Optimized design of steel buildings against earthquake and progressive collapse using cables

PAPAVASILEIOU, Georgios and PNEVMATIKOS, Nikos

Published version

PAPAVASILEIOU, Georgios and PNEVMATIKOS, Nikos (2017). Optimized design of steel buildings against earthquake and progressive collapse using cables. International Journal of Progressive Sciences and Technologies, 6 (1), 213-220.

International Journal of Progressive Sciences and Technologies (IJPSAT)
ISSN: 2509-0119.

http://ijpsat.ijsht-journals.org
Vol. 6 No. 1 December 2017, pp. 213-220

Optimized Design of Steel Buildings Against Earthquake and Progressive Collapse Using 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 **– Progressive collapse is a procedure in which local failure of a structural component can cause failure of the overall structure or a smaller part of it. This phenomenon is the subject of intensive investigation by researchers the last decade. This work presents a design of structures against earthquake and progressive collapse. Cables are used as means to achieve the desired structural performance when the buildings are subjected to (a) seismic excitations, (b) accidents which result in failure of structural members. The design strategy is based on the use of cables located in suitable locations in the structure. The element sizes and cable topology are attained by an automatic optimization procedure in an effort to achieve the most effective use of structural materials. The effect of various design constraints is evaluated in the performance of the optimized buildings. The analysis results indicate the promising potential of cables as a means to increase the building's progressive collapse resistance, as well as a promising alternative to typical bracing sections used in practice.**

Keywords – Steel Structur; Earthquake; Progressive Collapse; Steel Cables; Optimization.

I. INTRODUCTION

Progressive collapse, of structures has attracted the attention of researchers mainly during the past decade. The collapse of the World Trade Center (WTC) in New York brought to light a phenomenon which had already been indicated almost 40 years earlier, after the collapse of the Ronan Point apartment building [1]. Researchers have since worked on (a) the definition of the collapse mechanism of buildings, (b) measurement of the overall robustness of buildings and (c) design of buildings against progressive collapse.

The term "progressive collapse" refers to the disproportionate propagation of structural damage within a structure, resulting from relatively minor initial damage. It begins from the initial failure of structural element and very fast expands to the entire structure or a large part of it. This type of failure is treated as a "brittle" failure, as the time from the occurrence of the initial failure, until neighboring elements are damaged is particularly small. Additionally, the results are devastating if local or global collapse occurs. Hence, it is an undesirable failure mechanism which needs to be prevented. Progressive collapse can have caused by extreme actions, such as a strong earthquake, blast load or impact loading. Such an action cause initially a failure of small number of structural elements and this can be triggering the progressive collapse of the entire structure.

Design building codes implemented after 2000 give directions against progressive collapse design. In the Eurocodes, such guidelines can be found in EN 1992-1 [2] which proposes the increase of the overall robustness of structures which are not designed explicitly to withstand accidental actions by a suitable tying system which provides alternative load paths after local damage. Guidelines intended to address specifically the issue of progressive collapse are UFC 4-023-03 [3], the GSA guidelines [4] and the CO.S.T. TU0601 guidelines [5]. The general approach of the aforementioned is also based on the creation of tying forces within the building which constrain the relative displacement of structural elements in case of structural dam age. Additionally, they provide further details on protection methods and the procedure of assessment of the building's performance under such a scenario. The alternative load path should be provided by peripheral ties, internal ties, horizontal column or wall ties, and where required, vertical ties, particularly in panel buildings. Additionally, when a building is divided by expansion joints into structurally independent sections, each section should have an independent tying system.

Various retrofit methods can be applied by engineers to increase the overall strength and robustness of structures in order to prevent the occurrence of progressive collapse when a structure is damaged by a severe earthquake or an accidental load. However, when a similar philosophy is adopted during design, an improved structural performance can be achieved with the same or reduced cost, as structural elements which cannot be retrofitted in existing buildings, can be suitably designed in a new one and, consequently, increase the building's inherent robustness significantly.

Cables have been extensively researched on in literature ([6] to [29]). As a retrofit method for masonry buildings, cables are used as horizontal ties to keep together in contact vertical walls in two perpendicular directions. In reinforced concrete structures, post-tensioned cables are used along beams to increase their stiffness. Alternatively, they are used as peripheral ties to achieve better load distribution among structural elements at the same floor. As diagonal elements, cables can be used as an alternative to typical bracing sections (i.e. angular sections, hollow sections, etc.) with significant advantages. Cables receive only tensile forces, while they are free to deform under compression, preventing them of undesirable types of failure such as flexural buckling, or various local buckling types which might occur to the aforementioned section types.

