the building; the fundamental period values obtained for all cases considered in this section are provided in Table 2.

Table 2. Fundamental periods (s) calculated based on the formulas of section 4.

Number of storeys	**Total Height**		**Fundamental Period Calculation Formula**				
	(m)	**(ft)**	**0.1n (Eq. 7)**	**Goel & Chopra minimum (Eq. 8)**	**EN 1998 (Eq. 6)**	**ASCE/SEI 41-06 (Eq. 5)**	**Goel & Chopra maximum (Eq. 9)**
2	7.00	22.97	0.2	0.34	0.37	0.43	0.55
4	14.00	45.93	0.4	0.60	0.62	0.75	0.96
6	21.00	68.90	0.6	0.83	0.83	1.03	1.33

It can be noticed in Fig. 3 that, for both pure steel and composite columns, increased seismic demands lead to designs with lower fundamental periods, reducing the targeted top displacement the buildings are required to reach. The variation in the calculated periods is significantly larger for the pure steel buildings. Optimized composite designs, even when designed for gravitational loads only, have an increased stiffness due to the columns' concrete, resulting in lower fundamental periods. For this design approach, the Goel & Chopra maximum period approximation is lower than the defined feasible designs and the '0.1n' approximation significantly higher. Since the designs illustrated are optimized ones, their fundamental periods are the largest possible values of a feasible design for each level of seismic demand. Therefore, the Goel & Chopra maximum limit would not alter the optimization results of the composite buildings; for the pure steel building it would only affect the designs below δ≈0.5.

5.2. The effect of increased seismic mass (q=5kN/m²)

The second set of optimization runs aims to investigate the effect of the total seismic mass on the designs obtained. For this purpose, a change in the intended use from residential building (q=2kN/m²) to office building (q=5kN/m²) is considered. Such a change results in increased applied loads on the building and, consequently, in larger total mass per storey. Therefore, higher stiffness per storey is required to achieve adequate structural performance under seismic loads. The actual aim of this change is to reduce the effect of the constraints related to the evaluation of the candidate designs' individual members with respect to the capacity criteria defined in Eurocodes 3 and 4. Instead, an increased effect of the structural system performance on the determination of the optimized design in each case is expected.

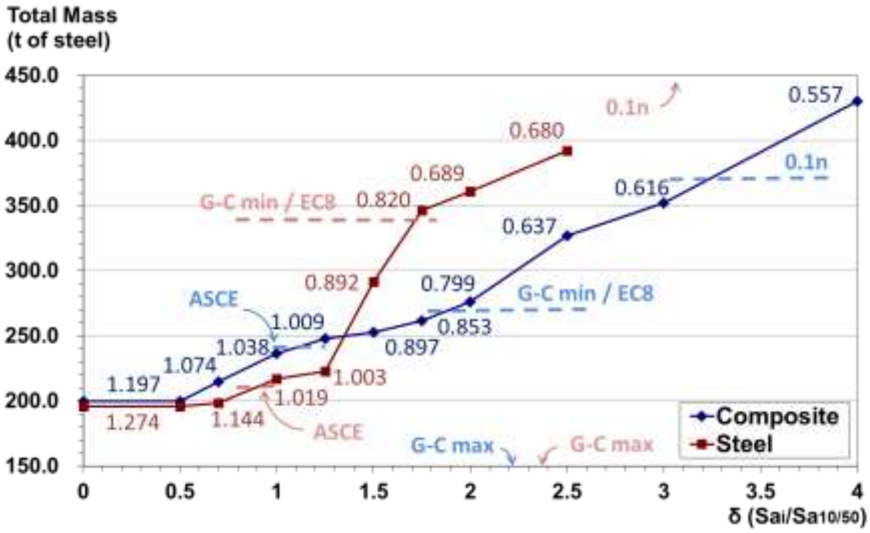

Fig. 4. Total equivalent steel mass versus δ for C/S-6-5-δ(q=5kN/m²)
(the values illustrated next to each point are the corresponding maximum fundamental periods)

