COMPDYN 2013
4th ECCOMAS Thematic Conference on
Computational Methods in Structural Dynamics and Earthquake Engineering
M. Papadrakakis, N.D.Lagaros, V. Plevris (eds.)
Kos Island, Greece, 12–14 June 2013

COMPARATIVE ASSESSMENT OF BUILDINGS WITH PURE STEEL OR STEEL-CONCRETE COMPOSITE COLUMNS USING STRUCTURAL DESIGN OPTIMIZATION

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Keywords: Composite steel-concrete, structural optimization, seismic design

Abstract. *The present work investigates the cost-effectiveness of fully encased composite columns and concrete filled tubes as an alternative to pure steel I-shaped columns. In order to achieve objectiveness in the comparison of the three methods, the designs assessed are obtained by a structural optimization procedure. Thus, any subjectivity which would result from the designer's experience is avoided. The Evolution Strategies optimization algorithm is employed to minimize structural cost subject to constraints associated with: (a) Eurocode 4 provisions for safety of composite column-members, (b) Eurocode 3 provisions for safety of steel structural members and (c) structural system resistance. Furthermore, the assessment is performed for a variety of seismic intensities, as it provides important information on the ability of each method to meet high performance criteria and the total cost increase such requirements would lead to. The results obtained provide an insight to the advantages of partially substituting steel as a main structural material with concrete, in composite column sections, and demonstrate the effectiveness of the proposed optimization approach.*

1 INTRODUCTION

Steel-concrete composite elements are intended to fill the gap between reinforced concrete elements and pure steel elements. The utilization of steel-concrete composite elements is not a new concept, since they have gradually gained popularity during the course of the 20th century mainly in North America, Japan and Europe, while early applications of such elements at the end of the 19th century have been recorded. Over the past few decades, numerous steel-concrete composite structures have been erected worldwide. This form of construction is seen as an alternative mainly to constructing pure steel structures. The increasing preference in composite elements can be primarily attributed to the fact that concrete, a significantly less expensive material compared to steel, is utilized in an effort to cost-effectively replace a percentage of the required steel sections area. This way, overall material cost in a structure can be reduced and, at the same time, better lateral support and fire protection of the steel elements can be achieved, since concrete (which usually covers steel elements) as a material exhibits a much better performance at high temperatures than steel.

Although the incorporation of steel-concrete composite elements in a structure is nowadays regarded as established design and construction practice, a formal and extensive investigation on the conditions under which such practice is more cost-effective than other alternatives has not been conducted yet. The purpose of the present work is to comparatively assess multi-storey buildings with pure steel or steel-concrete composite columns with respect to their cost effectiveness and seismic performance. In order to ensure that 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 structural designs attained do not depend on a particular designer's experience and subjectivity, but are the outcome of an objective automatic design optimization procedure.

In the framework of this paper, structural optimization is applied for the design of buildings with steel-concrete composite columns, which consist of: (a) steel members with standard I-shaped sections (HEB) fully encased in concrete or (b) concrete-filled steel tubes with standard hollow sections (CHS or RHS). Moreover, buildings with pure steel columns are optimally designed using the above mentioned standard steel 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 using a displacement-controlled nonlinear static pushover analysis to the targeted top displacement, as it is defined in FEMA-440.

Optimal structural designs are identified for a variety of seismic intensities by means of an Evolution Strategies optimization algorithm. The optimization results allow for an objective comparison of various designs in terms of required material cost and achieved capacity to withstand earthquake actions.

2 OPTIMIZATION PROBLEM

The Evolution Strategies optimization algorithm [2,7] used in this work, in order to determine the most cost-effective design for each scenario considered, has been found to be suitable for structural optimization problems [8]. As an optimization problem, its aim is to minimize the objective function, by systematically selecting combinations of the independent variables and checking the satisfaction or not of the defined constraints.

