

The Cost of Retrofitting Steel-Concrete Composite Buildings Against Progressive Collapse With Steel Cables

Georgios S. Papavasileiou

Department of Civil Engineering, Surveying and Geoinformatics Engineering,

Athens University of Applied Sciences, Greece

george.papav@gmail.com

Nikolaos G. Pnevmatikos

Department of Civil Engineering, Surveying and Geoinformatics Engineering,

Athens University of Applied Sciences, Greece



Abstract — Steel cables are an attractive means of retrofit with various engineering applications. They have been extensively used to strengthen deficient buildings against gravitational or earthquake-induced loads. This work investigates the use of steel cables as a means of retrofitting steel-concrete composite buildings against progressive collapse. The effect of the building's characteristics on the total retrofit cost is studied. A fair assessment of designs defined for different requirements is achieved by definition of the most cost-effective solution for each scenario. This is achieved by an optimization algorithm, i.e. the Evolution Strategies, which is employed to define the solution with the desired performance and, at the same time, the minimum cost. For this purpose, a total number of 144 optimizations have been performed. The results yielded reveal the different properties of each retrofit scenario.

Keywords — *Steel-Concrete Composite; Retrofit Methods; Progressive Collapse; Steel Cables; Optimization.*

I. INTRODUCTION

In the past decade, engineering researchers have focused on the topic of progressive collapse of structures. Although this issue has been pointed out and researched on since the collapse of the Ronan Point building [1], intensive investigation took place after the collapse of the World Trade Center towers in 2001. The term “progressive collapse” refers to the disproportionate propagation of structural damage within a structure, as the result of relatively minor initial damage. It is an undesirable failure mechanism mainly due to its destructive results, as well as the fact that the time required from the occurrence of the initial failure until it reaches its

full extent is particularly small. Extreme actions, such as a strong earthquake, or an accident can cause severe damage or even total failure to load-bearing elements, triggering the progressive collapse of the structure.

The phenomenon has been related mainly to high-rise buildings, as the results are more prominent, while the number of structural elements is adequate in order to allow for the observation of the damage propagation from the location of the initial damage to the neighboring elements. However, the same applies on low-rise buildings, where minor loss of structural integrity could cause partial collapse of the building, as there is limited number of structural elements which could receive the loads from the failed elements and redistribute

them within the structure. Hence, unless specific provisions have been taken into account during the design procedure, the majority of buildings constructed until today are susceptible to this type of failure.

Design against progressive collapse is not compulsory in current building codes. However, relative guidelines exist, such as UFC 4-023-03 [2] and the GSA guidelines [3], aiming to address the issue of progressive collapse. In the aforementioned guidelines, the general philosophy is described in order to increase the inherent robustness of the buildings, as well as criteria in order to assess their performance under various damage scenarios. The approach adopted in these guidelines is the development of ties within the building which constrain the relative displacement of structural elements in case of loss of particular members ([2], [3], [4]). This is achieved by elements which perform in tension after the initial damage, even though their intended load-bearing mechanism in the undamaged building might be significantly different.

Numerous retrofit methods have been proposed and evaluated by engineers in order to enhance the performance of buildings against earthquakes. The major approaches are (a) improvement of the structural performance by introduction of new members in the system ([5] to [23]) and (b) strengthening of individual existing members ([24] to [52]) which do not necessarily have to be deficient when evaluated against typical load combinations. However, literature on retrofit methods intended to enhance the progressive collapse resistance of existing structures is relatively limited ([53] to [60]).

Cables are extensively used by engineers in practice as a means to retrofit buildings, thanks to their advantages over the alternatives: they are not susceptible to flexural or lateral-torsional buckling, since they receive only tensile forces. Furthermore, they can be easily installed in existing buildings thanks to the type of connections used, while their replacement in case of failure is also straightforward. The existing literature is rich of papers which propose and evaluate analytical models for the simulation of the performance of cables and wire strands ([61] to [84]). However, their application as a retrofit method is based mainly on the engineer's previous experience and the intended performance of the strengthened building. A thorough investigation of the method's potential is required. In order to define the range of applicability and the efficiency of the method, the most cost-effective solution needs to be determined for each application. The designs yielded from an

optimization procedure are combinations which achieve the most efficient use of the building materials: they meet all requirements and have the minimum cost at the same time.

II. STRUCTURAL MODELLING

The OpenSEES software [85] was used for the purposes of structural simulation in this work. Only three-dimensional models were used, as plane frames fail to capture torsional effects on the buildings 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 on the collapse resistance of the building, which might exceed the contribution of the elements of the main frame, depending on the properties of the elements and the type of beam-column connections realized. The individual structural elements (i.e. beams, columns, bracings and cables) were discretized using distributed plasticity elements (fiber elements), which have been found to capture the post-elastic behavior of structural elements better than lumped plasticity elements, especially when large deflections develop. The structural elements which affect the behavior of the frames, but are typically designed independently, such as composite slabs, beam-column connections and column bases, were not explicitly defined in the structural model. Their contribution was indirectly simulated based on their effect on structural behavior. In particular, rigid diaphragms were defined at each storey to model the effect of composite slabs, as well as secondary beams in the performance of the building under horizontal loads. Moment-restrained connections were modelled as fixed connections, while simply supported beams were simulated using hinges at their ends. Column bases were considered to be either fixed or pinned, based on their moment resistance.

OpenSEES contains a library of numerous material models, suitable for the simulation of various materials and elements. In this work, the material models were selected based on their effectiveness on the simulation of the behavior of an actual structure, as well as their effect on the time required for the structural element to converge to a compatible force-displacement pair. Due to the application of structural design optimization, the analysis procedure is particularly time-demanding. Material models which need a large number of iterations until they converge in compatible force-displacements might result in a substantial increase to the required computational time without further improvement in the accuracy of the simulation. Additionally, the potential

of analysis failure if the desired accuracy is not achieved increases. Such a failure would result in considering a design infeasible, even though in practice it is able to receive all applied load safely. Hence, simple yet efficient material models were selected. Concrete in steel-concrete composite columns was simulated with the ‘Concrete01’ material. Unconfined concrete was distinguished from the confined concrete using reduced strength and ductility. A bilinear material model, i.e. ‘Steel01’ was used for the simulation of the steel core of the columns, as well as the beams and bracings. The longitudinal and transversal reinforcement of the columns was simulated using the ‘ReinforcingSteel’ Steel material. Cables were not modeled using a typical steel material, as the available options can receive both positive

and negative stresses, so they would receive compressive forces as well. Instead, the ‘ElasticPP’ model was used, which creates a material with a linear elastic branch, but considers zero post-elastic stiffness. The advantage of this material is the option to define the position where the curve reaches zero stress (or strain). So, using the appropriate stress-strain combination, the user can define an initial post-tension or sagging of the cable. To ensure that the results yielded do not overestimate the capacity of the steel materials, their strain was monitored, so that it would not exceed the 20% threshold set as the ultimate strain in this work. Fiber section definition and material model assignment for each structural element group are illustrated in Fig. 1.