The new 22 optimization runs performed to investigate the effect of increased storey masses yielded the curves C-6-5-δ(q=5kN/m²) and S-6-5-δ(q=5kN/m²) of Fig. 4. A comparison with Fig. 3 shows that the increase in total structural cost of the new optimized designs is similar for both pure steel and composite columns. The value of δ at the intersection point of the two curves in Fig. 4 is not altered significantly compared to the reference curves of Fig. 3, as the curves illustrated in Fig. 4 are roughly parallel to the respective ones in Fig. 3. Therefore, although an increase in the total seismic mass of the structure results in larger member sections, the relative cost-effectiveness of pure steel and composite design approaches does not seem to be affected. However, it is sometimes affected by the utilized discrete section databases, since the selection of a larger section for a higher δ-value results in certain cases in an abrupt increase of the total cost. Another effect of the discrete section availability is that, while for composite column sections the optimization algorithm managed to determine building designs with δ-value up to 4, for pure steel sections it was unable to find feasible designs for δ-values higher than about 2 (larger sections are needed, which are not available in the database). As regards the fundamental periods of the optimized designs, it can be noticed that the increase of the total seismic mass affects differently the pure steel and composite design approaches. For the steel-concrete composite approach, seismic demands corresponding to δ-values up to 1.25 lead to higher fundamental periods for increased seismic mass compared to the ones for the reference building; for higher δ-values, fundamental periods for increased seismic mass are significantly lower. For the pure steel buildings, all optimized designs for increased seismic mass are characterized by lower fundamental periods compared to the ones for the reference building. Finally, Goel & Chopra maximum approximations are too low and therefore not applicable.

5.3. The effect of plan configuration (3×3, 7×7 and 9×9 bays)

In engineering practice, when increased stiffness requirements are induced to the design of a steel building, they are usually met by either installing steel bracings in a number of predefined bays of the structure or by selecting stronger columns (when the structure is designed as a moment resisting frame). In reinforced concrete buildings, even though shear walls provide a significant resistance to horizontal loads, the columns' stiffness contribution is significant as well. One would generally expect a composite building with concrete-encased steel columns to exhibit a favourable performance, as it can combine concepts both from steel and reinforced concrete buildings.

The focus in this subsection is on investigating the overall stiffness of the optimized building and the way it is attained. In general, moment resisting frames are structures with high fundamental period and, therefore, the required ductility demands are high. However, the more bracings are installed in the building, the more is this requirement reduced and the building's behaviour shifts toward that of a 'profoundly' braced frame, in which the columns' sections are mainly determined by gravitational loads. Therefore, the way the overall stiffness of a structural system is to be attained is directly linked to a more stiff or flexible design to be determined due to the system's structural members configuration, *i.e.* its topology.

Following the above discussion, a third set of optimization runs is performed, in which 3 additional buildings with different numbers of bays per direction are simulated: a 3×3-bay

(C/S-6-3-δ) (Fig. 2(b)), a 7×7-bay (C/S-6-7-δ) and a 9×9-bay building (C/S-6-9-δ). All these buildings have the same basic geometric characteristics as the reference building: 6 storeys, 5.5m bay span and 3.5m storey height. Furthermore, bracings are installed at the middle bays of all external sides of each building. The main difference between the examined buildings, including the reference 5×5-bay building, is the contribution of the bracings to the total stiffness of each storey. As the number of bays per direction increases, extra columns are added, while the total mass per storey increases significantly as well. However, the number of bracings remains the same in all buildings, as they are only installed in the middle bays and, therefore, the ratio of the stiffness contributed by the bracings to the total storey stiffness of each building cannot the same. Thus, a 3×3-bay building with 8 bracing members and 16 columns at each storey behaves basically as a braced frame. On the other hand, in a 9×9-bay building with 100 columns per storey, the contribution of bracings is of reduced significance and its structural performance is basically that of a moment resisting frame, rather than that of a braced frame. Indeed, the bracings' contribution to the total storey stiffness is reduced from about 75% down to less than 10% as the number of bays is increased from 3×3 to 9×9.

