COMPDYN 2015
5th ECCOMAS Thematic Conference on
Computational Methods in Structural Dynamics and Earthquake Engineering
M. Papadrakakis, V. Papadopoulos, V. Plevris (eds.)
Crete Island, Greece, 25–27 May 2015

# A COMPUTATIONAL INVESTIGATION FOR THE SEISMIC RESPONSE OF RC STRUCTURES STRENGTHENED BY CABLE ELEMENTS

**Angelos A. Liolios**

Democritus University of Thrace, Department of Civil Engineering,
Division of Structural Engineering, Xanthi, GREECE
e-mail: aliolios@civil.duth.gr

**Keywords:** Seismic Upgrading and Strengthening of RC Structures, Cable-braced Systems, Damage Indices.

**Abstract.** *A numerical investigation is presented for the seismic analysis of existing reinforced concrete (RC) structures, which have been damaged due to extreme actions and are seismically upgraded by cable elements (tension-ties). A double discretization, in space by the Finite Element Method and in time by a direct incremental approach, is used. The unilateral behavior of the cable-elements, which can undertake tension stresses only, is strictly taken into account and results to inequality constitutive conditions. So, in a first numerical approach, a non-convex linear complementarity problem is solved in each time-step. Alternatively, in a second numerical approach, an incremental problem formulation and treatment is applied. The decision for the optimal cable-strengthening scheme in praxis problems of RC structures under multiple earthquakes excitation can be obtained on the basis of computed damage indices. A calibration of the numerical approaches is realized in a typical example problem by comparing the computational results with available experimental ones. It is shown that cable strengthening has significant effects on the earthquake response and, hence, can be used effectively for the seismic upgrading of existing RC structures.*

# 1 INTRODUCTION

In Greece and other seismically active countries, many existing reinforced concrete (RC) frame buildings have been designed and built using the old seismic codes. Such old regulations for reinforcement details are often not adequate for proper seismic structural behavior, local and/or global. An example of inadequate local behavior is the case of the beam-column connections [1,2]. As concerns the global behavior, for such RC frames a need for seismic strengthening often arises. One of the simple, cheap and efficient methods for strengthening of RC frames against lateral induced earthquake load is the use of steel cross X-bracings, see e.g. Massumi [3,4] and the references given therein. The combination of reinforced concrete frame with steel cross X-bracing is not a common practice due to unknown behavior and performance that needs to be investigated. Research, mainly experimental one, on the use of this method of retrofitting has begun since 80s in which cross X-bracings have been used indirectly together with a steel frame confined by a concrete frame. Application of this technique is reported, among others, also in Greece, for the case of improving the seismic performance of existing old pilotis type multi-story RC buildings by strengthening only the ground story [5,6].

In general, seismic upgrading of existing RC structures makes use of well known today repairing and strengthening techniques, see e.g. Bertero et al [7], Penelis & Kappos [8], Dritsos [9], Fardis [10] and Penelis & Penelis [11]. An alternative technique, the one using cable-like members (tension-ties) instead of the mentioned steel cross X-bracing, can be used as a first strengthening and repairing procedure for RC building frames against earthquake actions, see Tegos et al, [12,13]. Cable restrainers are also used for concrete and steel superstructure movement joints in bridges [14]. These cable-members (ties) can undertake tension, but buckle and become slack and structurally ineffective when subjected to a sufficiently large compressive force. Thus the governing conditions take an equality as well as an inequality form and the problem becomes highly nonlinear, see Liolios et al, [15].

Regarding the strict mathematical treatment of the problem, the concept of variational and/or hemivariational inequalities can be used, as it has been introduced and successfully applied in engineering by Panagiotopoulos [16]. As concerns the numerical treatment, non-convex optimization algorithms are generally required [15-17].

On the other hand, current seismic codes, e.g. EC8 [18-22], suggest the exclusive adoption of the isolated and rare “design earthquake”, while the influence of repeated earthquake phenomena is ignored. This is a significant drawback for the realistic design of building structures. Despite the fact that the problem has been qualitatively acknowledged, few studies have been reported in the literature, especially the last years, regarding the multiple earthquake phenomena, see e.g. [23].

This study presents a numerical investigation for the seismic analysis of existing reinforced concrete (RC) building frames, which have been strengthened by cable elements. The investigation results to two proposed approaches, one based on linear complementarity problem formulation and treatment [15], and the second based on incremental formulation. The latter procedure uses the Ruaumoko structural engineering software [24]. Damage indices are computed and used for comparing various cable-bracing strengthening versions, in order the optimum one to be chosen, especially under multiple earthquakes. A calibration of the second approach is realized by using available experimental results for a simple typical one-bay one-story RC frame.

## 2 METHOD OF ANALYSIS

### 2.1 Formulation of the problem

As is usual in Structural Dynamics [25], a double discretization is applied. First, the structural system is discretized in space by using finite elements. Pin-jointed bar elements are used for the cable-elements. The behavior of these elements can in general include loosening, elastoplastic or/and elastoplastic-softening-fracturing and unloading - reloading effects. All these characteristics, concerning the cable full constitutive law, as well as other general non-linearities of the RC structure, can be expressed mathematically by using concepts of convex and non-convex analysis, see Panagiotopoulos [16]. So, for the cable-elements behavior, the following relation holds:

$$s_i\,(d_i) \;\in\; \hat{\partial}\,SP_i(d_i) \tag{1}$$

Here $s_i$ and $d_i$ are the (tensile) force (in [kN]) and the deformation (elongation) (in [m]), respectively, of the i-th cable element, $\hat{\partial}$ is the generalized gradient and $SP_i$ is the superpotential function, see Panagiotopoulos [16] and Mistakidis & Stavroulakis [17]. By definition, relation (1) is equivalent to the following hemivariational inequality, expressing the Virtual Work Principle:

$$SP_i^{\uparrow}\,(d_i, e_i - d_i) \;\geq\; s_i(d_i)\,\cdot\,(e_i - d_i) \tag{2}$$

where $SP_i^{\uparrow}$ denotes the subderivative of $SP_i$ and $e_i$, $d_i$ are kinematically admissible (virtual) deformations.