II. Structural Modelling

Many software packages use the finite element method in order to simulate structural elements and their non-linear behavior. For the purposes of structural simulation in this work, the OpenSEES software [30] was used.

Three-dimensional models were defined consisting of plane frames in two vertical directions. Use of two-dimensional models ignores the effect of beams and bracings lateral to the investigated frames on the reduction of vertical deflections, which can be significant. Distributed plasticity elements were selected over lumped plasticity elements, as the latter consider linear shape of the beam between the two joints, so it might underestimate the deflections that develop on the beams of the damaged building when the gravitational loads are applied on them. Composite slabs and secondary beams were not defined, but their effect on the structural behavior was modeled using the 'RigidDiaphragm' command which was found to be suitable for the analyses performed. It defines a rigid diaphragm at the defined direction (here it is on x-y plane), but when large deformations in the lateral direction(s) occur, they are not constrained. Beam-column connections and column bases were simplified as fixed or hinged connections, based on their properties.

The database of OpenSEES software contains numerous material models, suitable for the simulation of a variety of materials and elements. Two criteria were used for the selection of the material models: the effectiveness on the simulation of the behavior of an actual structure and the required convergence time. Because structural design optimization is used, a lot of combinations and numerous of iterations for each combinations are required, making the analysis procedure particularly time-demanding. Complex material models require much iteration until they converge to a compatible constitutive law, while they have increased potential of failure to define suitable stress-strain pairs when the desired level of accuracy is high. Such a failure during structural simulation might be misinterpreted as structural failure (e.g. due to local collapse), so designs that have highly desirable characteristics might be considered infeasible. So, simplified material models which achieve the required level of accuracy were used. The bilinear material model 'Steel01' was used for the simulation of the steel core of the columns, as well as the beams and bracings. 'ReinforcingSteel' material was used to simulate the longitudinal and transversal reinforcement of the columns. Cables were modeled using the 'ElasticPP' model, which creates a material with a linear elastic branch, but considers zero post-elastic stiffness. The user can have the option to define an initial post-tension or sagging of the cable using the appropriate stress-strain combination.

In order to simulate the behavior of structure four analysis types were used for each building. Firstly, an elastic analysis under vertical loads, secondly, modal analyses, thirdly, two nonlinear static pushover analyses and finally, 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 [31]. Modal analysis was performed in order to define the dynamic characteristics of the buildings such as the fundamental period and corresponding mode shape. Non-linear static analyses were also performed. 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 horizontal 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 [32]. For design purposes the maximum inter-storey drift limit defined in ASCE/SEI 41-06 [33] for steel buildings was used. The criteria defined in UFC 4-023-03 [3] for buildings with steel beams were selected for the steel-concrete composite buildings evaluated in this work. In order to calculate the vertical displacement and the redistribution of the internal forces for each scenario, pushdown analysis per damage scenario considered was performed for each building. The pushdown analysis was initiates with initial conditions the final state of each pushover analysis with horizontal loads.

III. Configuration Of The Optimization Problem

To attain automatically the most cost-effective designs, an optimization algorithm needs to be employed. Various optimization algorithms have been proposed and applied in engineering problems. In this work, the "Evolution Strategies" optimization algorithm was selected. This algorithm was initially proposed by Rechengerg [34] and Schwefel [35] and is based on the natural mechanism of evolution of a species. During transition from one generation to the next one, there is a higher probability that the fittest members will succeed in passing on their characteristics. Additionally, stochastic mechanisms (mutation, reordering and crossover) attribute characteristics to the offsprings which make them different than their parents, so the algorithm can scan the search space with reduced probability of being trapped near a local optimum. This algorithm has been found to successfully yield optimized designs in a variety of engineering applications. In the work of Papavasileiou and Charmpis [36] an analytical description of the algorithm's characteristics takes place, as well as its configuration for application on structural design optimization of steel-concrete composite buildings against earthquake is proposed. This work uses a modified version of the aforementioned in order to incorporate the use of cables as an alternative to typical bracing sections. The optimization problem solved is a mixed sizing and topology problem. In particular, the optimization algorithm is allowed to define (a) the sections of the linear structural elements, *i.e.* columns, beams and cables, (b) the locations where cables are installed and (c) the level of post-tensioning that is applied on each cable.