A total number of 66 optimization runs were carried out to optimize pure steel and composite buildings for the same range of δ-values as in the case of the reference building; the results are depicted in Figs. 5-7. In Fig. 5 (3×3 bays), it can be noticed that roughly the same cost is retained for a relatively wide range of low horizontal displacement capacity requirements δ. This is an expected outcome, because, when the contribution of bracings to the total stiffness is increased, the proportion of horizontal loads they receive is increased as well; therefore, for low δ-values, small column sections suffice, as these are mainly defined due to the gravitational loads. In the case of the 7×7-bay and 9×9-bay buildings (Figs. 6 and 7), the corresponding δ-range retaining a constant cost is much shorter.

The general tendency of the curves obtained in Figs. 5-7 is not changed compared to that of the reference building. However, the intersection point of the pair of curves in each figure is shifted. For the 3×3-bay building, the intersection point is at $\delta \approx 1.8$, increasing the range of values for which pure steel columns can provide more cost-effective solutions than composite columns. On the other hand, the two curves intersect at lower δ-values for the 7×7-bay and 9×9-bay buildings: at $\delta \approx 0.8$ and $\delta \approx 0.55$, respectively. The decisive difference between the 4 plan configurations of the buildings examined is the increase in the number of columns as the number of bays per direction increases while the number of bracings per storey remains the same. Hence, bracings seem to have a more critical effect to pure steel design than to steel-concrete composite design. For larger numbers of columns, the bracings' effect is reduced, while the total concrete area in the steel-concrete composite columns is increased significantly. It is also interesting to note the less steep curves corresponding to composite design in Figs. 3-7, which implies that the composite design adapts better to varying column numbers, without abrupt increases of the total structural cost as δ is increased. Hence, the reinforced concrete encasing the steel section in larger numbers of steel-concrete composite columns appears to substitute effectively the proportion, by which the contribution of bracings to the total stiffness is reduced, rendering composite design more suitable for moment resisting frames than pure steel design.

Another important aspect is that, when optimizing using pure steel members, the largest available column and bracing sections in the respective databases need to be utilized to reach

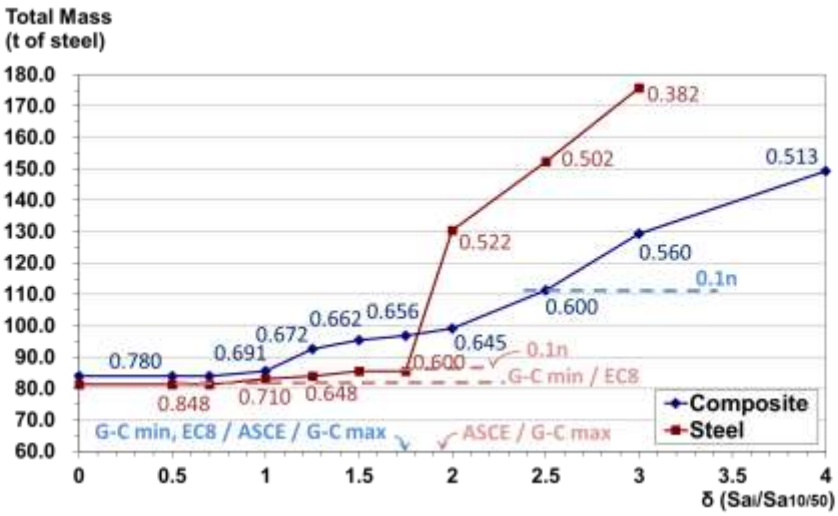

Fig. 5. Total equivalent steel mass versus δ for C/S-6-3-δ
(the values illustrated next to each point are the corresponding maximum fundamental periods)

