Optimized Retrofit of Steel-Concrete Composite Buildings against Earthquake and Progressive Collapse Using Steel Cables

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Abstract – **This Master Thesis investigates the optimum retrofit approach for steel-concrete composite buildings in order to withstand progressive collapse. Apart from the building's collapse resistance, cables contribute to its stiffness, improving its performance against lateral loads as well. The investigation takes place in four distinct steps. Initially, buildings designed against earthquake are assessed under different damage scenarios. Following, the deficient buildings are manually retrofitted against specific scenarios using various steel cable configurations and the effectiveness of each configuration is evaluated. The same is realized automatically using an optimization algorithm, in order to determine the most cost-effective solution for each case. Finally, a guided optimization procedure is prepared which takes advantage of the remarks made in the previous steps by defining and imposing suitable penalty functions in the optimization problem. The results yielded are indicative of the effectiveness of the procedure.**

Keywords: steel-concrete composite, retrofit, optimization, steel cables, progressive collapse

I. Introduction

Progressive collapse is a topic of increased scientific interest for engineering researchers, mainly due to its destructive results, as well as the fact that it is large scale structural failure caused by small scale initial damage. Extreme actions, such as a strong earthquake, or an accident can cause severe damage to load bearing elements. Even though it seems as a problem that mainly high rise buildings face, due to the exploitation of the materials' capacity within the deformation range of their non-linear performance during structural design, the majority of newly designed buildings are susceptible to this type of failure.

Cables are used as a means to retrofit buildings, because of their advantages over their alternatives: they are not susceptible to flexural or lateral torsional buckling, as they receive only tensile forces. Also, their installation in existing buildings and their replacement in case of failure is easy thanks to the type of connections realized.

For the determination of the most cost-effective design, the "Evolution Strategies" optimization algorithm (Rechenberg, 1973) is employed, in order to enable the investigation of the overall design philosophy. An optimized design is the one which meets all applicable requirements and has the smallest cost at the same time, i.e. the design which achieves the optimum use of the materials.

Initially, an investigation of the effectiveness of various cable configurations in retrofitting steel-concrete composite buildings against progressive collapse takes place. Following, the optimization algorithm is used in order to determine the most cost-effective retrofit scheme. The optimized designs are evaluated against designs defined using manual retrofit strategies. The results yielded are indicative of the effectiveness of the retrofit approach.

II. Methodology

A. *Structural Simulation and Performance Criteria*

The simulations performed for the purposes of this work were performed using the OpenSEES software (Mazzoni *et al.*, 2006). Only three-dimensional models were used, as plane frames fail to capture torsional effects on the building due to stiffness and mass eccentricities. Additionally, the structural elements running in the direction vertical to the plane of the assessed frame have been found to have a beneficial effect which could even exceed the contribution of the elements of the frame, depending on the characteristics of the elements and the connections realized. Distributed plasticity elements (fiber elements) were used in order to model beams, columns, bracings and cables of the modelled buildings. All structural elements which are typically designed independently, such as composite slabs, beam-column connections and column bases were modelled taking into consideration their effect on structural behavior.

Steel elements were designed according to the provisions of EN 1993-1-1 (C.E.N., 2005) and steel-concrete composite members according to EN 1994-1-1 (C.E.N., 2004). Seismic design of buildings was performed using the provisions of FEMA-440 (F.E.M.A., 2005) and ASCE/SEI 41-06 (A.S.C.E., 2006) for typical building usage and type of soil. The guidelines of UFC 4-023-03 (D.O.D., 2005; D.O.D., 2010) were used for the assessment of the progressive collapse resistance of the simulated buildings.

Four analysis types were used for each building: (a) elastic analysis under gravitational loads, (b) eigenvalue analysis, (c) nonlinear static pushover analysis and (d) nonlinear static pushdown analysis. The elastic analysis under gravitational loads was performed in order to design structural members according to EN 1993-1-1. Eigenvalue analyses were performed in order to define the fundamental period of the buildings. Two displacement controlled pushover analyses, one in each horizontal direction, were performed in order to assess the performance of the buildings against seismic loads. A load pattern was defined and increased incrementally, until the control node at the top of the building reached the targeted top displacement (Δ_{target}) defined in FEMA-440. The maximum interstorey drift limit defined in ASCE/SEI 41-06 for steel buildings was used for all designs. Finally, one pushdown analysis per damage scenario considered was performed for each building. The criteria defined in UFC 4-023-03 for buildings with steel beams were selected for the steel-concrete composite buildings evaluated in this work.