Next, dynamic equilibrium for the assembled structural system with cables is expressed by the usual matrix relation:

$$\boldsymbol{M}\,\ddot{\boldsymbol{u}} + \boldsymbol{C}(\dot{\boldsymbol{u}}) + \boldsymbol{K}(\boldsymbol{u}) \;=\; \boldsymbol{p} + \boldsymbol{A}\,\boldsymbol{s} \tag{3}$$

Here $\boldsymbol{u}(t)$ and $\boldsymbol{p}(t)$ are the displacement and the load time dependent vectors, respectively. The damping and stiffness terms, $\boldsymbol{C}(\dot{\boldsymbol{u}})$ and $\boldsymbol{K}(\boldsymbol{u})$, respectively, concern the general non-linear case. When the linear-elastic case holds, these terms have the usual form $\boldsymbol{C}\dot{\boldsymbol{u}}$ and $\boldsymbol{K}\boldsymbol{u}$. Dots over symbols denote derivatives with respect to time. By $\boldsymbol{s}(t)$ is denoted the cable stress vector. $\boldsymbol{A}$ is a transformation matrix. For the case of ground seismic excitation $\boldsymbol{u}_g$, the loading history term $\boldsymbol{p}$ becomes $-\boldsymbol{M}\ddot{\boldsymbol{u}}_g$.

The above relations (1)-(3), combined with the initial conditions, consist the problem formulation, where, for given $\boldsymbol{p}$ and/or $\ddot{\boldsymbol{u}}_g$, the vectors $\boldsymbol{u}$ and $\boldsymbol{s}$ have to be computed. Regarding the strict mathematical point of view, using (l) and (2), we can formulate the problem as a hemivariational inequality one by following [16, 17] and investigate it.

### 2.2 Numerical treatment

A numerical treatment of the problem, based on a piecewise linearization of the above constitutive relations as in elastoplasticity [35], is described in Liolios et al [15] for cable-braced RC systems. By using a direct time-integration scheme, in each time-step Δt a relevant non-convex linear complementarity problem (NC-LCP) of the following matrix form is solved :

$$\mathbf{v} \geq \mathbf{0}, \quad \mathbf{D\,v} + \mathbf{d} \leq \mathbf{0}, \quad \mathbf{v}^{\mathbf{T}}.(\mathbf{D\,v} + \mathbf{d}) = 0. \tag{4}$$

Here **v** is the vector of unknown unilateral quantities at the time–moment t, $\mathbf{v}^{\mathbf{T}}$ is the transpose of **v**, **d** is a known vector dependent on excitation and results from previous time moments (t- $\Delta$t), and **D** is a transformation matrix. The problem (4) is solved by using available optimization algorithms [15-17].

So, the nonlinear Response Time-History (RTH) for a given dynamic and/or seismic ground excitation can be computed.

An alternative approach for treating numerically the problem is the incremental one. Now, relation (3), taking into account both, load history and ground seismic excitation, is written in incremental form:

$$\boldsymbol{M}\,\Delta\ddot{\boldsymbol{u}} + \boldsymbol{C}\,\Delta\dot{\boldsymbol{u}} + \boldsymbol{K_T}\,\Delta\boldsymbol{u} = -\boldsymbol{M}\,\Delta\ddot{\boldsymbol{u}}_{\boldsymbol{g}} + \boldsymbol{A}\,\Delta\boldsymbol{s} + \Delta\boldsymbol{p}, \tag{5}$$

Here $\boldsymbol{K_T}$ is the tangent stiffness matrix, by which restoring nonlinear force-displacement relations are taken into account [24, 25].

The structural analysis software Ruaumoko [24], which uses the finite element method, is based on this approach. For the time-discretization, the Newmark scheme is here chosen. Ruaumoko provides results which concern, among others, the following critical parameters: local or global structural damage, maximum displacements, interstorey drift ratios, development of plastic hinges and various response quantities, which allow the using of the incremental dynamic analysis (IDA) method [26].

Further, Ruaumoko has been applied successfully for multiple earthquakes concerning the cases of concrete planar frames [23] and RC frames strengthened by cables [27]. It is reminded that multiple earthquakes consist of real seismic sequences, which have been recorded during a short period of time (up to some days), by the same station, in the same direction, and almost at the same fault distance. In the reported applications [23, 27], complete lists of the used earthquakes sequences were downloaded from the strong motion database of the Pacific Earthquake Engineering Research (PEER) Center [36].

### 2.3 Damage indices used for assessment and comparison

After the seismic assessment [10,11,22] of the existing RC structure, the choice of the best strengthening cable system can be realized by using damage indices [28-33]. Here the overall structural damage index (OSDI) is used. This parameter summarizes all the existing damages on columns and beams of reinforced concrete frames in a single value, which is useful for comparison reasons [33].