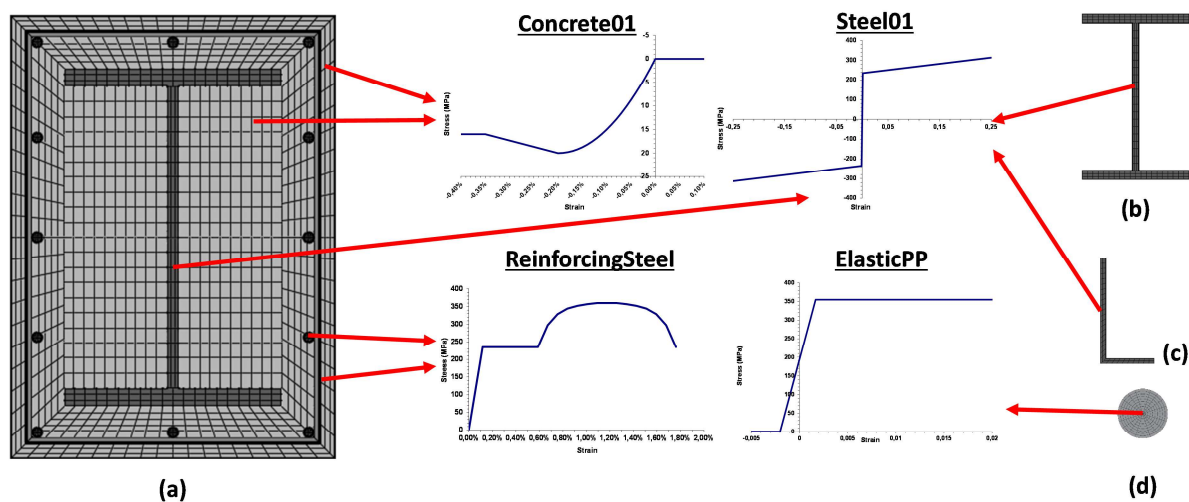


Figure 1. Structural element material modeling and discretization into fibers: (a) composite column, (b) steel beam, (c) bracing, (d) steel cable.

Four analysis types were used for each building: (a) an elastic analysis under gravitational loads, (b) two eigenvalue analyses, (c) two nonlinear static pushover analyses and (d) a nonlinear static pushdown analysis for each damage scenario considered. The elastic analysis under gravitational loads was performed in order to design structural members according to the provisions of EN 1993-1-1 [86] and EN 1994-1-1 [87]. 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 [88]. To ensure the desirable performance of the steel-concrete composite columns, the maximum interstorey drift limit defined in ASCE/SEI 41-06 [89] for reinforced concrete buildings was used for all designs, as it is more conservative than the one defined for steel buildings. Finally, one pushdown analysis per damage scenario considered was performed for each building. The criteria defined in UFC 4-023-03 [2] for buildings with steel beams were selected for the steel-concrete composite buildings evaluated in this work.

III. CONFIGURATION OF THE OPTIMIZATION PROBLEM

The determination of the most cost-effective design in this work is achieved using the “Evolution Strategies” optimization algorithm ([90], [91]), which imitates the evolution of a population in time. The particular algorithm was selected for (a) its stochastic search procedure and (b) its general applicability. The stochastic transition from one generation to the next, as well as the mechanisms which introduce randomly selected members to the population reduce significantly the probability of the algorithm being trapped near a local optimum. The algorithm was found to be effective in a wide variety of engineering problems, which makes it suitable for the purposes of this work. A concise description of the optimization procedure used is described in [92].

In the optimizations performed, the properties of the cables were set as independent variables, i.e. their values were selected by the optimization algorithm. The maximum number of cables was set equal to the number of bays of each building. The diameter of the cables was selected from a range from 20mm to 32mm using a step of 1mm. Cables with diameter up to 78mm could be used if available wire strands are taken into account. However, such elements have been found to develop bending moment and torsional effects, so a more accurate modelling than the one adopted in this work is required. The option of not installing a cable in a specific location was also available in the section database. It was found in a previous investigation [59] that the cables could be installed most cost-effectively on the x-z and y-z plane. Hence, the algorithm was restricted from installing cables on other directions, as this would be less efficient and, consequently, not an overall optimum.

The constraints of the optimization problem correspond to the provisions of the design codes imposed, regarding the capacity of individual structural members, as well as the overall performance of the building under the considered actions. In particular, all structural members should meet the requirements on their capacity in axial force, shear and bending moment, as well as their interaction where applicable. Additionally, provisions were considered against flexural, shear and lateral-torsional buckling, in addition to various forms of local buckling which might occur. Steel elements were designed according to the provisions of EN 1993-1-1 [83] and steel-concrete composite members according to EN 1994-1-1 [84]. Seismic design of buildings was performed using the

provisions of FEMA-440 [85] and ASCE/SEI 41-06 [86] for typical building usage and type of soil. The guidelines of UFC 4-023-03 [2] were used for the assessment of the progressive collapse resistance of the simulated buildings. The maximum plastic rotation at the end of the beams was used as an indicator of the building’s progressive collapse resistance: the smaller the plastic rotation is, the better the expected performance of the building is.

The objective function of the optimizations performed was defined as the overall cost of the retrofit procedure. The retrofit cost typically includes (a) the removal of existing partitions or external walls, in order to allow for the installation of the cables, (b) the material cost of the cable and the connections, (c) the realization of the connections and (d) the cost of restoration of the affected bay and the corresponding labor cost. Cost types (a) and (d) are the same for all cable diameters, so they are constant and should be removed from the objective function. Additionally, when a second cable is installed in a different bay, there is an abrupt increase in the total cost, as the minimum diameter is 20mm. So, this increase in the total cost is adequate to simulate the effect of cost types (a) and (d) on the selection procedure during the optimization. Furthermore, the cost of the connections is related to the maximum force they can receive which is a function of the cable area. Considering all aforementioned, the objective function can be defined as the total steel area multiplied by the length of the cables. In buildings where beam length is the same in x- and y-direction, while all storeys excluding the ground floor have the same height, the total cable length is a constant value. Taking into account that cables are installed in pairs, the objective function is:

$$C_{tot} = \sum_{i=1}^{n_{cables}} 2 \cdot [(\pi/4) \cdot D_i^2] \cdot l_i \quad (\text{Eqn . 1})$$

To ensure the desired performance of the optimization algorithm, the database needs to be properly set up. In the particular problem, this is straightforward: increased cable diameter results in corresponding increase in the objective function. However, as illustrated in Fig. 2, in various cases, the total cost of an increased number of cables with the minimum diameter is reduced compared to that of less cables of the maximum diameter, so a solution with multiple cables is found to be more attractive than one with a single strong cable. This is in accordance to the remark made in [59] that a better distribution of structural robustness should be favored when possible, as it is

expected to achieve improved structural performance in multiple damage scenarios. Hence, the objective function defined in Eqn. 1 is considered to be fit for the purposes of this investigation.