2.1 Objective Function

Since the objective of this paper is to assess the cost-effectiveness of different design methods, the objective function used was the total cost of the materials of the structural elements. In common engineering practice, the estimation of the structural cost incorporates several parameters which depend on the region where the works will take place. Such parameters are the total labor cost, the availability of the materials on the market, the characteristics of the soil, etc. In this work all the aforementioned were considered to be incorporated into the cost P_C and P_S of the concrete and steel, respectively. Additionally, all structural parts and details which can be designed separately, such as the slabs, the beam-column connections, the bracings' connections and the foundation scheme (including the column base connections) were excluded from the total cost calculation. However, their contribution to the structural performance was taken into account in the structural modeling process. Taken all these into consideration, it becomes obvious that, in this work, the term total cost refers to the cost of the materials for beams, columns and bracings only. Furthermore, because the beams and bracings in all designs are simulated as pure steel sections only, one can understand that the cost of concrete refers specifically to the steel-concrete composite columns. The total material cost could be simply calculated as:

$$P_{tot} = P_C \cdot V_C + P_S \cdot V_S \tag{1}$$

where

P_{tot}: the total cost calculated in local currency
P_C : the total cost for the concrete in local currency per m^3
P_S : the total cost for the steel in local currency per m^3
V_C: the total volume of concrete (m^3)
V_S : the total volume of steel (m^3)

In Equation 1 the total cost is calculated in monetary units, so its actual value should be constantly altered in order to be consistent with the current prices. Therefore, one can understand that, after any change to the prices of the materials, the currency exchange rate or the labour cost, this work would be rendered outdated. In order to avoid this implication, a more robust equation had to be used. Instead of using the average current prices of the materials, the total cost was calculated in equivalent steel weight. Such an assessment was made possible by introducing the ratio of concrete cost over steel cost, in order to convert the total volume of concrete to equivalent steel volume. Finally, the total equivalent steel volume was converted into weight by multiplying by the density of steel. in order to provide the final value in equivalent tons of steel. The final form of the objective function used is described in Equation 2:

$$\frac{W_{tot}}{\gamma_{\mathrm{steel}}} = V_{tot} = CR \cdot V_C + V_S \tag{2}$$

where

W_{tot} : the total material cost calculated in equivalent steel weight (tn of steel)
γ_{steel}: the density of steel (tn/m^3)
C_R: the ratio of the concrete cost to the steel cost ($CR = P_C/P_S$)
V_C: the total volume of concrete on the storey (m^3)
V_S : the total volume of steel on the storey (m^3)

2.2 Design Variables

Use of structural optimization implies a large number of structural simulations which need to take place, until the optimization algorithm converges to an optimum solution. Additionally, all designs simulated during the procedure, are evaluated using specific performance criteria.

In order to significantly reduce to the total time required for the whole optimization procedure to acceptable limits, one needs to reduce the number of design variables as much as possible. It was noticed in previous works [10,11] that the division of the columns into four groups, as they are described in Paragraph 3, was effective in limiting the computational effort and, at the same time, allowing the algorithm to reduce the total structural cost significantly, comparing to using the same section for all columns.

At this point, it has to be noted that the rest of the design parameters such as the dimensions of the buildings and the mechanical properties of the materials of both the column and the beam sections were the same for all scenarios. Also, the concrete cover of the fully encased composite columns had been found to have a minimal effect to the total performance of the structure, so its value also remained the same for all designs.

Six design variables were defined for this particular optimization problem: four variables for the steel sections of the columns, one for the beams and one for the bracings. Standard IPE sections were used for the beams, while the columns' database consisted only of HEB sections for the fully encased composite columns and various RHS and SHS sections for the concrete filled tubes. For the bracings, specific L type sections were used.

2.3 Performance Criteria & Constraints

All the constraints of the optimization problem are defined by the design codes and methodology used. They can be categorized into two groups: one consisting of provisions related to the overall structural behaviour, and another comprising of the individual member checks. The two groups differ mainly in the way they affect the optimization procedure. The overall performance criteria define the number and type of analyses required in order to evaluate each design. On the other hand, the individual member criteria refer to the structural elements' capacity, so they need to be checked during all analyses.

2.3.1. Overall Performance Criteria

Naturally, earthquakes are events with a probability of occurrence; therefore a probabilistic approach of the problem and use of various accelerograms would be required in order to simulate more accurately the seismic actions. However such an assessment would increase dramatically the computational time needed for an optimization and would deviate from the objectives of this paper. Hence, the nonlinear static pushover analysis with deterministic criteria was used instead.