A database of standard wide-flange HEB sections was used for the steel columns. Use of different section for each column would render the solution of the problem cumbersome, while the assembly of the frames would be particularly difficult, as different connections should be realized at each splice point. Hence, columns are grouped in order to allow for a level of prefabrication in the construction process, as well as to reduce the independent variables of the optimization problem. On the floor plan four distinct section groups are defined: (a) corner columns, (b) peripheral columns in x- direction, (c) peripheral columns in y-direction and (d) internal columns. In [36], grouping of sections on multiple storeys is also proposed. So, the definition of a section group for each two or three consecutive storey would reduce significantly the number of variables, while it is not expected to result in particular increase of the total material cost. Beams were designed using standard IPE sections. Grouping of beams was also considered to be required. Since the gravitational loads received depend mainly on the direction in which they run, one section group was selected for each direction. Additionally, same sections were used for the same number of consecutive storeys that was defined for columns.

The cable database consists of solid circular sections with diameters ranging from 20mm to 32mm, rounded up to 1mm. Multiple cables were also considered to allow for increased stiffness to be provided by the cables to the building. Contrary to the typical approach selected for bracings, *i.e.* specific bays are selected and bracings are installed on all storeys at these bays, cables in this work were allowed to be installed at any location or direction. To prevent the concentration of cables around specific locations [37] and to avoid stiffness irregularities, which would result in undesirable seismic performance of the building, the following constraints were used:

(a) two distinct groups of cables are defined: (i) cables that are intended to be used against lateral loads and (b) cables that take part mainly in the alternate load paths and contribute only to the building's collapse resistance
(b) the number of cables on each of the two aforementioned groups is fixed
(c) the storey on which the cables of the first group are installed is standard, but the bay may vary
(d) the cables of the second group can be freely installed in any location

(e) cable sections and post-tensioning ratios might differ, but the total interstorey stiffness provided by the cables on one level should be less than that provided in the level below.

Two objective functions have been used separately in this work: (a) the building's progressive collapse resistance and (b) the total material cost. The structural performance against progressive collapse is typically assessed using indicators of the extent of the damage in the building, after the initial failure has occurred and the loads have been applied on the damaged building. UFC 4-023-03 [3] uses the maximum plastic rotation developing at the end of the beams as such an indicator, also employed in this work. The larger the maximum plastic rotation is yielded by a pushdown analysis, the higher the probability partial of total collapse will occur in the actual building. The total material cost used as an objective function, is that of the individual elements related to the defined independent variables: columns, beams, cables. All elements have pure steel sections, so the total cost can be calculated as the total steel mass required, considering that an average price applies on all steel members during construction. Varying elements such as the connections between the linear structural members should be properly taken into account in the objective function. In this work, the total cost of the connections is considered to be directly related to that of the connecting members: the larger the individual members are, the larger the connections need to be. So, buildings with increased values of the objective function are typically not expected to be more costly compared to ones with lower values of the objective function, even if the cost of connections is explicitly taken into account.

All feasible designs need to comply with the requirements of the applicable codes for (a) individual element capacity, (b) overall structural performance against earthquake and (c) progressive collapse resistance. To this end, EN 1993-1-1 [31] was used to define the capacity of steel columns, beams and cables, as well as for the design of the connections. The building's behavior under seismic loads was assessed using the provisions of FEMA-440 [32] and ASCE/SEI 41-06 [33] for steel buildings, typical building usage and type of soil. The overall collapse resistance of each building was assessed using the guidelines of UFC 4-023-03 [3] for buildings with steel beams.

IV. APPLICATION

To investigate how the most cost-effective design can be determined for various design characteristics; a typical six-storey building was designed using structural design optimization as described in the previous section. The building has five bays in each horizontal direction with a total beam span of 7m. The height of the first storey is 4.0m, while all storeys have a total height of 3.2m. As discussed previously, there are no bracings used, as they are substituted by cables.