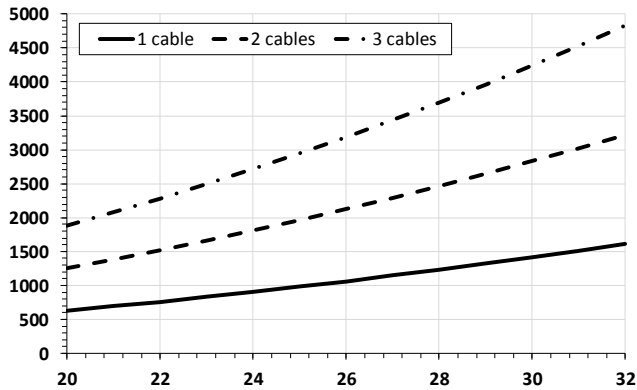


Figure 2. Database evaluation: cable diameter vs. total area.

IV. APPLICATION

Four damage scenarios (DS1 to DS4, illustrated in Fig. 3) were determined in order to simulate the result of various accidental actions. All scenarios consider the cause of the damage to be located on the ground floor, while its effect on the structural integrity might involve multiple storeys. DS1 simulates the effect of an accident typically taken into consideration when designing against progressive collapse, i.e. the collision of a heavy loaded

truck at a corner column on the building. The corner columns at the base of the building are the elements mainly exposed to such a hazard, while the number of neighboring elements on which the loads will be redistributed is relatively small (there are twice as many elements for an internal column). A similar scenario, i.e. loss of a peripheral column at the base of the building, is modeled in DS2. Another hazard associated with design against progressive collapse is an explosion originating either from the interior of the building, or from the surrounding area. Depending on the characteristics of the explosion (explosive substance, intensity of the explosion, proximity to the building, etc.), the effect might range from limited damage on a single element to immediate failure of multiple elements which leads to partial collapse of the building. Two scenarios considering the effect of an explosion near the corner of the building were considered in this work. DS3 simulates the explosion of a gas tank in at the corner bay of the ground floor. The explosion is considered to have damaged severely three columns which are considered failed and removed from the model, while the neighboring elements including the beams of the first floor were adequately protected to be considered undamaged. DS4 simulates the effect of a larger scale explosion which has damaged also the beams as well as the corner column of the first floor. Fig. 3 shows the simulated damage scenarios applied on a typical 5x5-bay building.

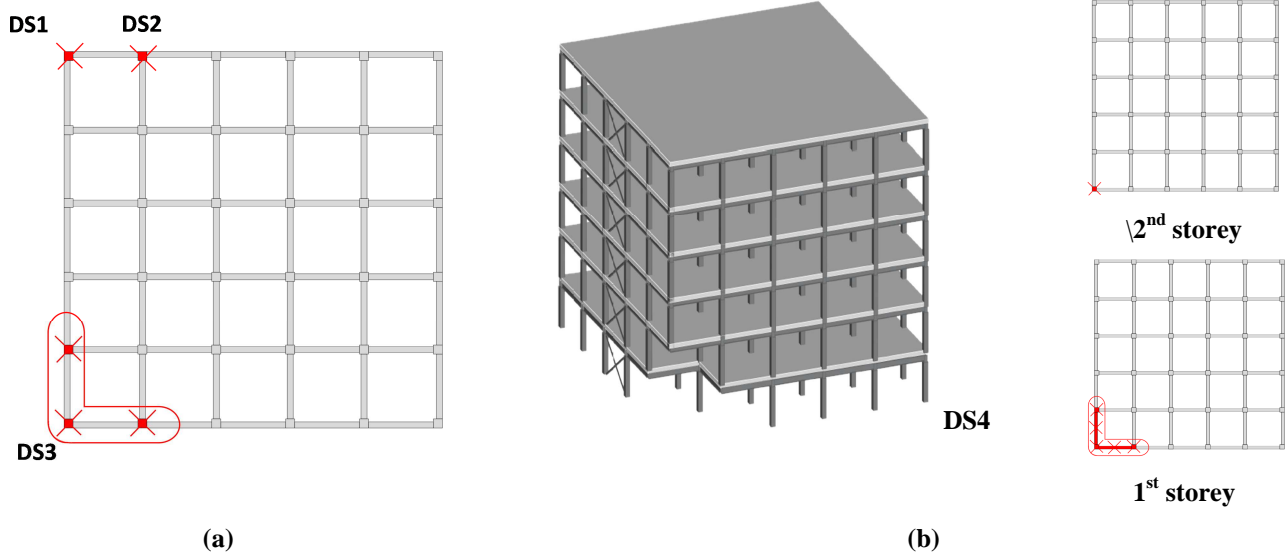


Figure 3. Simulated (a) single-column and multiple-column damage scenarios and (b) three-dimensional damage scenario.

Three buildings were initially optimized against earthquake as described in [92], considering typical soil

type and increased seismic hazard. The main difference between the buildings is the total beam span which ranges

from 5m up to 9m with a step of 2m. They consist of steel-concrete composite columns, steel beams and bracings installed externally in the middle bay of each face of the building. All buildings consist of five by five bays and nine storeys. Three beam groups were considered: one section group for three consequent storeys. Four groups were defined for the columns based on their location on the floor layout: (a) corner, (b) peripheral in x-direction, (c) peripheral in y-direction and (d) internal columns. Each group was further divided into three new groups taking into account the storey on which the columns are installed, defining a total number of twelve column groups. Two section groups were defined for the bracings, due to the limited number of elements, one group for each direction. All buildings were subjected to the damage scenarios illustrated in Fig. 3, in order to assess their progressive collapse resistance. To simulate the dynamic effect of the cause of the element loss, during the pushdown analysis the dead loads of the building are multiplied by a Dynamic Increase Factor ($DIF = 2.0$).

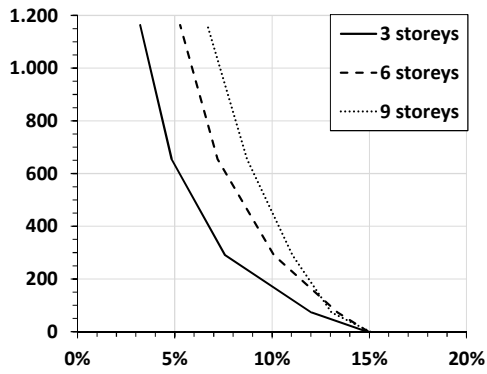
Two more buildings were defined for each beam span: a three-storey and a six-storey building, in order to investigate the effect of the number of storeys, resulting to nine buildings total. The additional buildings consist of the same sections as the respective elements at the top storeys of the nine-storey buildings. Hence, the six-storey building consists of column groups 5 to 12, beam groups 2 and 3 and bracing groups 1 and 2, while the three-storey building consists of column groups 9 to 12, beam group 3 and bracing groups 1 and 2. All buildings were retrofitted using steel cables. A topology optimization problem was defined: structural performance should be optimized using a given number of cables with standard diameters, which could be installed at any location in the building. Cables should be installed in pairs at each bay. Four cable diameters were defined from 5mm to 20mm with a 5mm step. A total number of 144 optimizations were performed. Selected results are illustrated in Figs. 4 to 7.