Two displacement-controlled pushover analyses up to the targeted top displacement described in F.E.M.A. 440 [1,5], needed to take place [12], one for each direction on the horizontal plane. The feasibility criterion used was the maximum inter-storey drift, which was calculated for each pair of analyses.

$$\Delta_{target} = C_0 \cdot C_1 \cdot C_2 \cdot C_3 \cdot \frac{S_a}{\omega^2} \tag{3}$$

where

Δ_{target}: the targeted top displacement of a M.D.O.F. system to be used in the pushover analysis

C_i: coefficients used in order to convert the S.D.O.F. to M.D.O.F. displacement

Sa: the design pseudo-acceleration defined for a S.D.O.F. system with fundamental period T

ω: the fundamental frequency of the structure (= 2π/T)

2.3.2. Individual Member Checks

The structural elements of the buildings were checked independently during both analyses per design for the capacity criteria defined in the Eurocodes. EN1993 [3] was used to check all beams, bracings and columns which were designed as pure steel sections. They were checked for all types of actions: bending moment, shear and axial force, as well as the respective buckling types that might occur as a result of these actions. It has to be noted that the bracings are subjected only to axial force, so they do not need to be checked for the other types of actions. The steel section capacities are calculated by:

$$M_{Rd} = \frac{f_{yk} \cdot W_{el}}{\gamma_{M0}} \tag{4}$$

$$V_{Rd} = \frac{f_{yk} \cdot A_V}{\gamma_{M0}} \tag{5}$$

$$N_{c,Rd} = \frac{f_{yk} \cdot A_{tot}}{\gamma_{M0}} \tag{6}$$

where

M_{Rd}, V_{Rd}, $N_{c,Rd}$: the design bending moment, shear and axial force capacity respectively

W_{el} : the elastic moment of resistance of the steel section

A_V : the effective shear area for each direction

A_{tot}: the total area of the section

f_{yk} : the nominal yielding stress of the steel category used

γ_{M0} : the safety factor used for sections of category 1 to 3

All fully encased steel and concrete composite sections and concrete filled tubes were checked for all design actions mentioned for the beams, according to the EN1994 [4]. The axial shear force criterion, used in order to determine the number and diameter of the required shear headed stud connectors in composite columns, was also checked during the analysis procedure, however it was found not to render a solution unfeasible at any case, so one could ignore it in order to speed up the optimization procedure, as long as there is a preliminary check which confirms that all sections available have the dimensions required for the installation of the headed studs. Considering that the composite operation of the columns ensured, their total capacity can be calculated as the sum of the respective concrete and steel part capacities:

$$M_{Rd,tot} = M_{C,Rd} + M_{S,Rd} \tag{7}$$

$$V_{Rd,tot} = V_{C,Rd} + V_{S,Rd} \tag{8}$$

$$N_{Rd,tot} = N_{C,Rd} + N_{S,Rd} \tag{9}$$

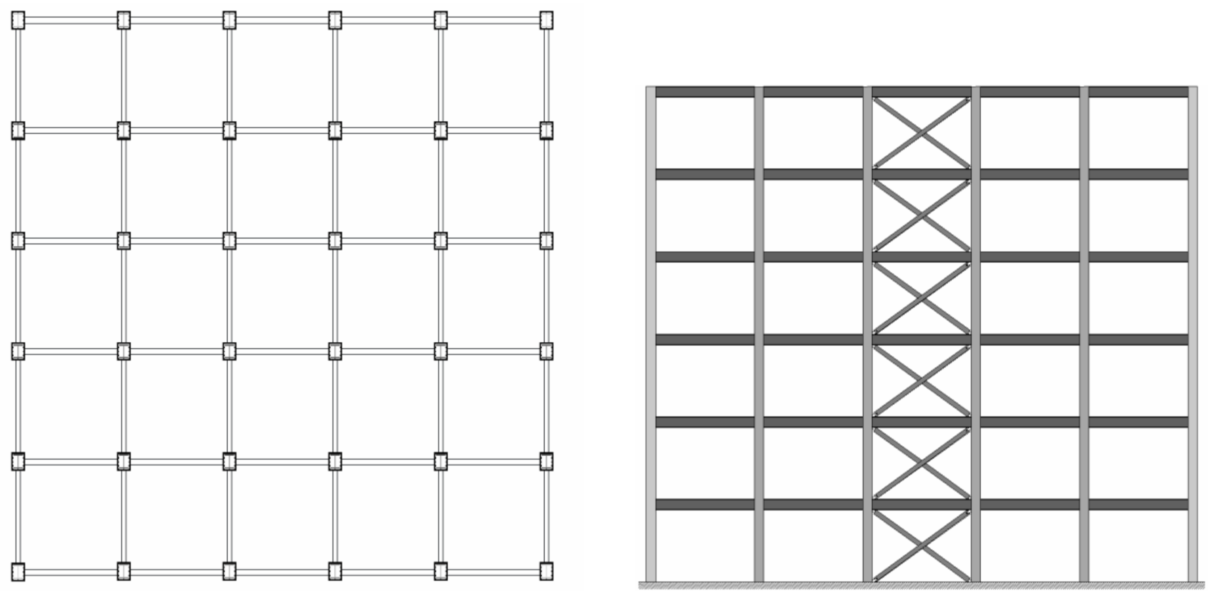
where

$M_{Rd,tot}$, $V_{Rd,tot}$, $N_{Rd,tot}$:	the design bending moment, shear and axial force capacity of the composite section
$M_{C,Rd}$, $V_{C,Rd}$, $N_{C,,Rd}$:	the design bending moment, shear and axial force capacity provided by the concrete part of the section
$M_{S,Rd}$, $V_{S,Rd}$, $N_{S,Rd}$:	the design bending moment, shear and axial force capacity provided by the steel part of the section

3 STRUCTURAL SIMULATION

The reference building simulated was a six-storey symmetrical building of 5 bays per direction. Its dimensions were 5.00m for each beam's span, in both directions, and 3.50m for the height of each storey. Bracings were used in the middle bay at each of the four sides. In Figure 1 is illustrated the horizontal and vertical layout of the elements, in an indicative floor plan and side view respectively.

All structural members were modeled with fibber elements. This element type is considered to be suitable for analyses where brittle types of failure are not expected to occur. As described in Section 2.2, the columns were divided into four groups regarding their location in the floor layout: group 1 includes all design variables associated with the corner columns, groups 2 and 3 refer to all side columns in x-direction and y-direction, respectively, and group 4 involves all internal columns.



(a) **(b)**

Figure 1: (a) Floor plan and (b) side view of the reference building

All gravitational loads from the slabs were modeled directly as distributed loads on the beams. Nonlinear static analysis under the combination of dead and live loads would take place before the implementation of the horizontal displacements. All structural simulations were performed using the OpenSEES software (Open System for Earthquake Engineering Simulation) [9]. The effect of the slabs to the structural performance was simulated by the definition of a rigid diaphragm on each floor level. Also, all columns were considered to be fixed on their base. Finally, the beam-to-column connections were modeled both as hinges and fixed connections. The first scenario would provide the maximum bending moment in the beam's span and, consequently define the minimum required beam section. The fixed connection results in transfer of an amount of the beam's bending moment to the columns, increasing the capacity demands on bending moment for the columns.

4 NUMERICAL RESULTS

The aim of this work is to examine the advantages and disadvantages of the three different columns types in the context of design against earthquake. In order to define the capabilities of each design methodology, an investigation of their cost-effectiveness in increased horizontal deformation demands was performed. This was achieved by introducing the coefficient δ [10] in the calculation of the targeted top displacement defined in Equation 3.

$$\Delta_{target} = \delta \cdot C_0 \cdot C_1 \cdot C_2 \cdot C_3 \cdot \frac{S_a}{\omega^2} \qquad (10)$$

It becomes obvious that, values of δ higher than 1.0 simulate increased ductility demands, i.e. more strict design codes. So, for δ = 1.50, the structure is required to reach 50% further on the horizontal plane, remaining, however, at the same performance level. Eight values of δ ranging from 0.50 to 2.0 were used for the purposes of this research. Also, an extra optimization for gravitational loads only took place for each design method, in order to define the actual increase to the total cost due to the seismic requirements. All structures were required to perform within the collapse prevention limit state. The results obtained from the optimizations are illustrated in Figure 2.

It has to be noted here that the placement of the bracings in the middle bay of the structure was not an arbitrary decision, but it came as a result of preliminary structural optimizations. These analyses had shown that this option allows the optimization algorithm to provide more cost-effective solutions than installing them in the corner bays. The later option failed specially for low values of δ, which for the minimum bracings section was required, as it resulted in use of twice as much the steel as the scheme finally adopted.