B. Damage Scenarios Simulated

In this work, two types of accidents were considered: (a) the collision of a heavy loaded truck at the base of the building and (b) an explosion outside the building, but near its base. All events were considered to occur simultaneously. Hence, their effect can be simulated using suitable element removal scenarios and increasing the gravitational loads by a Dynamic Increase Factor. In the current literature (McKay *et al.*, 2012; Naji and Irani, 2012; Liu, 2013), as well as the available guidelines (G.S.A. 2003; D.O.D. 2005; D.O.D. 2010; G.S.A. 2013), a factor of 2.0 is considered to be adequate for the simulated events.

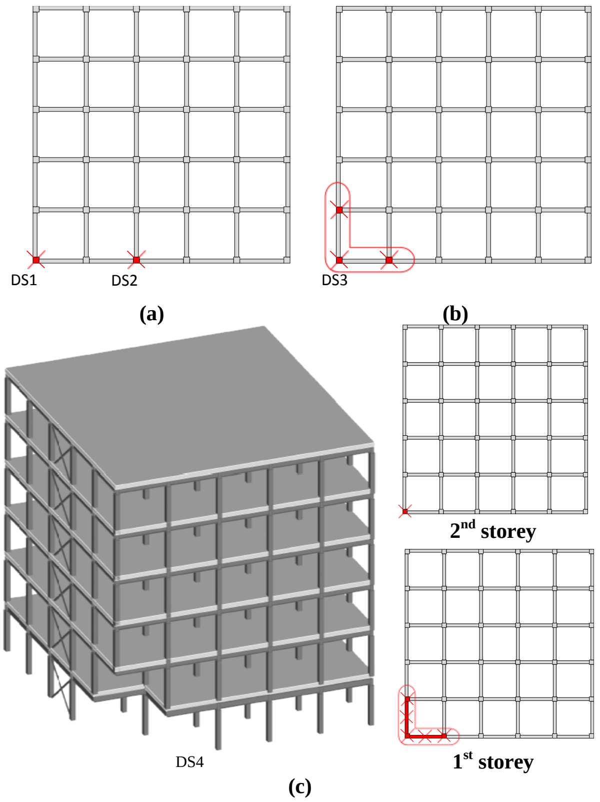

Figure 1. Simulated (a) single-column, (b) multi-column and (c) three-dimensional damage scenarios.

The selection of the Damage Scenarios took place based on (a) the location of the affected columns, (b) the number of elements on which the load of the failed member(s) is redistributed, (c) the total number of damaged elements and (d) the extent of the assumed initial damage. All affected structural elements are considered failed and are removed from the structural model. In particular, four Damage Scenarios were simulated, modelling the loss of: (a) a corner column, (b) a peripheral non-corner column, (c) multiple neighboring columns and (d) multiple adjacent beams and columns in three dimensions. Figure 1 illustrates the damage scenarios considered.

III. SELECTED RESULTS

Five buildings with steel-concrete composite columns and pure steel beams and bracings were initially designed against earthquake using the optimization procedure described in the work of Papavasileiou and Charmpis (2016). All buildings are six-storey buildings with 5 bays in each horizontal direction. The height of the first storey is 4.0m and 3.2m for the remaining storeys. Beam-columns connections are considered moment-restrained in x-direction and simple supports (hinges) in y-direction. Column bases are modelled as fixed supports.

The designs yielded are presented in Table I. The maximum recorded interstorey drift for each building is indicative of the use of an optimization algorithm in the design procedure, as it is particularly close to the limit value, *i.e.* 4%. This happens due to the trade-off between the section size, directly related to its stiffness and the total cost of the building, being a function of the section size, as its length is fixed.

TABLE I
BUILDINGS DESIGNED AGAINST EARTHQUAKE

Beam span (m)	Columns			Beams	Bracings	Maximum interstorey drift
	1st - 2nd storey	3rd - 4th storey	5th - 6th storey	All storeys	All storeys	
5m	HE280B	HE260B	HE180B	IPE200	L70x70x7	3,91%
6m	HE300B	HE280B	HE220B	IPE220	L90x90x10	3,83%
7m	HE360B	HE280B	HE240B	IPE270	L110x110x10	3,98%
8m	HE550B	HE320B	HE260B	IPE330	L150x150x10	3,95%
9m	HE600B	HE320B	HE280B	IPE550	L200x200X12	3,87%

A. *Manual retrofit using steel cables*

Initially, all buildings were retrofitted manually, installing steel cables in specific locations, which were defined based on the considered Damage Scenario. The common characteristic of the selected topologies is that both ends of the cable need to be on storeys over the location where the damage occurs. The end that is closer to the affected bay, does not necessarily have to be above a failed column, but it should be close enough so that the cables participate in the alternate load path developed, receiving a proportion of the loads from the damaged bay. The other end of the cable could be anywhere within a single bay, based on the intended function:

- In x- or y-directions parallel to the undamaged beams.
- In z- direction parallel to the columns above the affected bay.
- On x-z or y-z planes installed as bracings.
- On x-y plane, below the composite slab, installed as horizontal bracings.