The Pareto-type cost-versus-rotation curves illustrate the performance of the building retrofitted with the most cost-effective cable layout at each case. The optimum solution for each case is defined by the damage scenario simulated and the desired level of improvement. In Figs. 4 to 7, cost is calculated in kilograms of steel. Typically, the slope of the curves increases for smaller maximum plastic rotations, as a result of the need to ensure that all beams

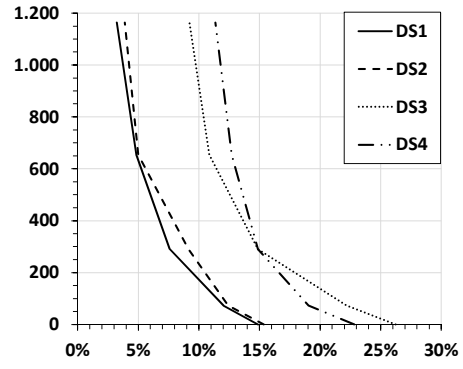
rotate up to that maximum, so multiple cables need to be increased or rearranged. In particular cases, this does not seem to apply. This is the effect of the uniform diameter used for all cables. The same maximum plastic rotation could be achieved using smaller cable diameters in particular cables if such an option was available. It should be noted that, while vertical drifts recorded reach values even higher than 50%, the total strain of the materials does not exceed the ultimate strain defined.

The curves shown in Fig. 5 indicate two damage-scenario pairs, i.e. DS1-DS2 and DS3-DS4, the results of which can be assessed against each other. Typically, structural behavior under DS2 is expected to be improved compared to DS1, as illustrated in Fig. 5(c), as the alternate load path utilizes the catenary action of the beams in x-direction. However, for beam spans 5m and 7m, the results defined for DS1 and DS2 are very close to each other, limiting the significant variance between the two cases to the 9m-beam-span building. Comparison between the results yielded for retrofit against DS3 and DS4 (Fig. 5) does not indicate which damage scenario can be addressed more cost-effectively. Depending on the building characteristics, DS3 might yield larger plastic rotations than DS4 or the opposite, while there is a case (i.e. the three-storey building with 5m beam span) in which initially DS4 is retrofitted with reduced cost over DS3, but this is reversed below a maximum plastic rotation of 15.2%. Nevertheless, the results yielded for DS3 and DS4 seem to be comparable, as the defined cost-versus-rotation curves are similar to each other, while in some cases they are also very close.

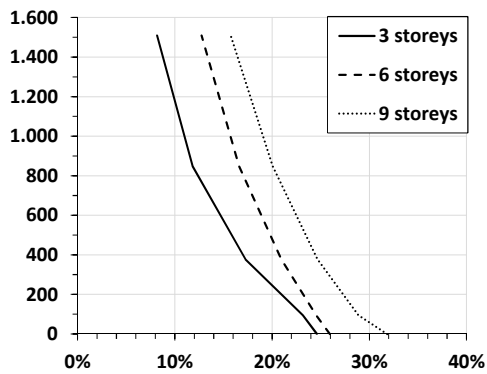
For large maximum plastic rotations developed in the non-retrofitted buildings, the level of improvement seems to be relatively higher. This can be partially expected, since it is infeasible to achieve very small or no rotation at the end of the beam, so all cost-versus-rotation curves approach the vertical axis asymptotically. The smaller the maximum plastic rotation developed in the non-retrofitted building is, the sooner the curve will reach the limit below which further improvement to structural behavior is particularly expensive. However, this limit is not the same for all buildings, as it seems to be related to various structural characteristics, such as the number of storeys and the beam span, as well as to the damage scenario investigated.



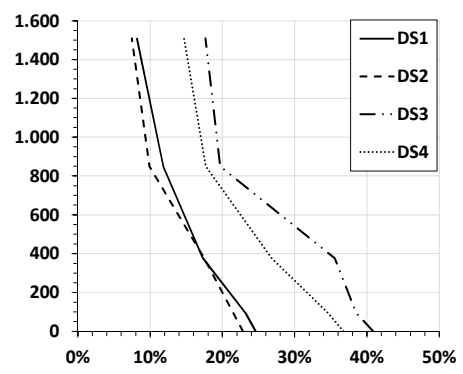
(a)



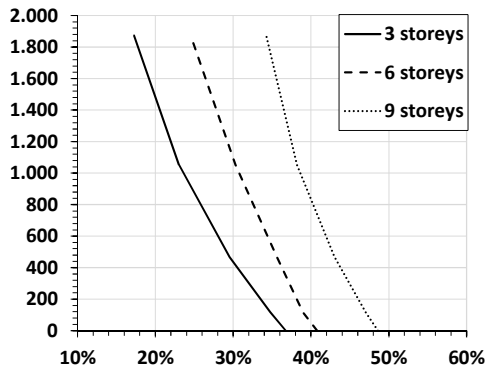
(a)



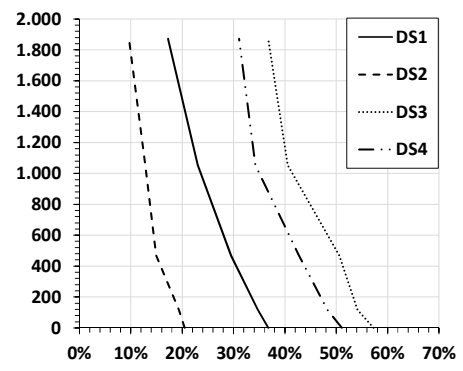
(b)



(b)



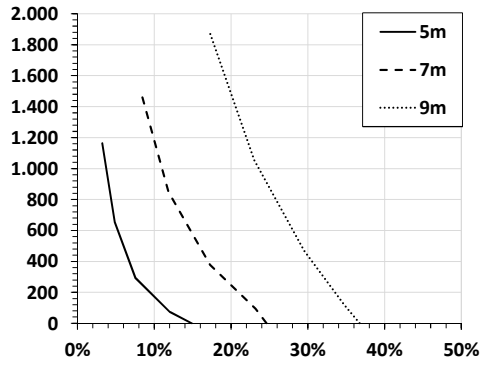
(c)



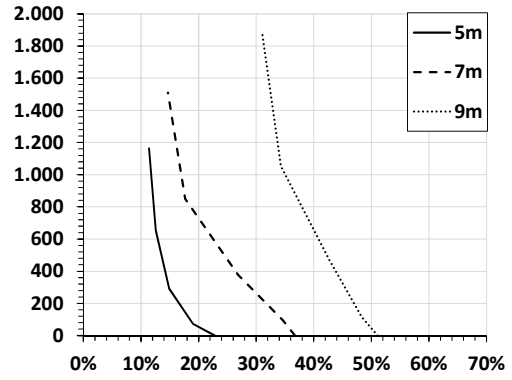
(c)

Figure 4. Total retrofit cost (in kg of steel) vs. maximum recorded plastic rotation under DS1; beam span = (a) 5m, (b) 7m, (c) 9m.