UFC 4-023-03 [3] proposes the use of multiple damage scenarios in various storeys of the building in order to assess its overall robustness. Similarly, to achieve increased overall robustness, such damage scenarios can be used during the initial design of a building. In this work, multiple damage scenarios were simulated involving the loss of a single column. The location of the damage is at the corner of the building to create the most unfavorable loading conditions for the beams which would force the algorithm to either increase them resulting in significant increase to the total cost, or find an alternative using cable which would achieve the desirable performance. The damage is considered to be the result of an accident during typical building use (e.g. the explosion of a gas tank) so it may occur at any elevation. Hence, the number of damage scenarios required is equal to the number of storeys of the building, *i.e.* six.

Definition of one variable for each of the aforementioned independent variables could possibly provide the most cost-effective design for the objective function and the constraints set, as the optimization algorithm would be allowed to change each of them independently seeking for the optimum value of the objective function. However, in engineering practice this is not preferred. On the one hand, the computational time required would be particularly increased, as the number of independent variables would be very large, so the possible combinations would be practically infinite. On the other hand, using different sections for each element renders the design process cumbersome, as a different connection would have to be designed for each pair of structural elements and the construction more complex and expensive, as the advantages of prefabrication in steel structures are waived.

To reduce the number of variables, the following constraints apply:

(a) Columns consist of the same section from ground to top floor.

(b) Beams have the same section in both directions.

(c) Cables are installed in pairs and symmetrically in both parallel faces to avoid possible stiffness asymmetries. Additionally, the cable pattern defined for one direction is repeated for the other, so for the definition of cable topology, this needs to be determined for one face only and the others are automatically defined.

(d) There is at least one cable-pair at each storey which as substitute for typical bracing sections.
(e) There is at least one more cable-pair for each storey which is used only to improve the building's progressive collapse resistance. However, they can be installed at any storey or bay.
(f) One post-tensioning ratio is selected for all cables against earthquake. Cables against progressive collapse have initial sagging larger than the maximum admissible interstorey drift, so they do not participate in the building's load-bearing mechanism under seismic excitations.
(g) One section is used for all cables, so only the topology of the cables needs to be determined by the optimization algorithm.

Based on the aforementioned:

(a) Four variables need to be defined: one for each of the four aforementioned column section groups.
(b) One beam group needs to be defined for all beams in model.
(c) The number of cables installed against earthquake is equal to the number of storeys. So, the total number of cables installed is equal to two times the number of storeys.

For a six-storey building, this results in 17 independent variables: 4 column section groups, 1 beam section group, 6 cable topologies for post-tensioned cables (installed against horizontal actions) and 6 cable topologies for sagging cables (installed for progressive collapse resistance). The problem constraints are outlined in the previous section.

Two distinct problems were defined. In the first problem, the intended maximum plastic rotation is fixed and the objective function is the building's total cost expressed in total steel mass. Beams and columns are selected from the aforementioned databases. Cable diameters and post-tensioning ratio can be freely selected, while the target is to define the most cost-effective design that does not exceed the maximum admissible vertical drift. Three limit values were selected for the vertical drift, *i.e.* 7.5%, 5.0% and 2.5%, defining this way three design cases (D1 to D3). The post-tensioned cables could be installed in any bay, one at each storey, while the sagging cables can be freely installed at any bay and/or storey. All cables installed against progressive collapse have an initial sagging of 5%, which is equal to the maximum admissible interstorey drift defined in ASCE/SEI 41-06 [33] for steel buildings. Buildings exceeding this drift in the pushover analyses are considered infeasible, so in all feasible designs these cables do not interfere with the building's earthquake response. To investigate the effect to the total cost of the building if cables has a fixed section, three more designs (D4 to D6) were optimized for cable diameter 10mm and the vertical drift limits previously outlines.

In the second problem, the optimization algorithm needs to reduce as much as possible the maximum vertical drift (which corresponds to the maximum plastic rotation at the end of the beams). Cable diameters are the same for all cables, *i.e.* 10mm, 14mm and 21mm. Beam and column sizes are selected by the algorithm from a database of IPE sections for the beams and HEB sections for the columns. Two post-tensioning ratios were defined: (a) 0%, and (c) 40% for cables installed against horizontal actions, defining a total number of six designs to be optimized (D7 to D12). This ratio is the same for all cables in each design case. The designs yielded are presented in Table 1.

The designs defined for D7, D8 and D9 have the same topology as those defined for D10, D11 and D12 respectively, so they have been grouped in Table 1. The reason for this is that, the post-tensioned cables are away from the location of the damage, so the alternate load path employs only the sagging cables.