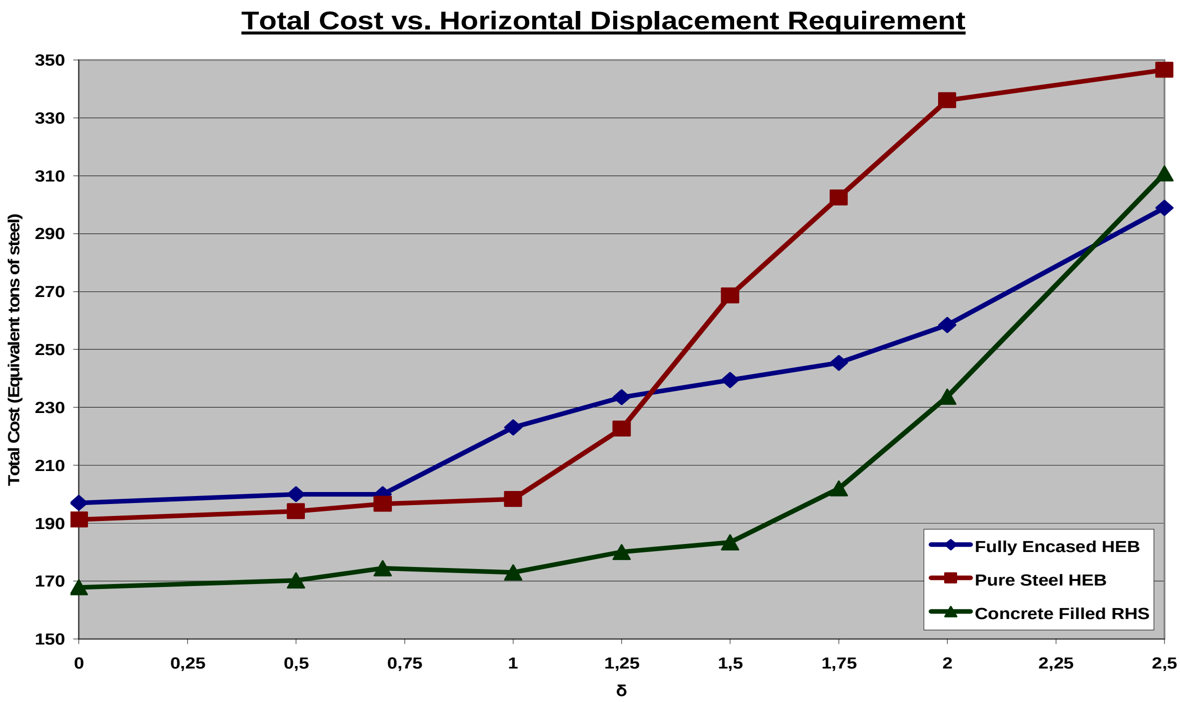


Figure 2: Optimized designs' total cost versus horizontal displacement capacity demand

	y-y'		z-z'			y-y'		z-z'	
	K_s/K_{tot}	K_c/K_{tot}	K_s/K_{tot}	K_c/K_{tot}		K_s/K_{tot}	K_c/K_{tot}	K_s/K_{tot}	K_c/K_{tot}
HE100B	26,4%	73,6%	11,7%	88,3%	**HE340B**	60,3%	39,7%	31,1%	68,9%
HE120B	32,4%	67,6%	14,8%	85,2%	**HE360B**	61,2%	38,8%	31,1%	68,9%
HE140B	37,5%	62,5%	17,5%	82,5%	**HE400B**	62,4%	37,6%	30,7%	69,3%
HE160B	42,0%	58,0%	20,0%	80,0%	**HE450B**	63,5%	36,5%	30,4%	69,6%
HE180B	45,6%	54,4%	22,3%	77,7%	**HE500B**	64,5%	35,5%	30,1%	69,9%
HE200B	48,7%	51,3%	24,3%	75,7%	**HE550B**	64,6%	35,4%	29,1%	70,9%
HE220B	51,4%	48,6%	26,1%	73,9%	**HE600B**	64,7%	35,3%	28,3%	71,7%
HE240B	53,6%	46,4%	27,7%	72,3%	**HE650B**	64,8%	35,2%	27,5%	72,5%
HE260B	54,9%	45,1%	28,5%	71,5%	**HE700B**	65,0%	35,0%	26,9%	73,1%
HE280B	56,1%	43,9%	29,3%	70,7%	**HE800B**	64,4%	35,6%	25,2%	74,8%
HE300B	57,7%	42,3%	30,5%	69,5%	**HE900B**	64,5%	35,5%	24,3%	75,7%
HE320B	59,3%	40,7%	31,1%	68,9%	**HE1000B**	64,0%	36,0%	23,0%	77,0%

