Design Optimization of Steel - Concrete Composite Structures with Requirements on Progressive Collapse Resistance

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SUMMARY:
The present work presents a sizing optimization procedure for collapse-resistant composite steel-concrete frames. An evolutionary 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 beam-members, (c) structural system resistance and (d) progressive collapse resistance. In the numerical examples tested, a variety of damage scenarios is considered. These scenarios are realized by artificially removing column-members from the structural system. The results obtained demonstrate the effectiveness of the proposed optimization approach. Of particular importance is the investigation of the variation in the structural cost achieved when collapse resistance constraints are incorporated in the design process. By enforcing the satisfaction of additional design requirements on system resistance and safety against local failure, structural cost is inevitably increased. This increase is quantitatively explored by comparing designs obtained with and without collapse resistance constraints.

Keywords: Composite Steel-Concrete, Structural Optimization. Progressive Collapse

1. INTRODUCTION

Since the collapse of the World Trade Center towers in September 11, 2001, the need to take into account progressive collapse scenarios, when designing structures has been increasing. However, this is not a newly developed concept; since the Ronan Point collapse in London on May 16, 1968, it has been a major concern amongst structural engineers. The term progressive collapse is referring to a large scale damage which occurs as a result of a chain reaction failure initiated by a minor loss of structural integrity. Even though, statistically, the possibility of such an event is relatively small, it is a type of failure which happens almost instantaneously after the initial damage, so it is an unacceptable hazard for the majority of structures. Design strategies in order to control the amount of damage or reduce the risk of initial failure, such as the use of column removal scenarios, have been proposed (COST TU0601, 2011), however they result in significantly increased structural cost. This work aims to show how the application of an optimization algorithm in the design procedure of composite steel - concrete structures can provide a cost effective solution, when, apart from the requirements against seismic loads, extra criteria against progressive collapse are implemented.

2. THE OPTIMIZATION PROBLEM

At any optimization problem, the aim is to minimize the objective function, subject to the satisfaction of the defined constraints. In this work, an Evolution Strategies algorithm was used in order to determine the design with the minimum cost for each scenario considered.

2.1. Objective Function

The objective function which was minimized is the total cost of the materials of the structural elements, which can be simply calculated as:

$$P_{tot} = P_C \cdot V_C + P_S \cdot V_S \tag{2.1}$$

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 general practice, for the estimation of the structural cost several factors need to be taken into account, such as the total labour cost, the availability of the materials on the market, the characteristics of the site, etc. In this work all these are assumed to be incorporated into costs P_C and P_S. In addition, all structural parts and details designed separately, such as the slabs, the beam-column connections, the bracings' connections and the foundations (including the column base connections) are excluded from the total cost calculation, but their contribution to the structural performance is taken into account in the structural simulation. Having taken all these into consideration, it becomes obvious that a reference to total cost in this work means the cost of the materials for beams, columns and bracings only. Furthermore, since for the beams and bracings only pure steel sections have been used, one can understand that the cost of concrete refers specifically to the columns.

One can notice that Eqn. 2.1 provides the total cost in monetary units, so its value is a mater of current prices used and doing so would render this work outdated once they have changed (e.g. when the prices of the materials, the currency exchange rate or the labour costs change). In order to avoid this, the ratio of concrete cost to steel cost was used in order to convert the total volume of concrete to equivalent steel volume. Finally, since the design engineers use the total weight of steel when it comes to calculate the cost of a steel structure the total equivalent steel volume is multiplied by the density of steel in order to provide the final value. So, the objective function minimized is:

$$\frac{W_{tot}}{\gamma_{\text{steel}}} = V_{tot} = CR \cdot V_C + V_S \tag{2.2}$$

where W_{tot} : the total material cost calculated in equivalent steel weight (tn of steel)
γ_{steel}: the density of steel (tn/m^3)
CR: 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

The use of an optimization algorithm in an engineering problem means that a large number of structural simulations needs to take place in order to evaluate each design according to the performance criteria used. So, in order to keep the total time needed for the whole optimization procedure within acceptable limits, one needs to reduce the number of design variables as much as possible. It was noticed in a previous work (Papavasileiou, Charmpis and Lagaros, 2011) that the division of the columns into four groups (see Section 3) was effective enough in limiting the computational effort, but still allowing the algorithm to reduce the cost significantly comparing to using one section for all columns.