TABLE 1. CHARACTERISTICS OF THE OPTIMIZED DESIGNS YIELDED.

Design ID	Cable Diameter (mm)	Post-Tensioning Ratio	Column Section	Beam Section	Vertical Drift	Vertical Drift Target	Maximum interstorey drift	Total Steel Mass (tn)
D1	10	40%	HE400B	IPE220	7.216%	7.500%	3.591%	177.300
D2	14	40%	HE400B	IPE220	4.649%	5.000%	4.028%	177.735
D3	21	40%	HE400B	IPE220	2.454%	2.500%	4.846%	178.844
D4	10	-5%	HE400B	IPE220	7.305%	7.500%	3.612%	177.300
D5	10	-5%	HE550B	IPE450	4.906%	5.000%	4.297%	337.300
D6	10	-5%	HE1000B	IPE600	3.993%	2.500%	4.215%	531.726
D7 / D10	10	0% / 40%	HE1000B	IPE600	3.997%	minimum	4.326% / 4.215%	531.726
D8 / D11	14	0% / 40%	HE1000B	IPE600	3.235%	minimum	4.491% /4.377%	532.161
D9 / D12	21	0% / 40%	HE1000B	IPE600	2.054%	minimum	4.996% /4.749%	533.270

D6 does not achieve the vertical drift target set. However, as the maximum column and beam sections have been used, while the cable diameter is fixed, it is safe to assume that this is the design closer to the target, so the target is infeasible under the constraints defined. Comparison with D9 shows that it is possible to achieve even smaller values of the maximum recorded vertical drift by allowing larger cables to be selected by the optimization algorithm.

There is a minimum size of beams and columns defined by the gravitational loads when applied on the undamaged building and the strength/performance criteria of EN 1993-1 [31] for steel members. So, increased cable diameters would not result in respective reduction of the beam/column sizes, but they can change the building’s response under seismic loads or accidents.

Increased post-tensioning ratio provides improved behavior against progressive collapse when these cables are installed over the damaged bays. Additionally, it reduces the maximum interstorey drift developing under seismic excitations. However, with the constraints defined in this problem, this creates stiffness asymmetries in the building, so its performance in earthquake is deteriorating. Additionally, as cables are intended to perform elastically, the maximum strain of the cables needs to be determined beforehand, so that it does not exceed the yielding strain of the cables.

For increased requirements on progressive collapse resistance, the total cost is inevitably increased as well. The more demanding the performance constraints become, the higher the rate of this increase is as well.

Use of small cable diameter (D4 to D6) forces the algorithm to increase beam sections in order to compensate for the required strength of the elements neighboring the location of the damage so that the loads from the failed elements are redistributed safely through them.

To achieve very small deflections, as required in D7 to D12, the algorithm tends to install all cables intended for earthquake-induced loads at corner of the building, which is the location where damage occurs in all modelled damage scenarios. The reason for this design is that cables installed against progressive collapse have initial sagging in order to avoid altering structural performance under horizontal actions. They start participating in the alternate load paths only after the deflection of the beams is larger than the initial sagging. So, their strength contribution in the load bearing mechanism is significantly low compared to that of cables without sagging or with significant post-tensioning. It should be noted that, while the building’s performance under horizontal actions is affected, as rotational effects develop due to the non-symmetric placement of cables, the deterioration is not large enough to render the designs infeasible, as the contribution of the columns is sufficient to counteract.

Based on the beneficial effect of the installation of cables in various bays, it is safe to assume that, if the diameter of the cables was not fixed for designs D4 to D6, but was handled as an independent variable, either uniform for all cables, or one variable for each cable group, the optimization algorithm would ultimately use the maximum available cable diameter in order to achieve the most cost-effective use of structural steel. Large cable sections can receive larger loads and relieve the neighboring beams from them under damage scenarios. So, use of strong cables allows the reduction of beam sections as low as possible.