In the OSDI model after Park/Ang [28] the global damage is obtained as a weighted average of the local damage at the section ends of each frame element or at each cable element. The local damage index is given by the following relation:

$$\mathrm{DI_L} = \frac{\mu_m}{\mu_u} + \frac{\beta}{F_y d_u} E_T \tag{6a}$$

where: $\mathrm{DI_L}$ is the local damage index, $\mu_m$ the maximum ductility attained during the load history, $\mu_u$ the ultimate ductility capacity of the section or element, $\beta$ a strength degrading parameter, $F_y$ the yield force of the section or element, $E_T$ the dissipated hysteretic energy.

The modified Park/Ang local damage index as proposed by Kunnath et al. [29] is given by the following relation:

$$DI_{L,M} = \frac{\theta_m - \theta_r}{\theta_u - \theta_r} + \frac{\beta}{M_y \theta_u} E_T \tag{6b}$$

where the deformation ductility was replaced by the rotation; while $\theta_m$ is the maximum rotation attained during the loading history; $\theta_u$ is the ultimate rotation capacity of the critical region; $\theta_r$ is the recoverable rotation after unloading; $M_y$ is the yield moment; and $E_T$ is the dissipated energy in the critical region. The element damage is selected as the largest damage index of the end critical region.

In the global damage index, which is a weighted average of the local damage indices, the dissipated energy is chosen as the weighting function. So, the global damage index is given by the following relation:

$$DI_G = \frac{\sum_{i=1}^{n} DI_L E_i}{\sum_{i=1}^{n} E_i} \tag{7}$$

where $DI_G$ is the global damage index, $DI_L$ the local damage index, $E_i$ the energy dissipated at location i and n the number of locations at which the local damage is computed.

## 3 CALIBRATION AND APPLICATION OF THE NUMERICAL APPROACH

### 3.1 The available experimental results

A calibration of the presented numerical approach is realized by using available experimental results for a simple typical one-bay one-storey RC frame strengthened with steel cross X-bracing and investigated by Massumi et al [3,4 ].The results concern two experimental models of reinforced concrete frames, which have been designed on the basis of old traditional codes. The first of them is a bare-frame and the second is the same frame, but strengthened with steel X-bracing.

The single-bay, one-storey frame and the experimental testing procedure are shown in Figures 1, 2 and 3 adapted from [3,4]. The frame was of concrete C25 and had section dimensions (12cm)x(12cm) for the two columns and the beam. The steel bracings were squared tubes with section (20mm)x(20mm) and thickness t=2mm. Yield strength of steel bracings was 240 MPa. For the details of the experimental program see Massumi et al [3,4].

During the experiments, a lateral additive cyclic static load P was applied to the storey beam in a displacement-controlled mode, see Fig. 3. The relevant diagrammes of the load P versus to the top displacement u hysteretic behavior for the unbraced (bare) frame and the X-braced frame are shown in Fig. 4. Similar hysteretic diagrammes for bare frames are reported in other experimental publications, see e.g. [37].

Based on the above experimental results and using Ruaumoko software, in the next paragraph the finite element model related to these frames is made by using cable-elements and calibrated, and then nonlinear analyses under additive cyclic static loading suggested by the experimental results are performed.

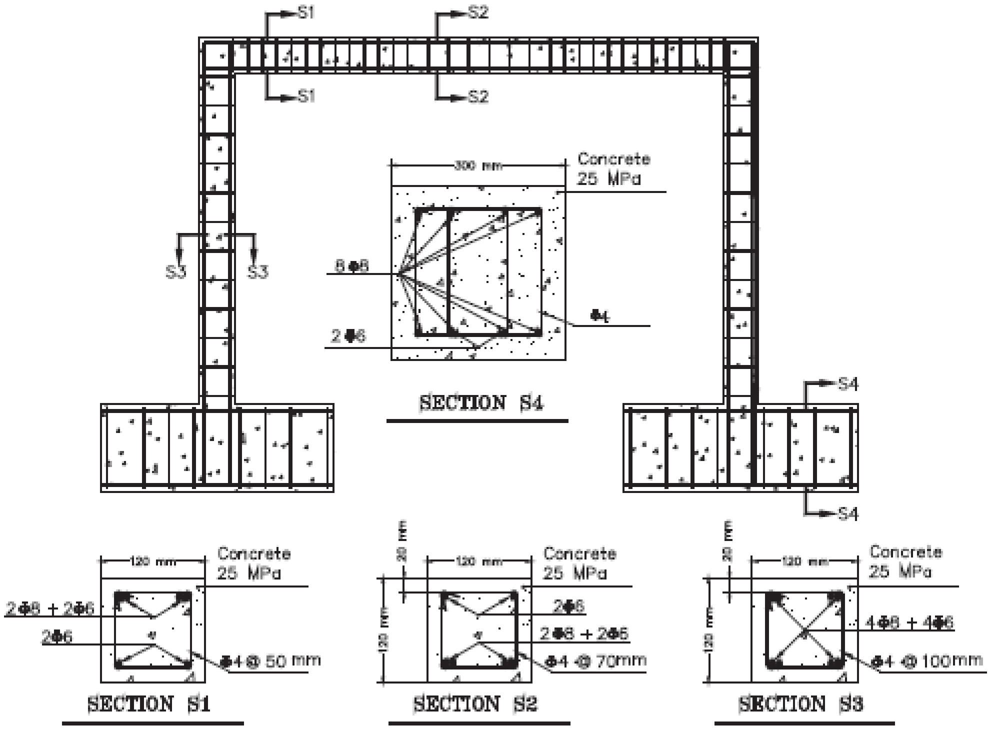


Figure 1. Details of the investigated RC frames (adapted from Massumi et al [4] after permission)