Table 1: Stiffness contribution of each material to the composite section (fully encased sections)

	Dimensions (mm)			y-y'		z-z'			Dimensions (mm)			y-y'		z-z'	
	h	b	t	K_s/K_{tot}	K_c/K_{tot}	K_s/K_{tot}	K_c/K_{tot}		h	b	t	K_s/K_{tot}	K_c/K_{tot}	K_s/K_{tot}	K_c/K_{tot}
1	200	100	5	75,7%	24,3%	82,3%	17,7%	**26**	300	300	10	76,9%	23,1%	76,9%	23,1%
2	200	200	5	70,5%	29,5%	70,5%	29,5%	**27**	400	200	10	75,7%	24,3%	82,3%	17,7%
3	200	100	8	84,7%	15,3%	89,7%	10,3%	**28**	200	200	16	91,4%	8,6%	91,4%	8,6%
4	250	150	6	73,2%	26,8%	78,6%	21,4%	**29**	250	150	16	90,6%	9,4%	93,4%	6,6%
5	220	220	6	72,5%	27,5%	72,5%	27,5%	**30**	300	200	12,5	83,6%	16,4%	86,8%	13,2%
6	200	100	10	88,2%	11,8%	92,5%	7,5%	**31**	250	250	12,5	84,6%	15,4%	84,6%	15,4%
7	300	200	6	68,0%	32,0%	72,7%	27,3%	**32**	260	260	12	83,2%	16,8%	83,2%	16,8%
8	250	250	6	69,5%	30,5%	69,5%	30,5%	**33**	350	350	10	73,6%	26,4%	73,6%	26,4%
9	260	260	6	68,6%	31,4%	68,6%	31,4%	**34**	300	200	16	87,6%	12,4%	90,3%	9,7%
10	250	150	8	79,3%	20,7%	84,0%	16,0%	**35**	250	250	16	88,5%	11,5%	88,5%	11,5%
11	200	200	8	80,6%	19,4%	80,6%	19,4%	**36**	500	300	10	68,9%	31,1%	74,7%	25,3%
12	220	220	8	78,7%	21,3%	78,7%	21,3%	**37**	400	400	10	70,5%	29,5%	70,5%	29,5%
13	200	100	12,5	91,2%	8,8%	94,7%	5,3%	**38**	260	260	16	87,9%	12,1%	87,9%	12,1%
14	300	300	6,3	66,3%	33,7%	66,3%	33,7%	**39**	450	250	12,5	77,0%	23,0%	82,6%	17,4%
15	250	150	10	83,5%	16,5%	87,6%	12,4%	**40**	350	350	12,5	78,4%	21,6%	78,4%	21,6%
16	220	220	10	83,0%	17,0%	83,0%	17,0%	**41**	400	200	16	84,7%	15,3%	89,7%	10,3%
17	300	300	8	72,0%	28,0%	72,0%	28,0%	**42**	300	300	16	85,7%	14,3%	85,7%	14,3%
18	400	200	8	70,6%	29,4%	78,0%	22,0%	**43**	400	400	12	74,7%	25,3%	74,7%	25,3%
19	200	200	12,5	88,1%	11,9%	88,1%	11,9%	**44**	500	300	12,5	74,1%	25,9%	79,4%	20,6%
20	300	200	10	79,4%	20,6%	83,1%	16,9%	**45**	400	400	12,5	75,6%	24,4%	75,6%	24,4%
21	250	250	10	80,6%	19,4%	80,6%	19,4%	**46**	450	250	16	81,9%	18,1%	86,8%	13,2%
22	220	220	12	86,0%	14,0%	86,0%	14,0%	**47**	350	350	16	83,1%	16,9%	83,1%	16,9%
23	260	260	10	79,8%	20,2%	79,8%	20,2%	**48**	500	300	16	79,3%	20,7%	84,0%	16,0%
24	450	250	8	66,7%	33,3%	73,5%	26,5%	**49**	400	400	16	80,6%	19,4%	80,6%	19,4%
25	350	350	8	68,4%	31,6%	68,4%	31,6%								

Table 2: Stiffness contribution of each material to the composite section (hollow sections)

An upper and a lower cost limit are defined, which differ for each curve. The upper limit is related to the availability of sections in the databases used. In other words, it is the total cost of a building designed with the largest sections available for the simulated structural members. On the other hand, the lower limit is defined by the total cost of an optimized structure, which was designed for gravitational loads only, i.e. for $\delta = 0$.