V. CONCLUDING REMARKS

To ensure that cables remain elastic throughout their use as part of the alternate path, the engineer needs to pay attention to the maximum strain expected to develop. If this exceeds the yielding strain, the cables will yield. So, even though post-tensioning has a beneficial effect in the building’s collapse resistance, their use against both hazards should be

In extent of the remark made on the use of strong cables, using cables with increased diameter and initial sagging can allow buildings with very weak beams to be designed, which would be expected to develop large deflections without cables, while at the same time they easily conform to the “weak beam – strong column” rule for ductile behavior during earthquakes. These buildings can achieve the desirable performance under both earthquake and progressive collapse and have particularly reduced cost than buildings designed conventionally, *i.e.* with increased beam stiffness.

When the location of the damage cannot be determined before construction, a generic approach with post-tensioned cables spread on the face of the building can provide significant robustness, improving the overall collapse resistance of the building.

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 Programs” 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] EN 1992-1-1. “Eurocode 2: Design of reinforced concrete structures – Part 1-1: General rules and rules for buildings”, Brussels, Belgium: CEN. 2004.

[3] Department of Defense (DoD). *Unified Facilities Criteria (UFC) – Design of buildings to resist progressive collapse, UFC 4-023-03*, USA, 2009.

[4] G.S.A. ‘*Progressive Collapse Design Guidelines Applied to Concrete Moment-Resisting Frame Buildings’, General Services Administration, Nashville, Tennessee*. 2004.

[5] CO.S.T. TU0601 – Canisius T.D.G. (Editor). *Structural Robustness Design for Practicing Engineers* 2011.

[6] 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.

[7] 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.

[8] 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.

[9] 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.

[10] G. A. Costello. *Theory of wire rope*. Springer Science and Business Media. 1997.

[11] 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.

[12] S. R. Ghoreishi, P. Cartraud, P. Davies and T. Messager. 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.

[13] F. H. Hruska. Calculation of stresses in wire ropes. “*Wire and wire products”*, vol. *26*, pp. 766-767. 1951.

[14] F. H. Hruska. Radial forces in wire ropes. “*Wire and wire products”*, vol. *27* no. 5, pp. 459-463. 1952.

[15] F. H. Hruska. Tangential forces in wire ropes. “*Wire and wire products”*, vol. *28* no. 5, pp. 455-460. 1953.

[16] 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.

[17] 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.

[18] 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.

[19] K. Kumar J. Botsis Contact stresses in multilayered strands under tension and torsion. “*Journal of Applied Mechanics*“ vol. 68:pp.432–440. 2001.

[20] 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.

[21] 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.

[22] 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.

[23] 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.

[24] JW. Philips and GA. Costello. Analysis of wire rope with internal-wirerope cores. ASME “*Journal of Applied Mechanics*”, vol. *52*, pp. 510–6. 1985

[25] 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.

[26] 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.

[27] 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.

[28] 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.

[29] S. A. Velinsky. General nonlinear theory for complex wire rope. "*International journal of mechanical sciences"*, vol. *27* no. 7, pp. 497-507. 1985.

[30] S. Mazzoni, F. McKenna, M. Scott and G.L. Fenves. "Open System for Earthquake Engineering Simulation (OpenSees)", PEER Center, California, USA. 2006.

[31] EN 1993-1-1. "Eurocode 3: Design of steel structures – Part 1-1: General rules and rules for buildings", Brussels, Belgium: CEN. 2005.

[32] Federal Emergency Management Agency (FEMA). "Improvement of nonlinear static seismic analysis procedures, FEMA-440", Washington DC, USA, 2005.

[33] American Society of Civil Engineers (ASCE). "Seismic rehabilitation of existing buildings, Standard ASCE/SEI 41-06 (incl. suppl. 1)", Reston, Virginia, USA. 2006.

[34] I. Rechenberg, "Evolutionsstrategie: Optimierung technischer systeme nach prinzipien der biologischen evolution". Stuttgart, Germany: Frommann-Holzboog Verlag. 1973.

[35] H.P. Schwefel. "*Evolutionsstrategie und numerische Optimierung"*. Berlin : (Doctoral dissertation, Technische Universität Berlin). 1975.

[36] G.S. Papavasileiou and D.C. Charmpis. "Seismic design optimization of multi-storey steel-concrete composite buildings", Comput. Struct., vol. 170, pp. 49-61. 2016.

[37] G.S. Papavasileiou and N.G. Pnevmatikos. "Retrofit of Steel Buildings against Progressive Collapse Using Cables". In Proceedings of the 2nd International Conference on Recent Advances in Nonlinear Modelling – Design and Rehabilitation of Structures. 2017. (pp. 202 – 210).