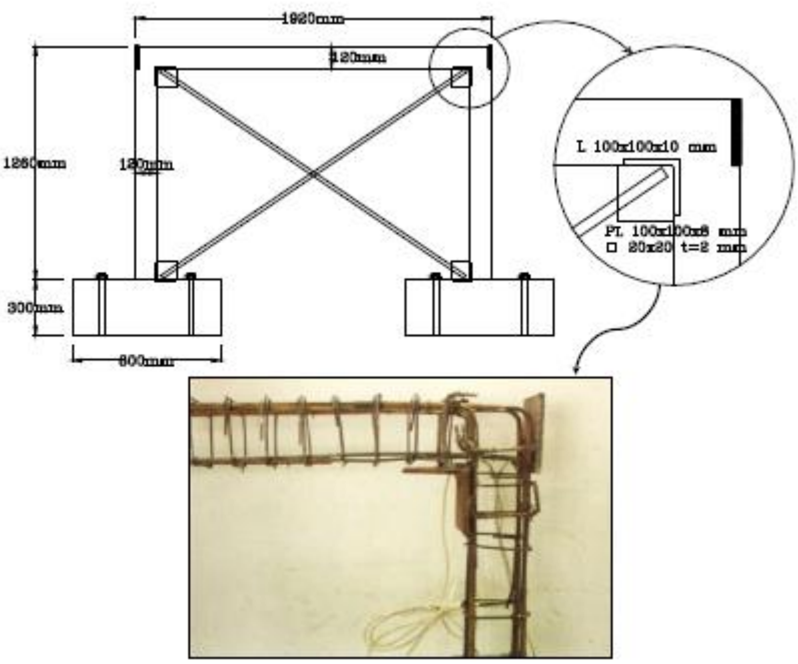


Figure 2. Strengthened RC frame with X-cross bracings and its connection details (adapted from Massumi et al [4] after permission)

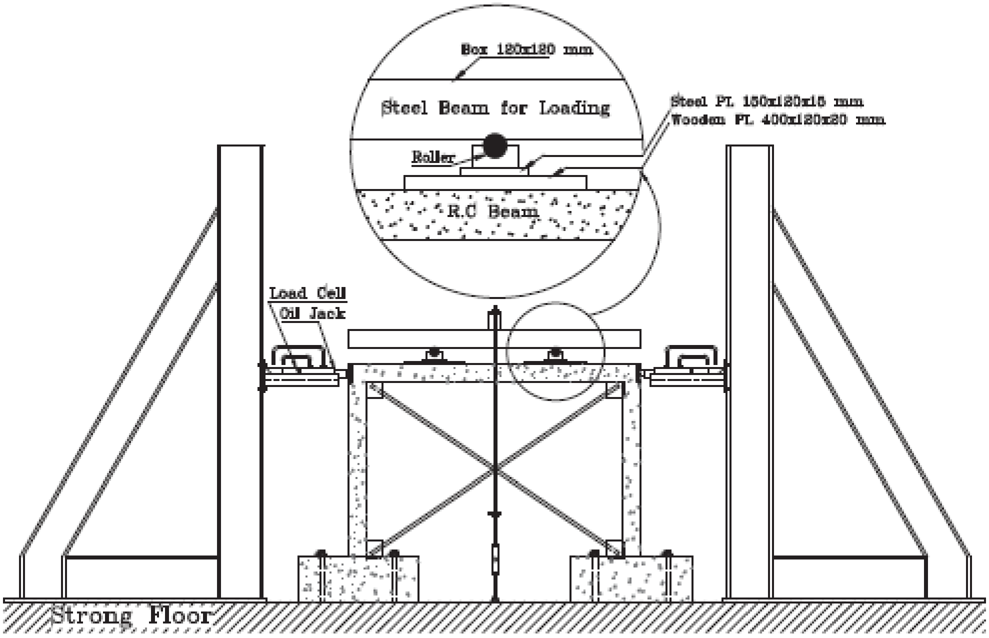


Figure 3. Test setup and loading systems (adapted from Massumi et al [4] after permission)

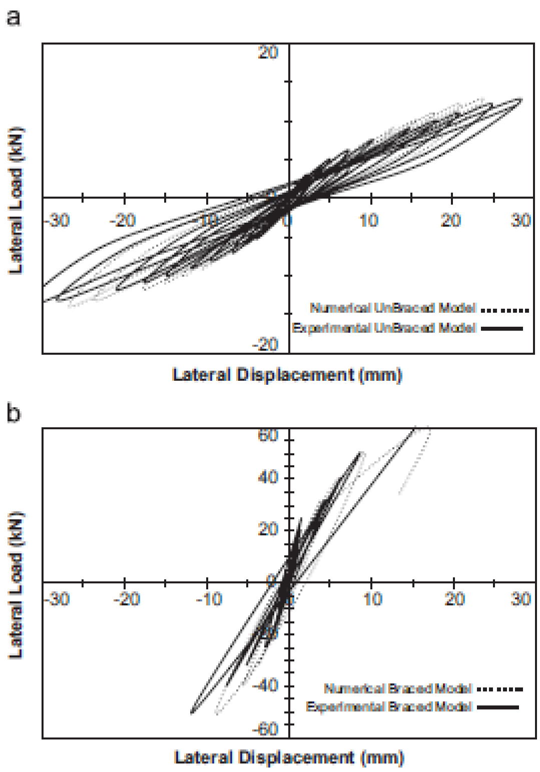


Figure 4. Experimental load-displacement hysteretic behavior for (a) unbraced frame, and (b) X-braced frame (adapted from Massumi et al [4] after permission)

### 3.2 Simulation of the Massumi et al [3,4 ] experiments

With respect to above experimental results, the finite element models related to the unbraced and braced frames are made using Ruaumoko software [24]. The two columns and the beam are modeled using prismatic frame elements. Nonlinearity at the two ends of RC members is idealized using one-component plastic hinge models, following the Takeda hysteresis rule. The effects of cracking on columns and beams are estimated by applying the guidelines of EC8-Part3 [10, 18, 19] and KAN.EPE [22]. So, the stiffness reduction due to cracking results to effective stiffness of 0.450 $I_g$ for the columns and 0.225 $I_g$ for the beam, where $I_g$ is the gross inertia moment of the section (12cm)x(12cm).