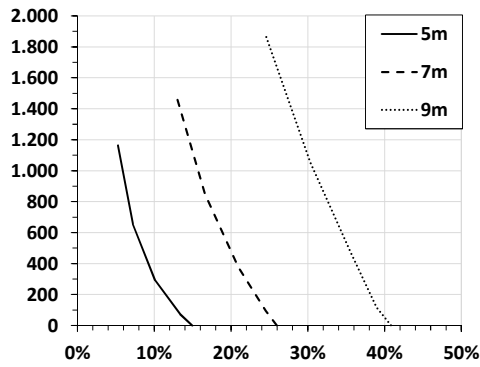
Figure 5. Total retrofit cost (in kg of steel) vs. maximum recorded plastic rotation – 3-storey building; beam span = (a) 5m, (b) 7m, (c) 9m.



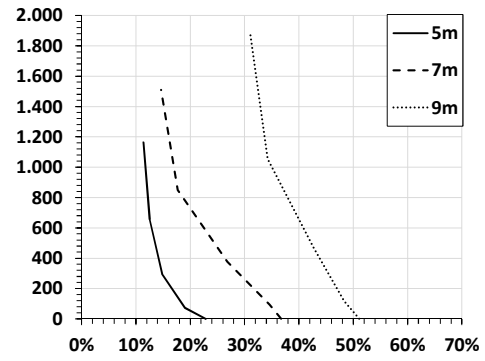
(a)



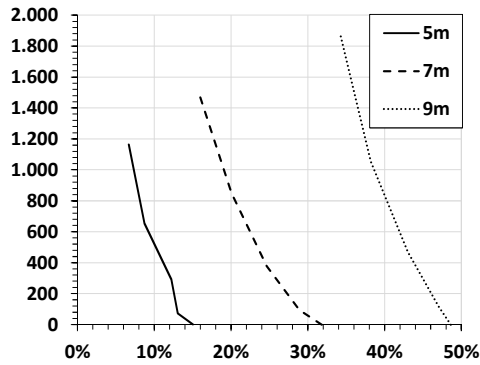
(a)



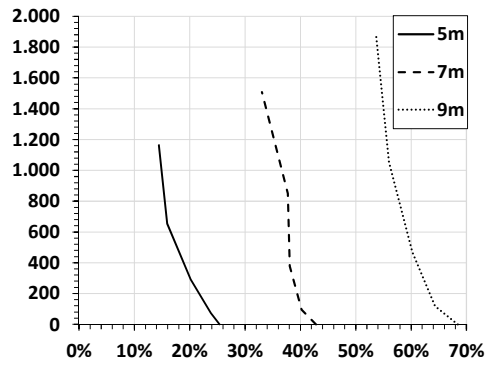
(b)



(b)



(c)



(c)

Figure 6. Total retrofit cost (in kg of steel) vs. maximum recorded plastic rotation under DSI – (a) 3-storey, (b) 6-storey, (c) 9-storey building; beam span = (a) 5m, (b) 7m, (c) 9m.

Figure 7. Total retrofit cost (in kg of steel) vs. maximum recorded plastic rotation under DS4 – (a) 3-storey, (b) 6-storey, (c) 9-storey building; beam span = (a) 5m, (b) 7m, (c) 9m.

When retrofitting is required in buildings where the plastic rotations that develop are small, additional retrofit techniques might be required. Such techniques are: (a) installation of cables without initial sagging or with post-tensioning [59], (b) installation of bracings [58], (c) beam strengthening techniques [60], etc. Particular attention is required during the design of a retrofit scheme including the aforementioned, as they affect the building's stiffness, so the retrofitted building's seismic performance needs to be re-evaluated.

Typically, buildings with increased number of storeys are expected to perform better under the loss of a load-bearing element, or multiple elements. This is because the number of structural elements above the location of the damage which participate in the alternate load path developed is increased. However, this does not necessarily apply on the total retrofit cost. Comparison between the optimized retrofits illustrated in Fig. 4 indicates a steeper decent of the cost-versus-rotation curve for the three-storey than for the six-storey buildings. The reason is that there is a limited number of bays above the location of the damage where cables can be installed. When they have all been occupied, the rest of the cables are installed in neighboring bays, so they are not as effective as the ones installed directly above the damaged bay(s) [59]. Hence, in the six-storey building, the cables can be used more cost-effectively. The opposite is observed comparing the results yielded for the six-storey and the nine-storey buildings. The cost-versus-rotation curves defined are either practically parallel, or a steeper decent occurs in the nine-storey building. This is related to the number of bays as well, but in this case, there is an excess of bays, so the number of cables that are installed does not suffice in order to cover all the available storeys, so the alternate path in some of these bays consists only of the beams and the respective columns.

Comparison between the maximum plastic rotations illustrated in Fig. 7 for the non-retrofitted buildings indicates increased deflections for larger beam spans. This is the effect of the change to the beam supports combined with the initial design parameters. In the undamaged building the beams are designed as moment restrained in x-direction so that together with the columns they form moment resisting frames and the section size is selected mainly based on the maximum moment in the mid-span of the beam. On the damaged building, the beams perform as cantilever so the bending moments developing are

significantly larger, while the relative deflection at the free end of the beam increases exponentially by its length.

V. CONCLUDING REMARKS

Unless their installation is disallowed, e.g. when particular architectural or usage limitations apply, steel cables offer an easily applicable retrofit solution for steel-concrete composite buildings, which provides a considerable level of improvement of the building's collapse-resistance with limited cost. A significant advantage of the method is the potential to define an initial sagging of the cables which is larger than the maximum interstorey drift expected to occur under seismic loads, so that the building's seismic response is not affected. Since deflections developing under structural damage are typically large, this feature does not hinder the solution from being effective. When the requirements against progressive collapse or the desired level of improvement are particularly high, alternative retrofit methods ([38], [60]) might need to be employed. Buildings with large bay spans are expected to develop significant deflections and, consequently, be particularly vulnerable to loss of load-bearing elements, unless the building is designed explicitly against progressive collapse. Extended initial damage does not necessarily lead to increased maximum plastic rotation, as a better redistribution of the unbalanced loads might be achieved. The same might not apply for damage caused intentionally, such as multiple columns destroyed on a single storey.