The post-tensioning of the cables was also considered as an alternative with values ranging from 0% up to 80% of the yielding strength of the cables. Selected results are illustrated in Figure 2.

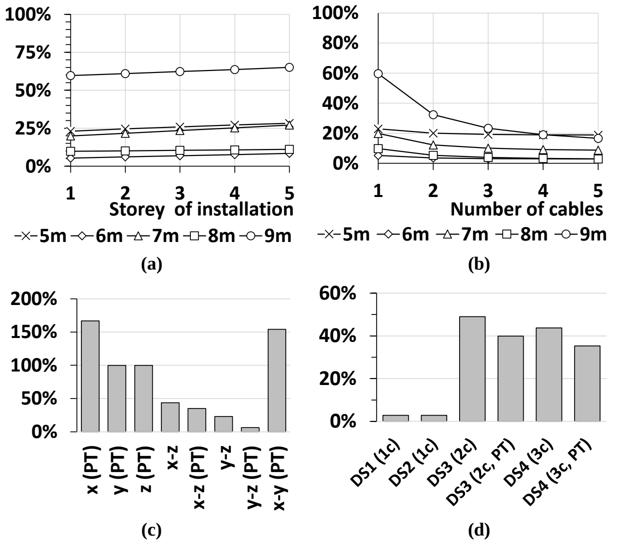

Figure 2. Assessment of manually determined retrofit configurations - normalized vertical drift in DS1 versus (a) the storey of installation, (b) the number of cables, (c) the cable direction and (d) for different Damage Scenarios (post-tensioning force is 500KN where applied).

The storey on which cables are installed affects their effectiveness (Fig. 2a). The reduction observed for top storeys compared to that when cables are installed directly above the damaged bay is insignificant. Hence, cables could be installed in higher storeys both for architectural purposes, and in order to allow the better protection against various Damage Scenarios, such as an explosion at a higher storey.

An increased number of cables aiming to protect against a single damage scenario is related to improved performance (Fig. 2b). However, the level of improvement is not proportional to the number of cables, while additional cables become ineffective after a certain number has been installed. The limit number of cables depends on the cable axial capacity, the building characteristics and the damage scenario simulated.

Installation of cables on y-z plane was found to be more beneficial than on the x-z plane (Fig. 2c). This is related to the type of beam-column connections modeled. Beams in x- direction are connected using moment restrained connections, forming moment resisting frames on x-z plane. Beams in y- direction are simply supported to the columns, so lateral resistance of the frames is achieved through the contribution of the individual columns and the installed bracings. Hence, when a column fails, a robust mechanism already exists on x-z plane in order to receive the additional loads, even if this might not suffice in order to restrict the deformations within the admissible limits. On y-z plane such a mechanism does not exist, as the columns above the damaged bay can move more freely, due to the hinges assumed between the beams and the columns. Installation of cables on this plane allows the formation of a truss-type mechanism which receives the loads from the failed elements. Even though its effect on x-z plane is also beneficial, the improvement on y-z plane is more evident.

Installation of cables in z-direction, does not seem to affect the plastic rotation developed at the beams after the loss of structural elements. The same applies when post-tensioning force is present, as the reduction observed is negligible. This is a strong indication that the columns' capacity in tension suffices in order to receive the loads from the failed columns below them, so the contribution of cables is particularly limited, as the ratio of a single cable's area over the column's is typically lower than 1%.

Cables installed in x- and y-directions and on x-y plane did not improve the performance of the buildings. Even though, when partial collapse occurs inside the building, all elements receiving loads in tension can contribute, the same does not apply when the collapse resistance of the building needs to be increased. The end of the beams that is above the location of a failed column does not move only vertically, but horizontally as well. The horizontal displacement of the beam's end is towards the interior of the building, which results in reduction of the cable's length, *i.e.* compressive deformations. Furthermore, when post-tensioning is applied on the cables, it was found to increase the deflection of the beam. The reason is that, as the lateral deformation of the beam is particularly increased, second order moments develop in the beam due to the horizontal component of the post-tensioning force. The same would not apply if the deflections were particularly reduced, e.g. applying larger post-tensioning forces, if possible. Increasing the flexural capacity of the beam using alternative methods would render the cables ineffective.

The effectiveness of each configuration is related to the Damage Scenario investigated (Fig. 2d). Damage Scenarios affecting multiple elements seem to be better retrofitted using multiple cables of small diameter, rather than few cables with high capacity, as it allows for better distribution of the structural robustness. Post-tensioning of cables installed on x-z and y-z planes seems to be beneficial against all Damage Scenarios.