For the steel bracings simulation, cable-elements are used herein by applying the bilinear with slackness hysteresis rule [24]. These cable elements have a cross-sectional area $F_c$ = 1.44 $cm^2$, which is equivalent to the cross-sectional area of steel braces tubes with section (20mm)x(20mm) and thickness t=2mm used in Massumi et al [3], see Fig. 2. Yield strength of cable bracings was considered to be also the same one of [3], i.e. 240 MPa. Further, yield strain $\varepsilon_y$ = 0.2 %, fracture strain $\varepsilon_f$ = 2 % and elasticity modulus $E_s$ = 200 GPa are taken for the cables. So, the cable constitutive law, concerning the unilateral (slackness), hysteretic, fracturing, unloading-reloading etc. behavior, has the diagramme depicted in Fig. 5. Ductility index is $\mu = d/d_y$ .

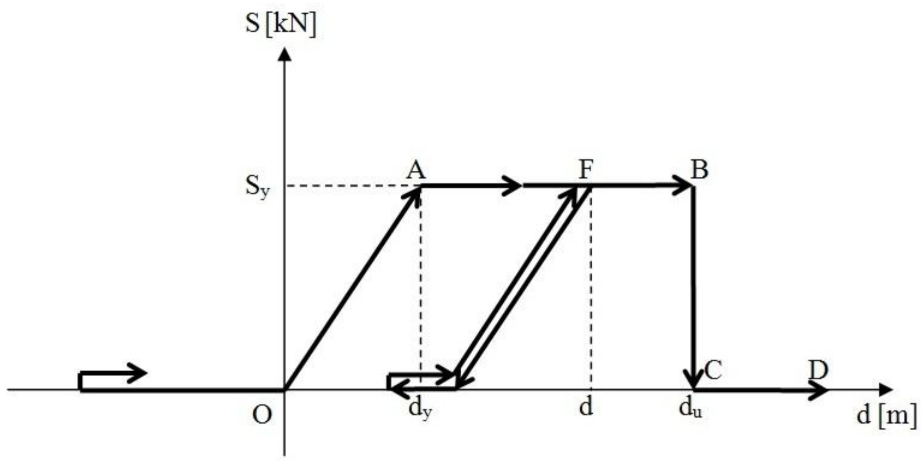


Figure 5. The diagramme for the constitutive law of cable-elements.

Based on the experimental procedure [3,4] and on Fig. 4, the loading history has been simulated as an displacement-controlled multiple earthquake excitation imposed on the top left node of the frame. This loading simulation had full loading steps with maximum displacements ±5, ±10, ±15, ±20, ±25 and ±30 [mm] for the unbraced frame, whereas for the X-braced frame the loading steps had maximum displacements ±2.5, ±5.0, ±7.5, ±10, ±12.5 and ±15 [mm].

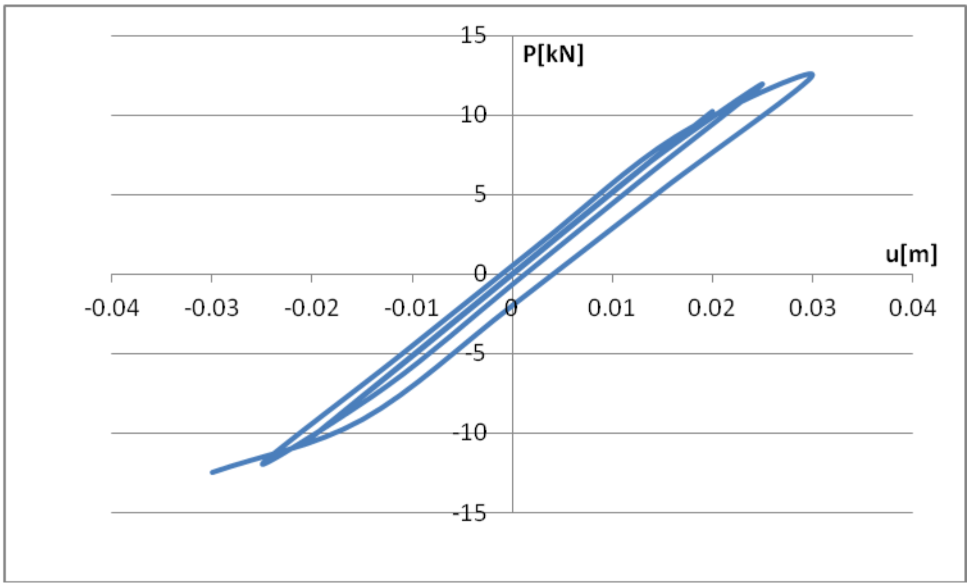


Figure 6. Simulated load-displacement hysteretic behavior for the unbraced frame.