It has to be noted here that the rest of the design parameters such as the total buildings' dimensions and the mechanical properties of the materials both on the column and the beam section were not altered during the optimization. Also, the concrete cover on the columns had been found to have a minimal effect to the total performance of the structure, so its value was also the same for all designs.

Summing up, there are 6 design variables in this problem: 4 variables for the steel sections of the columns, 1 for the beams and 1 for the bracings. Standard IPE sections and 6 HEB sections (HE600B to HE1000B) were used for the beams, while only HEB sections were used for the columns. For the bracings, specific L type sections were used.

2.3. Performance Criteria & Constraints

The performance criteria used in this work can be divided in two groups; the first group refers to the overall structural behaviour, while the second group consists of the individual member performance checks. The main difference between these two groups is that the overall performance criteria are the ones which define the number and type of analyses which need to take place for the evaluation of each design, while the individual member criteria need to be checked during every analysis.

2.3.1. Overall Performance Criteria

The aim of this work indicates that the structure needs to be checked both against seismic loads and accidental actions, which could lead to progressive collapse. From their nature, they are both actions with a probability of occurrence, however such an assessment would increase dramatically the computational time needed for an optimization, so deterministic criteria had to be used instead.

In order to take into account the effects of the seismic load, the targeted top displacement indicated by F.E.M.A. 440 (F.E.M.A., 2005) was used. Two displacement-controlled pushover analyses needed to take place (Spacone and El-Tawil, 2004), one for each direction of the horizontal displacement, up to the targeted top displacement. Then, the maximum inter-storey drift was calculated at each iteration.

$$\Delta_{target} = C_0 \cdot C_1 \cdot C_2 \cdot C_3 \cdot \frac{S_a}{\omega^2} \tag{2.1}$$

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
S_a: the design pseudo-acceleration defined for a S.D.O.F. system with fundamental period T
ω: the fundamental frequency of the structure (= $2\pi/T$)

The capacity of the structure to resist progressive collapse was estimated using a column removal scenario. A corner column at the base of the structure was removed and the structure was subjected to the combination of dead and live loads provided by EN1991 (C.E.N., 2003) for accidental loads. At each step, the total displacement of the node at the top of the removed column, as a percentage of the total beam length, was calculated as a damage indicator.

2.3.2. Individual Member Checks

The structural elements of the buildings were checked 3 times in each evaluation for the capacity criteria defined in the respective parts of the Eurocodes. Both the beams and the bracings were designed as pure steel sections, so they were checked according to the provisions of EN1993 (C.E.N., 2004) 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:

$$M_{Rd} = \frac{f_{yk} \cdot W_{el}}{\gamma_{M0}} \tag{2.2}$$

$$V_{Rd} = \frac{f_{yk} \cdot A_V}{\gamma_{M0}} \tag{2.3}$$

$$N_{c,Rd} = \frac{f_{yk} \cdot A_{tot}}{\gamma_{M0}} \tag{2.4}$$

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 the columns were designed as fully encased composite steel and concrete sections, so they were checked for all design actions mentioned for the beams, according to the EN1994 (C.E.N., 2004). Additionally, the axial shear force criterion, which is 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:

$$M_{Rd,tot} = M_{C,Rd} + M_{S,Rd} \tag{2.5}$$

$$V_{Rd,tot} = V_{C,Rd} + V_{S,Rd} \tag{2.6}$$

$$N_{Rd,tot} = N_{C,Rd} + N_{S,Rd} \tag{2.7}$$

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

In total three buildings of the same floor plan, but different height were simulated; a two-storey, a four-storey and a six-storey building of five spans per direction. The dimensions of the buildings were defined as 5.00m for each beam's span in both directions and 3.50m for each storey's height. Fig. 1 illustrates the floor layout used for all the simulated buildings, while Fig. 2 provides an indicative side view of the 3 buildings.

Beams, columns and bracings were simulated using fibber elements, which are considered to be suitable for this type of analyses, where brittle types of failure are not expected to occur. As mentioned previously, the columns were divided into 4 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.