An initial investigation of the results shows that, for low values of δ, all optimized designs' cost curves converge to their lower limit. Even though, it is logical to assume that a building designed only for gravitational loads also has a capacity for horizontal deformation, one would expect it to be minimal. However, it can be noticed that this capacity seems to be significant enough and, by the means of structural optimization, it could meet the current code requirements with negligible increase to its total material cost.

Each of the three curves, depicting the correlation between the total material cost and the horizontal displacement requirements, seems to be of a different type. In particular, the cost versus δ curve for the fully encased composite steel-concrete sections seems to have a mainly linear behavior. On the other hand, the one for the pure steel sections can be defined as bilinear, as its gradient changes dramatically for a δ value between 1.00 and 1.25. Finally, the behavior of the concrete filled tubes' curve could be characterized something between the two aforementioned types. One can notice that it is definitely not linear, since its gradient increases significantly for large values of δ. However, it is not bilinear either, as this change takes place smoothly. At any case, it is similar to the one of the pure steel columns curve.

In order to explain this type of behaviour, one needs to calculate the contribution of each material to the total section stiffness. This can provide an estimation of the behaviour to which a composite section is more similar. Tables 1 and 2, present this ratio for each material, for the fully encased HEB sections and the composite hollow sections. Taking into consideration that only HE200B or larger sections have been used in the first scenario, the contribution of concrete to the total stiffness ranges from 35.0% to 51.3% around the y-y' axis, and from 23% to 31% around the z-z' axis. In other words, the HEB sections' stiffness is increased from 53.8% up to 105.3% when they are encased in concrete. On the other hand, in the composite hollow sections, steel seems to be the dominant material. Its stiffness ranges from 86.3% to 91.4% and from 66.3% to 94.7% on the y-y' and z-z' axes respectively. Therefore, it becomes clear that the hollow sections' behavior is expected to be more similar to the pure steel design, than to the fully encased sections.

Also, what is remarkable is that designs which would seem unfeasible otherwise, satisfying the desired performance criteria, become available without a significant increase to the materials cost (61.2% for the fully encased sections). This observation is indicative of the efficiency of the optimization method, when it is used on structural design. It should also be noted that, in all feasible design scenarios, the usage ratio of the shear capacity of the columns ranged from 10% to 20%. Therefore, the assumption made during modelling that no brittle failure of the simulated structural members is expected to occur is verified.

5 CONCLUSIONS

- The application of optimization algorithms on structural design [6] has been proven a powerful tool which allows engineers to reduce significantly the total cost, without any compromise in the quality of the buildings.
- Structural design optimization enables the improvement of structural performance, far beyond their conventional limits, without excessive increase on their total cost.

- As the design codes become more demanding, in an effort to protect the structures against more dangers, optimization algorithms are expected to become necessary to designer engineers.
- Further decrease to the total cost of structures designed for high demands is feasible by further division of the element groups (e.g. by storey). However it results in significantly increased computational cost, therefore the engineer needs to determine the optimum configuration of the optimization problem.
- Steel-concrete composite design becomes more advantageous than pure steel design for increased demands, as it enables the definition of more cost-effective solutions. The increased cost-effectiveness is achieved by appropriately using concrete in order to partially replace the contribution of steel.
- Not being susceptible to buckling failure, concrete filled tubes are able to provide more cost-effective designs than fully encased I-shaped sections, for low horizontal displacement requirements. However, having a performance which is more similar to pure steel sections, due to the high contribution of steel, they lose this advantage, as the demands increase.
- An engineer's decision on the most preferable design scheme cannot be based only on the cost-effectiveness of the method. More parameters need to be taken into consideration, such as the applicability of the design method or its practicality regarding the building use.
- Increased fire resistance of the fully encased composite sections, thanks to the surrounding concrete, is an aspect which was not taken into consideration in the present paper; however it is important for a designer engineer in practice.

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