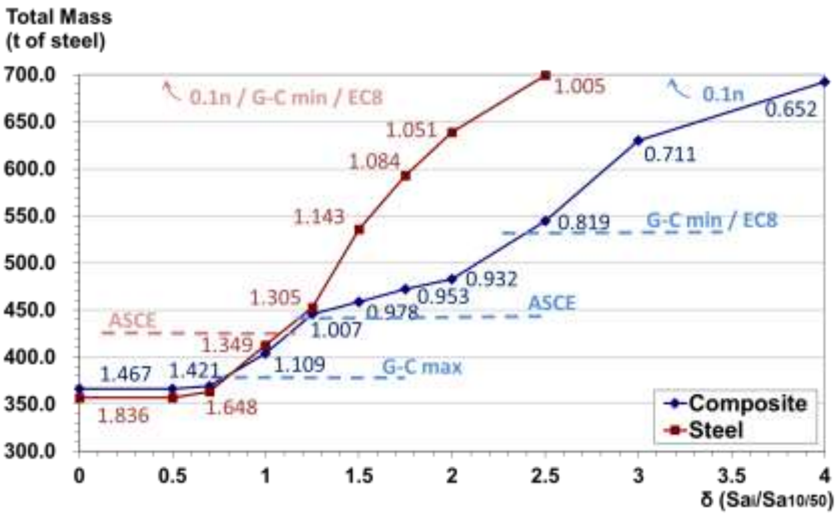

Fig. 6. Total equivalent steel mass versus δ for C/S-6-7-δ
(the values illustrated next to each point are the corresponding maximum fundamental periods)

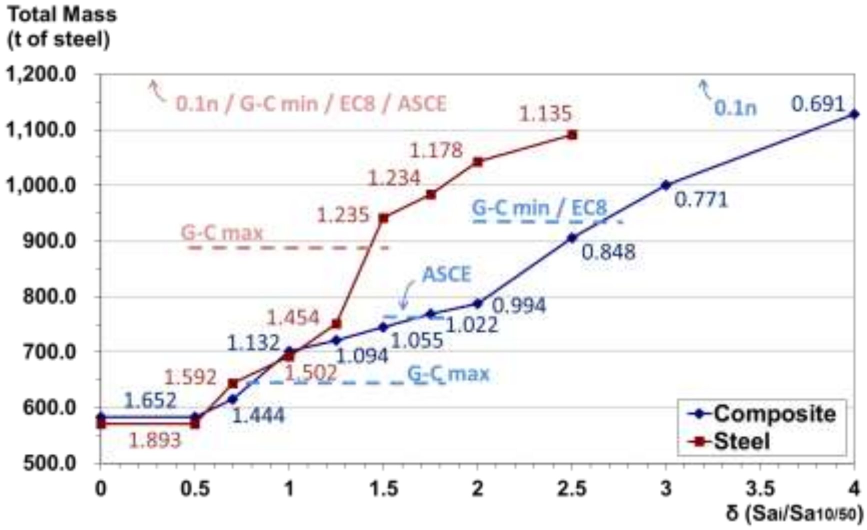

Fig. 7. Total equivalent steel mass versus δ for C/S-6-9-δ
(the values illustrated next to each point are the corresponding maximum fundamental periods)

δ-values up to 2.0 for the 7×7-bay and 9×9-bay buildings, 2.5 for the reference building and 3.0 for the 3×3-bay building. Using steel-concrete composite columns, feasible solutions can be determined for δ-values up to 4.0 for all numbers of bays investigated. This outcome is indicative of the significant role of concrete in providing lateral stiffness to a composite building. Nevertheless, buildings designed with very high seismic resistance demands ($\delta \rightarrow 4$), especially the ones with low overall area over height ratio (e.g. 3×3-bay building), need to be carefully assessed, as certain columns or column parts may be in tension instead of compression. Composite columns in that state have lost the contribution of concrete in the part of the section that is under tension and can safely count only on their steel cores to receive tensile stresses.

As regards the fundamental periods of the optimized designs, the same general tendency is exhibited by both pure steel and composite approaches: for larger number of bays, the proportional stiffness contribution of bracings is reduced, which leads to overall more flexible designs with higher fundamental periods. Buildings with large numbers of bays are able to meet the specified seismic performance criteria thanks to their high overall area over height ratio. This effect is more pronounced in the cases of the pure steel design approach, for which the approximation formulas of section 4 yield more restrictive fundamental period bounds, as can be verified from Figs. 3 and 5-7.