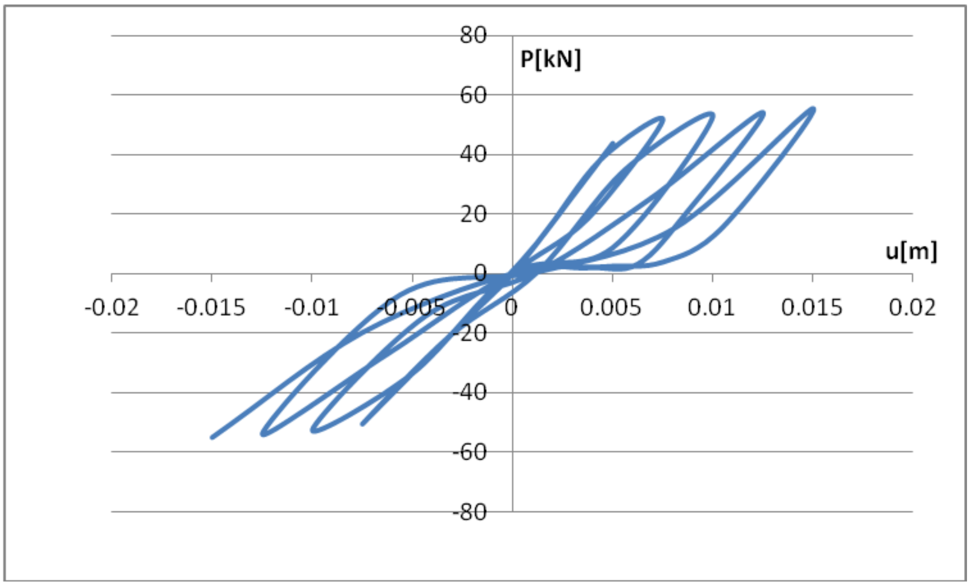


Figure 7. Simulated load-displacement hysteretic behavior for the X-cable braced frame.

The relevant diagrammes of the (load P)-(top displacement u) hysteretic behavior for the unbraced (bare) frame and the X-braced frame are shown in Fig. 6 and Fig. 7, respectively. As these figures show, the computed results are in a good agreement with the experimental ones of Fig. 4.

Some representative results of the numerical simulation are presented in next Table 1. In column (2) the Global Damage Index $DI_G$ and in column (3) the Local Damage Index $DI_L$ for the bending moment at the left fixed support of the frame are given. Finally, in the column (4), the developed maximum horizontal top force $P_{top}$ is given.

| FRAME | $DI_G$ | $DI_L$ | $P_{top}$ [kN] |
|---|---|---|---|
| (1) | (2) | (3) | (4) |
| Unbraced | 0.098 | 0. 201 | 12.52 |
| X-braced | 0.001 | 0.001 | 55.40 |

Table 1. Representative response quantities for the unbraced and braced frames

The table values show, as expected, that cable- strengthening generally on the one hand reduces the damage indices and on the other hand increases the global horizontal frame resistance. So, for the unbraced frame, is $P_{max}$ = 12.52 kN, whereas for the cable-X-braced frame is $P_{max}$ = 55.40 kN. That means that the frame-stiffness is increased by a factor 55.4/12.52 = 4.425. As concerns the damage indices, they are minimized for the cable-strengthened frame. Similar concluding remarks hold for multi-story and multi-bay real praxis RC frames as investigated in [27]. Therein is shown that a further improvement of the seismic behavior of the initial frames can be obtained by trying various cable-bracing schemes and by realizing a parameter sensitivity analysis concerning the cable characteristics, e.g. cross-sectional area $F_c$ or steel class.

### 3.3 Application for the case of one-tie braced frame

After the previous calibration of the numerical procedure, and in order to clarify the unilateral behavior of the cable-elements and their effects on the structural response, the same frame of Fig. 1 is investigated, but with only one tension-tie, the ascending diagonal cable, as shown in Fig. 8.

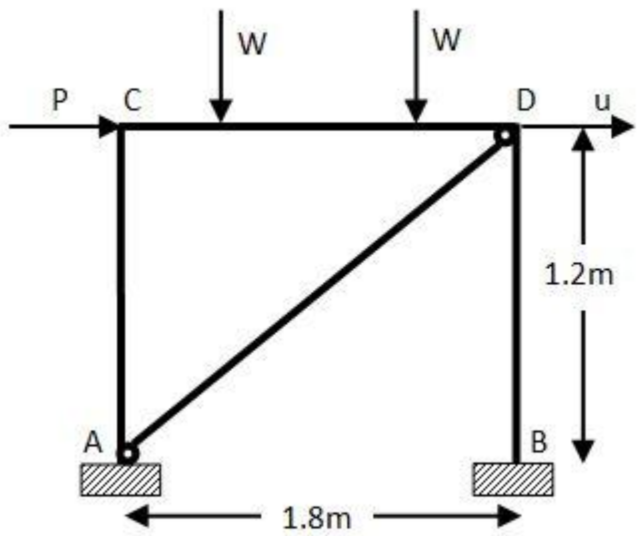


Figure 8. The one-tie braced frame

The loading history had the same full loading steps as for the X-braced frame, i.e. the loading steps had maximum displacements ±2.5, ±5.0, ±7.5, ±10, ±12.5 and ±15 mm.