ACKNOWLEDGMENTS

The first author would like to gratefully acknowledge the financial support received by the State Scholarships Foundation within the framework of the "IKY Fellowships of Excellence for Postgraduate Studies in Greece – Siemens Programme" for post-doctoral research in Greece.

REFERENCES

- [1] C. Pearson and N. Dellatte. "Ronan point apartment tower collapse and its effects on building codes", *Journal of Performance of Constructed Facilities*, vol.19, pp. 172-177, 2005.
- [2] Department of Defense (DoD). *Unified Facilities Criteria (UFC) – Design of buildings to resist progressive collapse*, UFC 4-023-03, USA, 2009.

- [3] G.S.A. ‘Progressive Collapse Design Guidelines Applied to Concrete Moment-Resisting Frame Buildings’, General Services Administration, Nashville, Tennessee. 2004.
- [4] CO.S.T. TU0601 – Canisius T.D.G. (Editor). Structural Robustness Design for Practicing Engineers 2011.
- [5] J. Berman and M. Bruneau. “Plastic analysis and design of steel plate shear walls”.vol. 129 no. 11,pp. 1448-1456. 2003.
- [6] D. Charmpis, P. Komodromos and M. Phocas. Optimized earthquake response of multi-storey buildings with seismic isolation at various elevations. “Earthquake Engineering and Structural Dynamics (Wiley), vol 41 no. 15, pp.2289-2310. 2012.
- [7] M. Elgaaly. Thin steel plate shear walls behaviour and analysis. “Thin-Walled Structures”, vol.32 no.1-3, pp.151-180. 1998.
- [8] P. Javadi and T. Yamakawa. Retrofitting of RC frames by steel braced frames utilizing a hybrid connection technique.” *Journal of Advanced Concrete Technology*”, vol. 11 no. 3, pp.89-107. 2013.
- [9] Kelly, T. Base isolation of structures. Design Guidelines. Wellington: Holmes Consulting Group. 2001.
- [10] Y. Lu and G. Li. Slim buckling-restrained steel plate shear wall and simplified model. :*Advanced Steel Construction*”, vol. 3, pp. 282-294. 2012.
- [11] M. R. Maheri and H. Ghaffarzadeh. Connection overstrength in steel-braced RC frames. “*Engineering Structures*”, pp. 1938-1948. 2008.
- [12] M.R. Maheri, R. Kousari and M. Razazan. Pushover tests on steel X-braced and knee-braced RC frames. “*Engineering Structures*”, 25, 1697-1705. 2003.
- [13] A. Martelli and M. Forni. Seismic isolation and protection systems: Seismic isolation and other antiseismic systems. Recent applications in Italy and worldwide. “*The Journal of the Anti-Seismic Systems International Society (ASSISi)*”,vol. 1,no 1, pp. 75-123. 2010.
- [14] A. Massumi and M. Absalan. Interaction between bracing system and moment resisting frame in braced RC frames.” *Archives of Civil and Mechanical Engineering*” vol. 13, pp. 260-268. 2013.
- [15] J.P. Moehle. State of research on seismic retrofit of concrete building structures in the US. “US-Japan Symposium and Workshop on Seismic Retrofit of Concrete Structures—State of Research and Practice”. 2000.
- [16] Naeim, F. and Kelly, J. M. Design of seismic isolated structures. From theory to practice. California: John Wiley and Sons INC. 1999.
- [17] J. Nie, J. Fan, X. Liu and Y. Huang. Comparative study on steel plate shear walls used in a high-rise building. “*Journal of structural engineering*”, vol. 139 no 1, pp. 85-97. 2013.
- [18] R. Ozelik, B. Binici and O. Kurc. Pseudo dynamic test of a deficient reinforced concrete frame upgraded with internal steel frames.” *Earthquake Engineering and Structural Dynamics*” vol. 42, pp. 763-778. 2013.
- [19] Skinner, R. I., Robinson, W. H. and McVerry, G. H. An introduction to seismic isolation. West Sussex: John Wiley and Sons Ltd. 1993.
- [20] M. D. Symans, F. A. Charney, A. S. Whittaker, M. C. Constantinou, C. A. Kircher, M. W. Johnson, et al. Energy dissipation systems for seismic applications: current practice and recent developments. “*Journal of Structural Engineering*”, vol. 134 no. 1, pp. 3-21. 2008.
- [21] V. Varnava and P. Komodromos. Assessing the effect of inherent nonlinearities in the analysis and design of a low-rise base isolated steel building. “*Earthquakes and Structures*” vol, 5 no. 5. 2013.
- [22] Q. Zhao and A. Astaneh-Asl. Cyclic behaviour of traditional and innovative composite shear walls. “*Journal of Structural Engineering*”, vol. 130 no. 2, pp. 271-284. 2004.
- [23] X. Zou and C. Chan. “Optimal Drift Performance Design of Base Isolated Buildings Subject to Earthquake Loads”. In 6th International Conference on Computer Aided Optimum Design of Structures (pp. 369-378). Southampton, Boston: WIT Press. 2001.
- [24] M. Arduini, A. Ambrisi and A. Di Tommaso. Shear failure of concrete beams reinforced with FRP plates. In *Proceedings of the 3rd Materials Engineering Conference*, (pp. 123-130). San Diego, CA, USA. 1994.
- [25] S.M. Alcocer and J.). Jirsa. Strength of reinforced concrete frame connections rehabilitated by jacketing. *ACI “Structural Journal”*, vol. 90 no. 3 pp. , 249-261. 1993.
- [26] S. Bousias, D. Biskinis, M. Fardis and A-L. Spathis. Strength, stiffness and cyclic deformation capacity of concrete jacketed members. *ACI “Structural Journal”*, vol. 104 no. 5, pp. 521-531. 2007.
- [27] Y. H. Chai, J. N. Priestley and F. Seible. Analytical model for steel-jacketed RC circular bridge columns. “*Journal of Structural Engineering*”, vol. 120 no. 8, pp. 2358-2376. 1994.