5.4. The effect of building height (2 and 4 storeys)

In this final set of optimization runs, 2 more buildings are simulated, a 4-storey (C/S-4-5-δ) (Fig. 2(c)) and a 2-storey building (C/S-2-5-δ), both of which have an identical element configuration as the reference building, so their only geometrical difference is the number of storeys. The new buildings have a lower height to area ratio and, consequently, increased inherent structural stiffness. Forty-four optimization runs were performed for this investigation, the results of which are presented in Figs. 8 and 9.

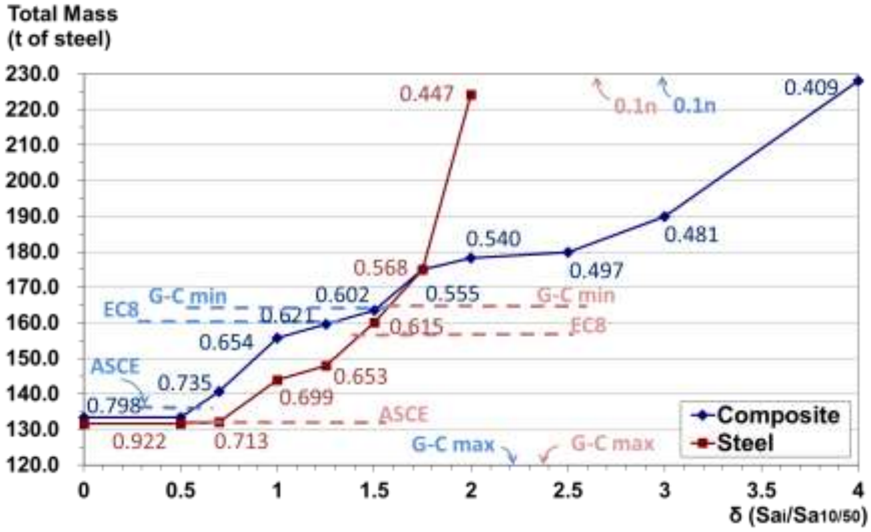

Fig. 8. Total equivalent steel mass versus δ for C/S-4-5-δ
(the values illustrated next to each point are the corresponding maximum fundamental periods)

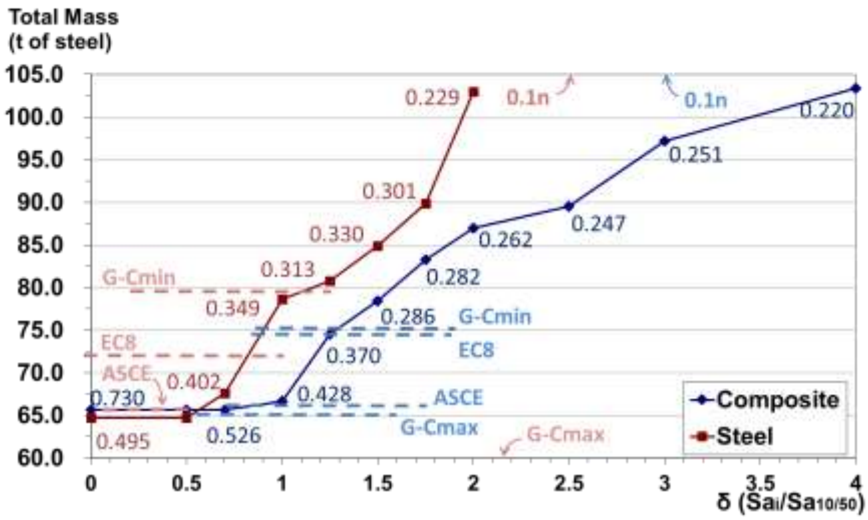

Fig. 9. Total equivalent steel mass versus δ for C/S-2-5-δ
(the values illustrated next to each point are the corresponding maximum fundamental periods)

The effect of the number of storeys on the shape of the total-mass-versus-δ curves seems to be similar to that of the number of bays. For the 2-storey building, the pair of curves in Fig. 9 intersect at a much lower δ-value (around 0.5) than for the 6-storey reference building, indicating that composite designs are more favourable for low-rise buildings. Apparently, due to the inherent lateral stiffness of the 2-storey building, the induced need for additional stiffness is relatively low and is inexpensively satisfied by the encasing concrete in composite columns (more costly steel bracings are not required as in the pure steel building). In the case of the 4-

storey building, the two curves in Fig. 8 are relatively close for δ-values lower than 2.0. In the case of the 2-storey building, the distance between the two curves is larger. This highlights the favourable cost-effectiveness of steel-concrete composite designs over pure steel designs for low-rise buildings. The fundamental periods of the optimized designs seem to be analogous to those of the designs defined for the reference building. Similar trends with Fig. 3 are also observed regarding the fundamental period values obtained from the approximate formulas of section 4.