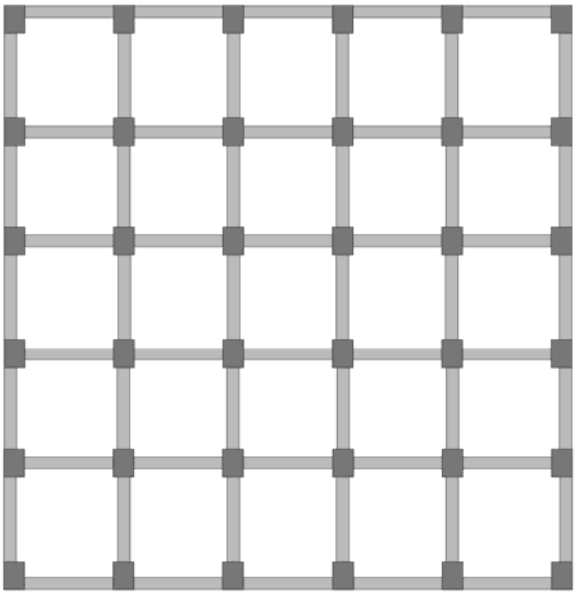

Figure 1. Common floor layout for all buildings

The contribution of the slabs was taken into account by distributing their load directly to the beams and defining a rigid diaphragm on each floor level. Also, all columns were considered to be fixed on their base. Finally, for the horizontal pushover analyses, the beam-column connections were considered to be either hinged, in order to take into account the maximum bending moment in the beam's span, or fixed, in order to result in the maximum transfer of moment to the columns. It is obvious that for the column removal scenario, a hinged connection would allow free movement of the beams, so the system would fail before even starting the application of the vertical loads to it, so only fixed connections were simulated for this category of analyses.

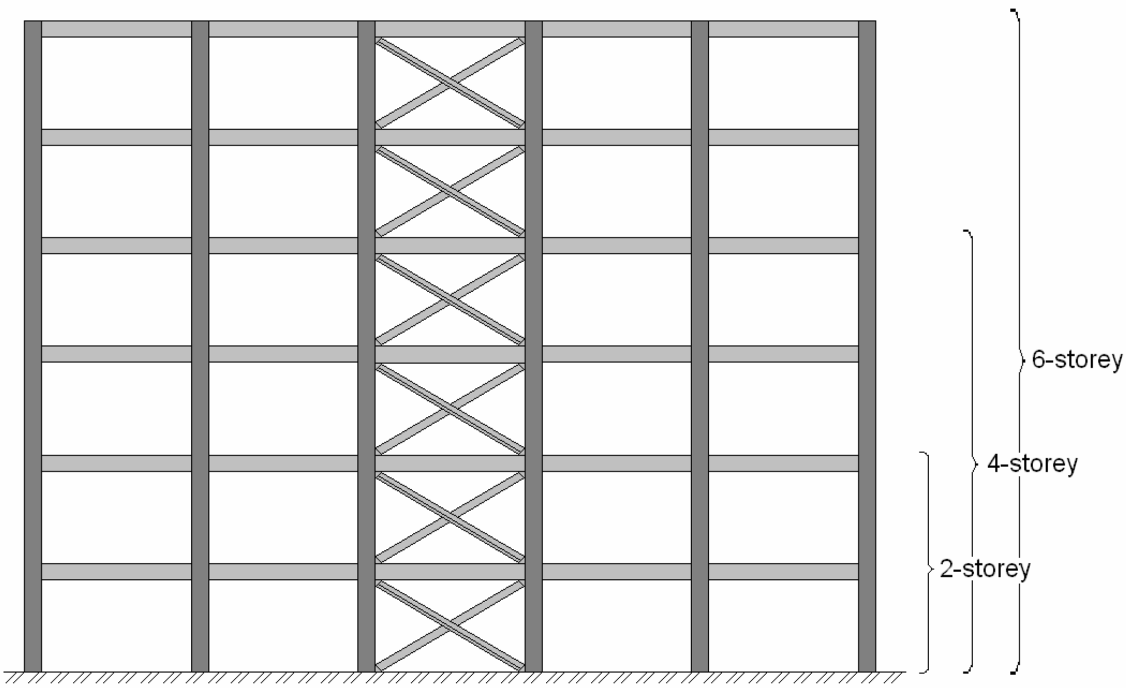

Figure 2. Vertical layout of the three buildings

4. RESULTS

Three similar buildings of different height were simulated, in order to investigate the difference of the behaviour related to the height of the structure. All of them were initially designed to be able to perform within the life safety limit state, for the targeted top displacement defined by F.E.M.A. 440 (F.E.M.A., 2005), without any requirements on progressive collapse resistance.