- [28] M. J. Chajes, T. F. Januszka, D. R. Mertz, A. Theodore, J. Thomson, W. William and J. Finch. Shear strengthening of reinforced concrete beams using externally applied composite fabrics. *ACI "Structural Journal"*, vol. 92 no.9 , pp. 295-303. 1995.
- [29] J. F. Chen and J. G. Teng. Shear capacity of FRP-strengthened RC beams: FRP debonding. *"Construction and building materials"*, vol. 17 no. 1, pp.27-41. 2003.
- [30] E. Choi, Y.-S. Chung, K. Park and J.-S. Jeon. Effect of steel wrapping jackets on the bond strength of concrete and the lateral performance of circular RC columns. *"Engineering Structures"*, vol. 48, pp. 43-54. 2013.
- [31] E. Choi, I. Rhee, J. Park and B. S. Cho. Seismic retrofit of plain concrete piers of railroad bridges using composite of FRP-steel plates. *"Composite Part B: Engineering"*, vol. 42 no. 5, pp.1097 - 1107. 2011.
- [32] E. Julio, F. Branco and V. Silva. Reinforced concrete jacketing - Interface influence on monotonic loading response. *ACI "Structural Journal"*, vol. 102 no.2,pp. 252 - 257. 2005.
- [33] K. Karimi, M. J, Tait and W, W. El-Dakhakhni. Testing and modeling of novel FRP-encased steel-concrete composite column. *"Composite Structures"*, vol. 93, pp. 1463-1473. 2011.
- [34] A.P. Lampropoulos and S.E. Dritsos. Modeling of RC columns strengthened with RC jackets. *"Earthquake Engineering and Structural Dynamics"*, vol 40, pp. 1689-1705. 2011.
- [35] M.-L. Lin, P.-C. Chen, K.-C. Tsai, Y.-J. Yu and J.-G. Liu. Seismic steel jacketing of rectangular RC bridge columns for the mitigation of lap-splice failures. *"Earthquake Engineering and Structural Dynamics"*, vol. 39, pp. 1687-1710. 2010.
- [36] Y. Mitsui, K. Murakami, K. Takeda and H. Sakai. A study on shear reinforcement of reinforced concrete beams externally bonded with carbon fiber sheets." *Composite Interfaces"*, vol. 5 no. 4, pp. 285-295. 1998.
- [37] V. K. Papanikolaou, S.P. Stefanidou and A.J. Kappos. The effect of preloading on the strength of jacketed R/C columns. *"Construction and Building Materials "* vol 38, pp. 54-63. 2013.
- [38] Papavasileiou, G. S., Charmpis, D. C. and Lagaros, N. D. (2011). Optimized seismic retrofit of steel-concrete composite frames. In *Proceedings of Third ECCOMAS Thematic Conference on Computational Methods in Structural Dynamics and Earthquake Engineering (Vol. 1, pp. 4573-4586)*.
- [39] X. S. Ren and B. Zhou. Design and analysis of reinforced concrete beam retrofitted by externally bonded H-type steel member. *"Procedia Engineering"*, vol. 14, pp. 2133-2140. 2011.
- [40] S. Rizkalla, M. Dawood and D. Schnerch. Development of carbon fiber reinforced polymer system for strengthening steel structures. *"Composites - Part A: Applied Sciences and Manufacturing"*, vol. 39, pp. 388-397. 2008.
- [41] M. P. Rodriguez. Seismic load tests on reinforced concrete columns strengthened by jacketing. *ACI "Structural Journal"*, vol. 91 no. 2, pp. 150-159. 1994.
- [42] R. Seracino, M. Raizal Saifulnaz and D. Oehlers. Generic debonding resistance of EB and NSM plate-to-concrete joints. *"Journal of Composites for Construction"*, vol. 11 no. 1, pp. 62-70. 2007.
- [43] R. Su, B. Cheng, L. Wang, W. Siu and Y. Zhu. Use of bolted steel plates for strengthening of reinforced concrete beams and columns. *IES journal Part A: "Civil and Structural Engineering"*, vol. 4 no. 2, pp. 55-68. 2011.
- [44] S. P. Tastani and S. J. Pantazopoulou. Detailing procedures for seismic rehabilitation of reinforced concrete members with fiber reinforced polymers. *"Engineering Structures"*, vol. 30 no. 2, pp. 450-461. 2008.
- [45] S. P. Tastani, S. J. Pantazopoulou, D. Zdoumba, V. Plakantaras and E. Akritidis. Limitations of FRP jacketing in confining old-type reinforced concrete members in axial compression. *"Journal of Composites for Construction"*, vol. 10 no. 1, pp. 13-25. 2006.
- [46] M. Tavakkolizadeh and H. Saadatmanesh. Strengthening of steel-concrete composite girders using carbon fiber reinforced polymers sheets. *"Journal of Structural Engineering"*, vol. 129 no. 1, pp. 30-40. 2003.
- [47] G. E. Thermou, S. J. Pantazopoulou and A. S. Elnashai. Flexural behavior of brittle RC members rehabilitated with concrete jacketing. *"Journal of Structural Engineering"*, vol. 133 no 10, pp. 1373-1384. 2007.
- [48] G. E. Thermou, S. J. Pantazopoulou and A. S. Elnashai. Analytical modeling of interface behavior in reinforced concrete jacketed members. *ASCE*. 2004.
- [49] T. Triantafillou and C. Antonopoulos. Design of concrete flexural members strengthened in shear with FRP. *"Journal of Composites for Construction"*, vol. 4 no. 4, pp. 198-205. 2000.
- [50] K. G. Vadoros and S. E. Dritsos. Concrete jacket construction detail effectiveness when strengthening RC

columns. "Construction and Building Materials", vol. 22 no. 3, pp. 264-276. 2008.

- [51] Y.-F. Wu, T. Liu and D. J. Oehlers. Fundamental principles that govern retrofitting of reinforced concrete columns by steel and FRP jacketing. "Advances in Structural Engineering", vol. 4 no. 9, pp. 507-533. 2006.
- [52] Y. Xiao and H. Wu. Retrofit of reinforced concrete columns using partially stiffened steel jackets. "Journal of Structural Engineering". Vol 129, pp. 725-732. 2003.
- [53] J.E. Crawford. "Retrofit methods to mitigate progressive collapse". In The Multihazard Mitigation Council of the National Institute of Building Sciences, Report on the July 2002 National Workshop and Recommendations for Future Effort. 2002.
- [54] A. Astanesh-Asl. "Progressive collapse prevention in new and existing buildings". In Proceedings of the 9th Arab Structural Engineering Conference, Abu Dhabi, UAE. 2003.
- [55] A. Saad, A. Said and Y. Tian. "Overview of progressive collapse analysis and retrofit techniques". In Proceedings of the 5th International Engineering and Construction Conference (IECC'5). ASCE. 2008.
- [56] K. Galal and T. El-Sawy. "Effect of retrofit strategies on mitigating progressive collapse of steel frame structures." Journal of Constructional Steel Research, vol. 66, pp. 520-531. 2010.
- [57] J. Kin, H. Choi and K.W. Min. "Use of rotational friction dampers to enhance seismic and progressive collapse resisting capacity of structures". The Structural Design of Tall and Special Buildings. Vol. 20, pp. 515-537. 2011.
- [58] G.S. Papavasileiou and D.C. Charmpis. "Optimized retrofit of seismically designed buildings to withstand progressive collapse". In Proceedings of the 5th International Conference on Computational Methods in Structural Dynamics and Earthquake Engineering, 2015.
- [59] G.S. Papavasileiou. "Assessment of the effectiveness of cabling system configuration in retrofitting steel-concrete composite buildings". In Proceedings of the 6th International Conference on Computational Methods in Structural Dynamics and Earthquake Engineering, 2017.
- [60] G.S. Papavasileiou and D.C. Charmpis. "Retrofit of seismically designed steel-concrete composite structures to withstand progressive collapse". In Proceedings of the 2nd International Conference on Recent Advances in Nonlinear Modelling – Design and Rehabilitation of Structures. 2017.
- [61] J.F. Beltran, J. Rungamornrat and E.B. Williamson. "Computational model for the analysis of damaged ropes".