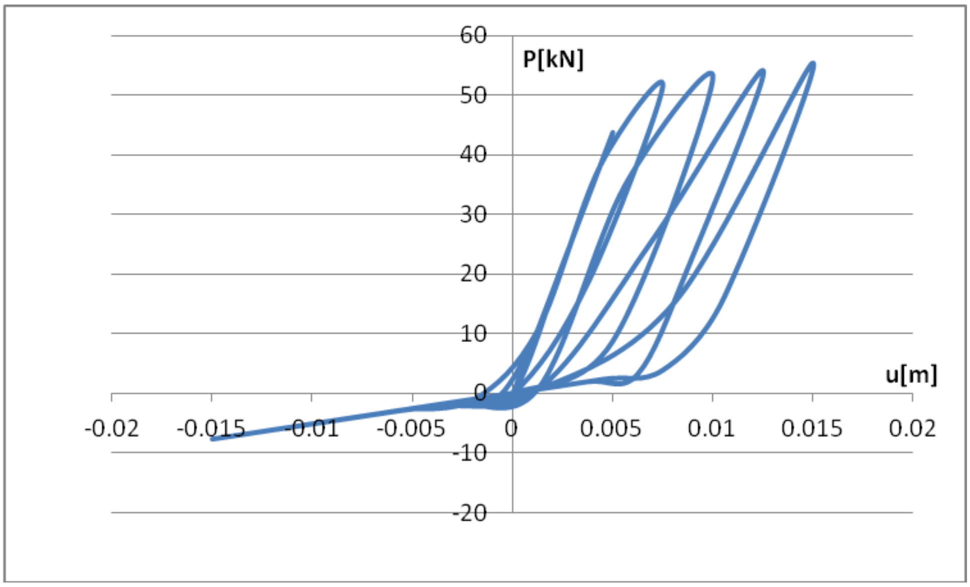


Figure 9. Computed load-displacement hysteretic behavior for the one-tie braced frame

In Fig.8 the relevant diagramme of the load-top displacement hysteretic behavior is shown. As this figure shows, when the loading is in the direction CD, the cable is activated and the system stiffness is increasing about 4.25 times. On the contrary, when the loading is in the inverse direction DC, then the cable becomes slack and so inactivated, and the system stiffness is decreasing and approaching the one of the bare frame.

## 4 CONCLUSIONS

The seismic inelastic behavior of planar RC frames strengthened by cable elements has been numerically investigated by the herein presented numerical approach. This approach has been calibrated by using available experimental results concerning the case of a single-bay one-storey RC frame, which was subjected to a recycling additive lateral loading in displacement-controlled mode.

The presented numerical approach can be effectively used for the seismic assessment and strengthening of existing RC structures by cable-elements. This holds for general dynamic excitations, as well as for sequential strong ground motions. For the last case of multiple earthquakes, the proposed numerical procedure has been already applied in [27]. Finally, on the basis of computed damage indices, an optimal cable-bracing scheme can be selected among investigated alternative ones by realizing a parameter sensitivity analysis concerning the cable characteristics.

## REFERENCES

[1] Chalioris, C. E., Favvata, M. J., & Karayannis, C. G. (2008). *Reinforced concrete beam–column joints with crossed inclined bars under cyclic deformations*. Earthquake Engineering & Structural Dynamics, 37(6), 881-897.

[2] Karayannis, C. G., Chalioris, C. E., & Sirkelis, G. M. (2008). *Local retrofit of exterior RC beam–column joints using thin RC jackets-An experimental study*. Earthquake Engineering & Structural Dynamics, 37(5), 727-746.

[3] Massumi, A. and A.A. Tasnimi, (2008). *"Strengthening of low ductile reinforced concrete frames using steel x-bracing with different details"*, in: Proceedings of the Fourteen World Conference on Earthquake Engineering (14WCEE), Beijing, China, 2008.

[4] Massumi, A., & Absalan, M. (2013). *"Interaction between bracing system and moment resisting frame in braced RC frames".* Archives of Civil and Mechanical Engineering, 13(2), 260-268.

[5] Antonopoulos, T.A., S.A. Anagnostopoulos, F.N. Tsirlis, (2008). *Evaluation of the seismic performance of old reinforced concrete buildings with an open ground storey, simple retrofitting solutions.* Proceedings of the 3rd Greek Conference on Earthquake Engineering and Engineering Seismology, 2061, Athens, Greece, 2008. (In Greek)

[6] Antonopoulos, T.A., S.A. Anagnostopoulos (2013), *Improving the seismic performance of existing old pilotis type buildings by strengthening only the ground story*, In: M. Papadrakakis, V. Papadopoulos & V. Plevris (eds.), *COMPDYN 2013, ECCOMAS: Thematic Conference on Computational Methods in Structural Dynamics and Earthquake Engineering*, Kos Island, Greece, 12–14 June 2013.

[7] Bertero, V. V., & Whittaker, A. S. (1989). *Seismic upgrading of existing buildings*, 5as Jornadas Chilenas de Sismología e Ingeniería Antisísmica, 1, 27-46.

[8] Penelis, G. G. & Kappos, A. J. (2010). *Earthquake resistant concrete structures*. CRC Press, London.

[9] Dritsos, S. E. (2001). *Repair and strengthening of reinforced concrete structures* (in greek). University of Patras, Greece.

[10] Fardis, M. N. (2009). *Seismic design, assessment and retrofitting of concrete buildings: based on EN-Eurocode 8*. Springer, Berlin.

[11] Penelis, G. Gr. & Penelis, Gr. G. (2014). *Concrete Buildings in Seismic Regions*. CRC Press.

[12] Tegos, I., G. Liakos, S. Tegou, G. Roupakias, K. Stilianidis, (2009). *An alternative proposal for the seismic strengthening of existing R/C buildings through tension-ties*, Proceedings, 16th Pan- Hellenic Concrete Conference, Cyprus, 2009.