B. *Automatic definition of the cable configuration using the Evolution Strategies optimization algorithm*

Following the assessment of manually determined configurations, an automatic approach is realized. In this approach, the Evolution Strategies optimization algorithm is employed in order to define the most cost-efficient configuration for each building retrofitted against the selected Damage Scenarios. Additionally, the results of the assessment are utilized in order to define penalty functions that would favour more efficient configurations and guide the optimization algorithm towards designs with increased inherit robustness, than designs that are adapted to the investigated Damage Scenarios.

In order to keep the number of variables at a minimum, but also allow the optimization algorithm to search numerous alternatives, particular restrictions were defined. Since the buildings are considered to be already constructed when the need to retrofit against progressive collapse occurs, cables cannot be installed internally on the x-z and y-z plane. In a different scenario, or when designing the buildings against earthquake and progressive collapse, this would not necessarily be a limitation. The maximum number of cables installed is 20, *i.e.* the number of bays per external side of the building multiplied by the number of faces available for installation, so that an enhanced peripheral zone around the building can be created if such a solution is more cost-effective than the other candidate optima. The diameter of cables could receive discrete values from 10mm up to 32mm. The upper limit could reach up to 75mm if high capacity wire strands are used. In this work such an alternative was not selected, as their installation would be particularly cumbersome, while torsional and flexural effects should also be taken into consideration for the wire strands. Finally, the cables should be installed symmetrically, so that stiffness irregularities that would affect the performance under seismic actions do not occur. Hence, the building is divided into four symmetrically designed quarters. The final number of variables for the optimization problems is 15 (location, diameter and post-tensioning force for 5 cables). Figure 3 illustrates the cable configuration defined for the 5m-beam-span building when retrofitted against DS1 (a) without penalty functions other than the ones defined for the optimization procedure already (OPT1) and (b) incorporating suitable penalty functions (OPT2).

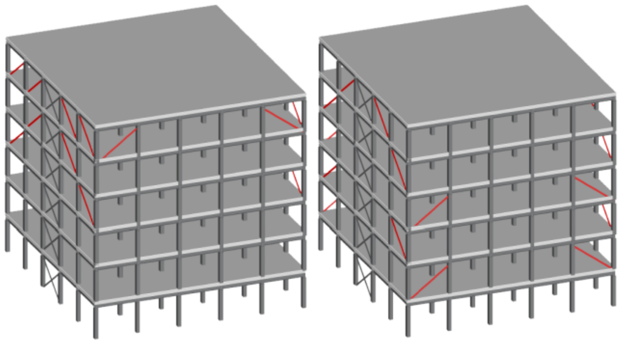
(a) (b)

Figure 3. Building retrofitted against progressive collapse for DS1 (a) without incorporating and (b) including suitable penalty functions in the optimization procedure.

Cables in OPT1 are installed in multiple storeys, not necessarily emphasizing on lower storeys, which are closer to the location of the initial damage. Two out of five cables are installed on x-z plane, even though installation on y-z plane was found to be more effective. The same applies on OPT2 where only 1 cable is installed on x-z plane. Further investigation revealed that their contribution is mainly against DS3, rather than DS1. In OPT2 a concentration of cables at the top storeys is observed, as intended by the penalty functions applied. Even though it would be favoured to install cables only in the top storey, cables were also installed in the top 3 out of 5 storeys possible. Additionally, 2 cables in OPT2 are installed continuously, forming a diagonal tie which receives loads from the corner of the building and transfers them to the center where a braced bay forms a much stronger core. The same mechanism is also observed in OPT1 one storey higher.

Comparison between the two designs is indicative of the effect of the penalty functions imposed. While in OPT1 the topology of the cables seems to be more random, in OPT2 the intended mechanisms are more apparent. Additionally, OPT2 could perform better in Damage Scenarios not considered in the analyses perform, as the alternate load path formed would not be affected by the initial damage. This alternate load path is achieved by the suspension of the loads from an enhanced zone at the top and transference to the braced bay core.

IV. Conclusions

This Thesis contributes to the scientific field of retrofit of structures, addressing the phenomenon of progressive collapse, which is an issue of increased scientific interest. Apart from the hazard addressed, the proposed approach is also innovative, as it combines an easily applicable retrofit method, *i.e.* steel cables, with a powerful computational tool, *i.e.* stochastic optimization algorithms.

The results obtained provide an insight to retrofit of structures using steel cables, as remarks of particular significance are made, such the problem occurring with post-tensioning of beams. Based on the optimized designs, a retrofit approach which utilizes the ability of steel to receive large tensile stresses, such as the suspension of hanging loads from the top storeys, seems to be the most cost-effective. It is a first but also important step for a more detailed investigation for a generic retrofit approach that utilizes the potential of traditional retrofit methods, as well as the multihazard design and retrofit of buildings.

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