In The Thirteenth International Offshore and Polar Engineering Conference. International Society of Offshore and Polar Engineers. 2003.

- [62] J. F. Beltran and E.B. Williamson. "Investigation of the damage-dependent response of mooring ropes". In The Fourteenth International Offshore and Polar Engineering Conference. International Society of Offshore and Polar Engineers. 2004.
- [63] J. F. Beltran and E. B. Williamson. Investigation of the damage-dependent response of mooring ropes. "Journal of engineering mechanics", vol 135 no. 11, pp. 1237-1247. 2009.
- [64] G. A. Costello and J. W. Phillips. Effective modulus of twisted wire cables. "Journal of the Engineering Mechanics Division", vol. 102 no. 1, pp. 171-181. 1976.
- [65] G. A. Costello. Theory of wire rope. Springer Science and Business Media. 1997.
- [66] D. Elata, R. Eshkenazy and M. P. Weiss. The mechanical behavior of a wire rope with an independent wire rope core. "International Journal of Solids and Structures", vol. 41 no. 5, pp. 1157-1172. 2004.
- [67] S. R. Ghoreishi, P. Cartraud, P. Davies and T. Messenger. Analytical modeling of synthetic fiber ropes subjected to axial loads. Part I: A new continuum model for multilayered fibrous structures. "International Journal of Solids and Structures", vol. 44 no. 9, pp. 2924-2942. 2007.
- [68] F. H. Hruska. Calculation of stresses in wire ropes. "Wire and wire products", vol. 26, pp. 766-767. 1951.
- [69] F. H. Hruska. Radial forces in wire ropes. "Wire and wire products", vol. 27 no. 5, pp. 459-463. 1952.
- [70] F. H. Hruska. Tangential forces in wire ropes. "Wire and wire products", vol. 28 no. 5, pp. 455-460. 1953.
- [71] N. C. Huang. Finite extension of an elastic strand with a central core. "ASME J. Appl. Mech", vol. 45 no. 4, pp. 852-858. 1978.
- [72] R. H. Knapp. Nonlinear analysis of a helically armored cable with nonuniform mechanical properties in tension and torsion. In OCEAN 75 Conference (pp. 155-164). IEEE. 1975.
- [73] R. H. Knapp. Derivation of a new stiffness matrix for helically armoured cables considering tension and torsion. "International Journal for Numerical Methods in Engineering", vol. 14 no. 4, pp. 515-529. 1979.
- [74] K. Kumar J. Botsis Contact stresses in multilayered strands under tension and torsion. "Journal of Applied Mechanics" vol. 68:pp.432–440. 2001.

- [75] K. Kumar and Jr. J. E. Cochran. Closed-form analysis for elastic deformations of multilayered strands. “Journal of Applied Mechanics”, vol. 54, pp. 899. 1987.
- [76] C. M. Leech, J.W. Hearle, M.S. Overington and S.J. Banfield. “Modelling tension and torque properties of fibre ropes and splices”. In The Third International Offshore and Polar Engineering Conference. International Society of Offshore and Polar Engineers. 1993.
- [77] S. Machida and A. J. Durelli. Response of a strand to axial and torsional displacements. “Journal of Mechanical Engineering Science”, vol. 15 no. 4, pp. 241-251. 1973.
- [78] A. Nawrocki and M. Labrosse. A finite element model for simple straight wire rope strands. “Computers and Structures”, vol. 77 no. 4, pp. 345-359. 2000.
- [79] J.W. Philips and G.A. Costello. Analysis of wire rope with internal-wirerope cores. ASME “Journal of Applied Mechanics”, vol. 52, pp. 510–6. 1985
- [80] J. Rungamornrat, J. F. Beltran and E. B. Williamson. Computational model for synthetic-fiber rope response. In 15th Eng. “Mechanics Conference, ASCE”, New York, USA. 2002.
- [81] S. Sathikh, M. B. K. Moorthy and M. Krishnan. A symmetric linear elastic model for helical wire strands under axisymmetric loads. “The Journal of Strain Analysis for Engineering Design”, vol. 31 no. 5, pp. 389-399. 1996.
- [82] W. S. Utting and N. Jones. The response of wire rope strands to axial tensile loads—Part II. Comparison of experimental results and theoretical predictions. “International journal of mechanical sciences”, vol. 29 no. 9, pp. 621-636. 1987.
- [83] W.S. Utting and N. Jones. The response of wire rope strands to axial tensile loads—Part I. Experimental results and theoretical predictions. “International journal of mechanical sciences”, vol. 29 no. 9, pp. 605-619. 1987.
- [84] S. A. Velinsky. General nonlinear theory for complex wire rope. “International journal of mechanical sciences”, vol. 27 no. 7, pp. 497-507. 1985.
- [85] S. Mazzoni, F. McKenna, M. Scott and G.L. Fenves. “Open System for Earthquake Engineering Simulation (OpenSees)”, PEER Center, California, USA. 2006.
- [86] EN 1993-1-1. “Eurocode 3: Design of steel structures – Part 1-1: General rules and rules for buildings”, Brussels, Belgium: CEN. 2005.
- [87] EN 1994-1-1. “Eurocode 4: Design of composite steel and concrete structures – Part 1-1: General rules and rules for buildings”, Brussels, Belgium: CEN. 2004.
- [88] Federal Emergency Management Agency (FEMA). “Improvement of nonlinear static seismic analysis procedures, FEMA-440”, Washington DC, USA, 2005.
- [89] American Society of Civil Engineers (ASCE). “Seismic rehabilitation of existing buildings, Standard ASCE/SEI 41-06 (incl. suppl. 1)”, Reston, Virginia, USA. 2006.
- [90] Rechenberg, I. “Evolutionsstrategie: Optimierung technischer systeme nach prinzipien der biologischen evolution”. Stuttgart, Germany: Frommann-Holzboog Verlag. 1973.
- [91] H.P. Schwefel. “Evolutionsstrategie und numerische Optimierung”. Berlin : (Doctoral dissertation, Technische Universität Berlin). 1975.
- [92] G.S. Papavasileiou and D.C. Charnpis. “Seismic design optimization of multi-storey steel-concrete composite buildings”, Comput. Struct., vol. 170, pp. 49-61. 2016.