It has to be noticed here that the placement of the bracings in the middle span came as a result of the structural optimization procedure, since this allowed the optimization algorithm to provide more cost effective solutions than installing them in the corner spans, which would require twice as much the steel when the minimum section would be required. Additionally, the central placement of bracings also helps to avoid problems in the simulation when the column removal scenario is implemented. If the bracings were placed in touch with the column which would be removed, then they would have to be removed as well, since it's obvious that the accident (e.g. an explosion or a truck collision) which would damage the column wouldn't leave them unaffected. So, in that case, the scenario wouldn't be the loss of a column, but of the bracings as well.

For the purposes of this work, all the structures were designed for 4 values of the vertical displacement of the node at the top of the removed column: 0.50m, 0.25m, 0.10m and 0.05m, which correspond to the respective proportions of the beam's length, as a damage indicator: 5-10% for severe damage, 2% for extended damage and 1% for localized damage. The results for all buildings are presented in Fig. 3. Additionally, Table 4.1 presents the cost for each of these requirements normalized by the cost of the buildings when no requirements against progressive collapse exist.

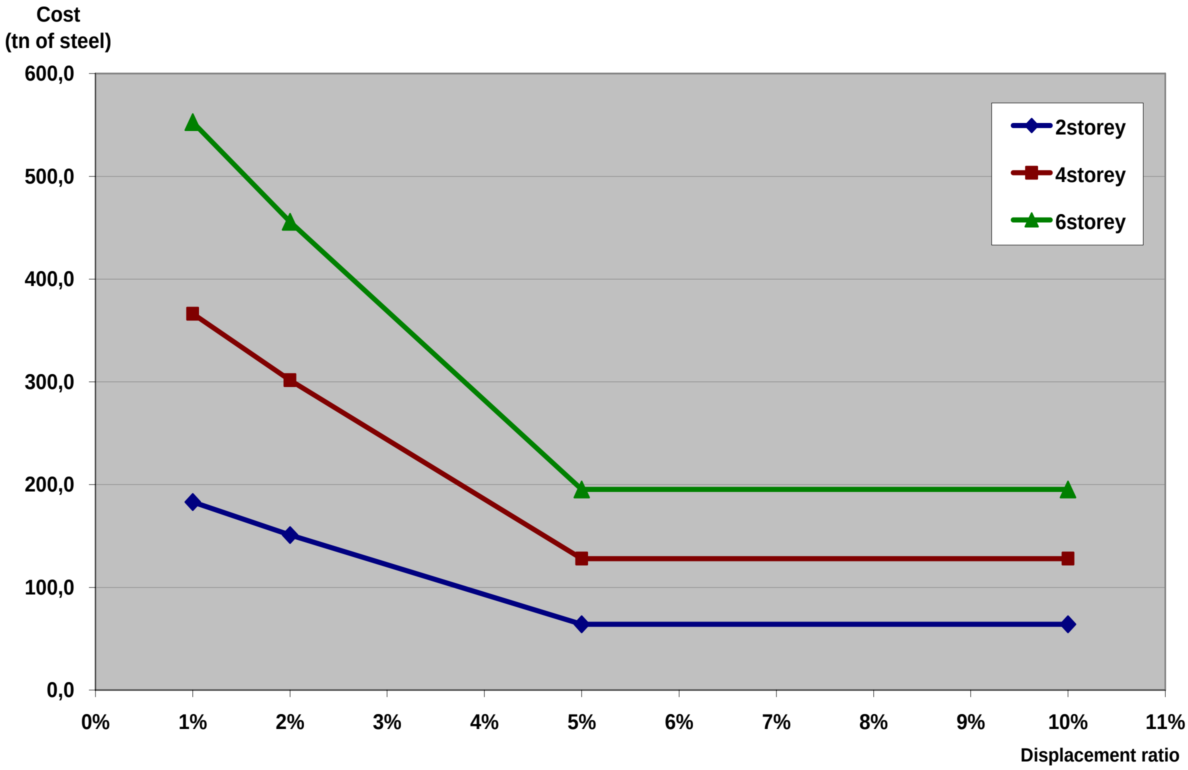

Figure 3. Total cost vs. vertical displacement

It's remarkable that the cost of the structures designed for maximum vertical displacement ratio 5% and 10% is the same as for the structures designed without such constraints. This is mainly the result of designing the beams as connected to the columns both hinged and fixed; this way, the beam sections need to have 50% increased bending moment capacity than if they were designed only as fixed, which was proved to be beneficial for the structure's behaviour, when the column removal scenario is implemented. At any case, it has to be noted that, if the beams to column connections were designed as hinges, then the removal of the column would result in the immediate collapse of this part of the system, so the fixed connection is what gives to the structure the required attributes to resist progressive collapse.