6. Concluding remarks

In this work, a total number of 154 structural optimization runs were performed to comparatively assess buildings with pure steel and steel-concrete composite columns with respect to their total material cost for a variety of horizontal displacement capacity demands. The optimization procedure employed enabled a fair comparison between these two design approaches, as the assessment of optimized designs ensures that each approach has been applied as effectively as possible to meet structural performance requirements with the least feasible cost of materials. The displacement modification factor δ introduced in this work to adjust ductility demands facilitated the assessments performed.

The total mass versus horizontal displacement requirement curves presented herein show that the relation between cost and capacity tends to be roughly linear for structures with concrete-encased composite columns. On the other hand, for pure steel designs, the observed behaviour tends to be roughly bilinear, with an abrupt gradient change in the linear relation occurring at certain value of factor δ. In general, steel-concrete composite designs were found to be more favourable than pure steel designs for higher seismic demands. In such cases, more cost-effective structural solutions are attained by partially replacing the contribution of steel using concrete, which is a significantly less expensive material.

The level of seismic demand, beyond which buildings with steel-concrete composite columns are more economical than with pure steel ones, is denoted by the intersection point of the pair of cost-versus-δ curves for composite and pure steel design approaches. The δ-value of the intersection point seems to depend mainly on the overall stiffness of the structural system considered. In fact, the intersection point tends to be at a lower δ-value for a structural system with a higher inherent stiffness, *i.e.* with a higher plan area to height ratio.

The composite design approach seems to adapt better to increasing seismic demands, since there is no need to alter the design philosophy of the optimum solution. The relation between the minimum cost and the horizontal displacement demand is practically linear, which implies that increased stiffness is achieved by a global increase of all member sections in the structure. On the other hand, the pure steel design approach seems to favour the bracings' contribution to the total stiffness, in order to cope with increasing seismic demands. Thus, the utilization firstly of all available bracing sections and then the increase in the columns' sections appears to be the most cost-effective approach for pure steel buildings. Hence, there is a change of design philosophy when the largest available bracing section is not sufficient anymore as the seismic demand is increased, therefore the relation between the minimum cost and the horizontal displacement demand is bilinear. It is evident that optimized buildings using pure steel design belong more to the category of braced frames, while optimized buildings with steel-concrete composite columns are primarily moment resisting frames.

A single cost ratio of concrete price over steel price was used throughout this work to convert concrete volume to equivalent steel mass. This ratio may vary in different regions, for dynamically changing global/local market conditions and for different applications. Such cost variations can have an effect on the presented design optimization results. An investigation of this effect is provided in [15], which reveals that, for typical cost ratios expected in the market, the essential features of the optimal design identified in each case are not significantly changed due to the cost variations, although the final objective function value may exhibit significant variations. In the present study, the optimality of each design approach seems to be more critically affected by the slope of the cost-versus-δ curve. Hence, a modification of the cost ratio is expected to shift the curves of the buildings with steel-concrete composite columns, with the corresponding slopes, however, being much less affected.

In the performed optimization runs, fundamental periods were determined using eigenvalue analyses. A maximum allowable limit for periods was not explicitly set in the optimization procedure, although any optimized design detected was assessed with respect to its maximum fundamental period. Application of such a constraint using approximate formulas defined by design codes or other sources might result in a considerable increase of the total cost [29], leading to significant overstrength of the optimally designed buildings. Nevertheless, such a consideration is important for the design of a building. Imposing an appropriate constraint will prevent the optimization algorithm from determining optimized designs with undesirable high periods.

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