[13] Markogiannaki, O., & Tegos, I. (2011). *Strengthening of a Multistory R/C Building under Lateral Loading by Utilizing Ties.* Applied Mechanics and Materials, 82, 559-564.

[14] Priestley, M. J. N., Seible, F. C., & Calvi, G. G.. M.(1996). *"Seismic Design and Retrofit of Bridges"*. John Wiley & Sons, Inc..

[15] Liolios Ang., Chalioris C., et al (2012). *"A Computational Approach for the Earthquake Response of Cable-braced Reinforced Concrete Structures under Environmental Actions".* Lecture Notes in Computer Science, LNCS, vol. 7116, pp. 590-597, Springer-Verlag, Berlin Heidelberg.

[16] Panagiotopoulos, P.D., (1993). *Hemivariational Inequalities. Applications in Mechanics and Engineering*. Springer-Verlag, Berlin, New York, (1993).

[17] Mistakidis, E.S. & Stavroulakis, G.E., (1998). *Nonconvex optimization in mechanics. Smooth and nonsmooth algorithmes, heuristic and engineering applications*. Kluwer, London.

[18] EC8-Part 3, Eurocode 8: *Design of structures for earthquake resistance, Part 3: Assessment and retrofitting of buildings.* European Standard EN 1998-3:2005. Comite Europeen de Normalisation. Brussels, 2005.

[19] EC8, Eurocode 8: *Design of structures for earthquake resistance, Part 1: General rules, seismic actions and rules for buildings*, European Standard EN 1998-1:2004. Comite Europeen de Normalisation, Brussels, 2004.

[20] EAK, (2000) *Greek Seismic Design Code*. Greek Organization for Seismic Planning and Protection (OASP), Greek Ministry for Environmental Planning and Public Works, Athens, Greece, 2003. (In Greek): www.oasp.gr

[21] FEMA (Federal Emergency Management Agency) (2000) *FEMA 356: Pre-standard and commentary for the seismic rehabilitation of buildings*, FEMA, Washington, DC.

[22] KAN.EPE., Greek Retrofitting Code. Greek Organization for Seismic Planning and Protection (OASP), Greek Ministry for Environmental Planning and Public Works. Athens, Greece, 2012. (In Greek): www.oasp.gr.

[23] Hatzigeorgiou G. and A. A. Liolios, , (2010). Nonlinear behaviour of RC frames under repeated strong ground motions. *Soil Dynamics and Earthquake Engineering*, vol. 30, 1010-1025, 2010.

[24] Carr A.J., (2008). *“RUAUMOKO - Inelastic Dynamic Analysis Program”.* Department of Civil Engineering, University of Canterbury, Christchurch, New Zealand.

[25] Chopra, A.K., (2007) . *Dynamics of Structures: Theory and Applications to Earthquake Engineering*, Pearson Prentice Hall, New York.

[26] Vamvatsikos, D. and C.A, Cornell, (2002). Incremental dynamic analysis. *Earthquake Engineering and Structural Dynamics*, vol. 31, 491–514.

[27] Liolios, Ang., Hatzigeorgiou, G. & Liolios, Ast., (2013). “A numerical approach for estimating the effects of multiple earthquakes to seismic response of structures strengthened by cable-elements”. *Journal of Theoretical and Applied Mechanics*, 43(3), 21-32.

[28] Park Y.J. and A.H.S. Ang, (1985). Mechanistic seismic damage model for reinforced concrete, *Journal of Structural Division ASCE*, vol. 111(4), 722–739.

[29] Kunnath, S.K., Reinhorn AM, Lobo RF, (1992). IDARC version 3.0: a program for the inelastic damage analysis of reinforced concrete structures. *Technical report NCEER-92-0022*. National Center for Earthquake Engineering Research, Buffalo, NY; 1992.

[30] Kappos, A. J. (1997). Seismic damage indices for RC buildings: evaluation of concepts and procedures. *Progress in Structural Engineering and Materials*, 1(1), 78-87.

[31] Mergos, P. E., & Kappos, A. J. (2013). A combined local damage index for seismic assessment of existing RC structures. *Earthquake Engineering & Structural Dynamics*, 42(6), 833-852.

[32] Mitropoulou, C. C., Lagaros, N. D., & Papadrakakis, M. (2014). Numerical calibration of damage indices. *Advances in Engineering Software*, 70, 36-50.

[33] Elenas, A., Liolios, Ast., Vasiliadis, L., Favvata, M. & Liolios, Ang., (2012). Seismic intensity parameters as damage potential descriptors for life-cycle analysis of buildings. *IALCCE 2012: Proc. Third Int. Symp. Life-Cycle Civil Engineering*, Vienna, Austria, October 2012, CRC Press, Taylor & Francis, London.

[34] Paulay T. and M.J.N. Priestley, (1992), *Seismic Design of Reinforced Concrete and Masonry Buildings*, Wiley, New York.

[35] Maier, G., (1971). Incremental Elastoplastic Analysis in the Presence of Large Displacements and Physical Instabilizing Effects, *Int. Jnl Solids and Structures*, Vol. 7, 345-372.

[36] PEER (2011). Pacific Earthquake Engineering Research Center. *PEER Strong Motion Database*. http://peer.berkeley.edu/smcat. Last access: 2011/02/15.

[37] Kakaletsis, D. J., & Karayannis, C. G. (2009). Experimental investigation of infilled reinforced concrete frames with openings. *ACI Structural Journal*, 106(2), pp. 132-141.