Table 4.1. Normalized cost per displacement level

Vertical displacement ratio	Number of storeys		
	2	**4**	**2**
1%	286,2%	286,2%	283,0%
2%	235,7%	235,7%	233,4%
5%	100,0%	100,0%	100,0%
10%	100,0%	100,0%	100,0%

Another important notice is that the normalized cost is the same for all heights of the buildings, at each level of vertical displacement ratio. The reason for that is the use of only one group of sections for the beams, so when there is need to increase a beam's capacity, all beam sections are increased. Of course, this also results in an even distribution of the damaged column's loads to the beams, which would be altered if more groups of design variables were assigned to the beams.

The same optimization series took place using two times the targeted top displacement requirement. The purpose of this second set of designs was to investigate any possible relationship between the design against horizontal loads and column removal scenarios. As expected, it was noticed that the increase in to the displacement capacity resulted in an increase of the total cost, which was the result of using larger sections for the columns or/and bracings, in order to increase the structure's total stiffness. However, this had no impact on the behaviour of the optimization algorithm, when the constraints on the vertical displacement were implemented. The results presented in Table 4.2 as the normalized cost are indicative of this remark.

Table 4.2. Normalized cost per displacement level ($2\times\Delta_{target}$)

Vertical displacement ratio	Number of storeys		
	2	**4**	**6**
1%	286,2%	286,2%	281,5%
2%	235,7%	235,7%	232,3%
5%	100,0%	100,0%	100,0%
10%	100,0%	100,0%	100,0%

An extra observation, which cannot be concluded by the results presented, is that during the optimization procedure, the columns of the feasible designs were found to have a significant overcapacity in axial force. This was happening because the column sections of the infeasible designs failed to meet the demand in bending moment, which was created by the total displacement of the building during the pushover analyses and resulted in the increase of their sections. It seems that this is the reason for which only the beams needed to be enhanced during the column removal scenario, despite the increased axial load transferred to the intact columns through the beams.

Last, but not least, the increase in the beams' sections results in beams which are stronger than the columns. In this work, since the failure type of the structure was not a part of the design, the capacity design was not taken into consideration, in order to keep the total cost as low as possible; however it is a significant problem which occurs when the structure is designed against progressive collapse and has to be taken into consideration.

5. CONCLUSIONS

The application of optimization methodologies to structural design (Fragiadakis, Lagaros and Papadrakakis, 2006; Fragiadakis and Papadrakakis, 2008) has proved to be a powerful tool, which allows engineers to determine solutions, under a variety of requirements, without excessive increase in the total cost. So, their application is expected to become necessary as the design codes become more demanding, in an effort to protect the structures against more dangers. Of course, the definition of the entire optimization problem, its design variables and constraints, still has to be made by the engineer, who can now take advantage of the available technology in order to face the upcoming challenges.

This work provided a first look at of the integration of this tool to the design procedures (i.e. against earthquake and progressive collapse) for composite buildings, showing this way its potentials and weaknesses. It is, however, an initial step, which indicates the areas where deeper investigation needs to take place.

The grouping of the columns which was used here was found to provide more cost effective designs than using the same section for all, keeping the required computational time within acceptable limits. Now, the need for further research with the assignment of more groups of design variables to the beams as well is emerging. Another interesting configuration, which would allow the algorithm to reduce the total cost even further, would be the division of the column and beam sections into more groups (e.g. per storey), but a significant increase in the total computational time is to be expected.

Since the implementation of column removal scenarios creates an extra demand in bending moment capacity to the beams and results in increase of their sections, special attention needs to be given to the capacity design. It is unavoidable that the total cost will be increased, because larger column sections will be required, but a better grouping, as mentioned above, could help limiting the extra cost.

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