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POUNDING EFFECTS ON THE EARTHQUAKE RESPONSE OF ADJACENT REINFORCED CONCRETE STRUCTURES STRENGTHENED BY CABLE ELEMENTS*

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Dedicated to Professor Konrad BERGMEISTER on the occasion of his 20th anniversary of activity at BOKU, the University of Natural Resources and Life Sciences, Vienna, Austria.

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Abstract. A numerical approach for estimating the effects of pounding (seismic interaction) on the response of adjacent Civil Engineering structures is presented. Emphasis is given to reinforced concrete (RC) frames of existing buildings which are seismically strengthened by cable-elements. A double discretization, in space by the Finite Element Method and in time by a direct incremental approach is used. The unilateral behaviours of both, the cable-elements and the interfaces contact-constraints, are taken strictly into account and result to inequality constitutive conditions. So, in each time-step, a non-convex linear complementarity problem is solved. It is found that pounding and cable strengthening have significant effects on the earthquake response and, hence, on the seismic upgrading of existing adjacent RC structures.

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Key words: Computational structural mechanics, earthquake engineering, pounding (seismic interaction) effects, strengthening by cable elements, unilateral problems.

1. Introduction

Pounding in earthquake engineering is one of the non-usual extremal actions (seismic, environmental etc.), which can cause significant strength degradation and damages on civil engineering structures [1]. Pounding concerns the seismic interaction between adjacent structures, e.g. neighbouring buildings in city centers constructed in contact. On the common contact interface, during an earthquake excitation, appear at each time-moment either compressive stresses or relative removal displacements (separating gaps) only [1–5]. These requirements result to inequality conditions in the mathematical problem formulation.

To overcome strength degradation effects, various repairing and strengthening procedures can be used for the seismic upgrading of existing buildings [6, 7, 36]. Among them, cable-like members (ties) can be used as a first strengthening and repairing procedure [8–10]. These cable-members can undertake tension, but buckle and become slack and structurally ineffective when subjected to a sufficient compressive force. So, in the mathematical problem formulation, the constitutive relations for cable-members are also inequality conditions.

Due to above considerations, the full problem of the earthquake response of pounding adjacent structures strengthened by cable-elements bracings has as governing conditions both, equalities as well as inequalities. Thus, the problem becomes a high nonlinear one. For the strict mathematical treatment of the problem, the concept of variation and/or hemivariation inequalities can be used and has been successfully applied [11, 12]. As concerns the numerical treatment, non-convex optimization algorithms are generally required [11–15].

The present study deals with two numerical approaches for the earthquake analysis of existing adjacent reinforced concrete (RC) building frames, which can come in unilateral contact and have to be strengthened by cable elements. The unilateral behaviours of both, the cable-elements and the interfaces contact-constraints, are taken strictly into account and result to inequality constitutive conditions. The finite element method is used for space discretization in combination with a time discretization scheme. In a mathematically strict first approach, optimization algorithms are used and in each time-step, a nonconvex linear complementarily problem is solved. In an alternative incremental second procedure, use is made of the Ruaumoko structural engineering software [16]. Both procedures aim mainly to practical applications in civil engineering.

The investigation purpose is to compare various cable-bracing strengthening versions for existing RC structures, in order the optimum one to be chosen.

2. Method of analysis

2.1. Problem formulation

A double discretization, in space and time, is applied as it is currently usual in structural dynamics for civil engineering structures [17]. First, the structural system of the two adjacent RC frames (A) and (B) is discretized in space by using finite elements. The usual frame elements are used for the reinforced concrete frames. Unilateral constrained elements are used as concerns the interfaces of unilateral contact, where pounding is expected to take place.

Pin-jointed bar elements are used on the other hand, for the cable strengthening system. The behaviour of both, the cable elements and unilateral contact elements, includes loosening, elastoplastic or/and elastoplastic-softening-fracturing and unloading - reloading effects. All these characteristics concerning both constitutive laws, on the one hand of the cable elements and on the other hand of the unilateral contact elements, can be expressed mathematically by non-convex relations of the the general form:

$$s_i(d_i) \in \widehat{\partial} S_i(d_i). \tag{2.1}$$

Here s_i and d_i are generalized stress and deformation quantities, respectively, $\widehat{\partial}$ is the generalized gradient and S_i is the superpotential function, see Panagiotopoulos [11] and Stavroulakis [12]. In specializing details, for the cables, s_i is the tensile force (in [kN]) and d_i the deformation (elongation) (in [m]), of the i-th cable element. Similarly, concerning the unilateral contact simulation, s_i is the compressive force p_i (in [kN]) and d_i the deformation (shortening) (in [m]), of the i-th unilateral constraint element.

By definition – see [11] – the relation (2.1) is equivalent to the following hemivariation inequality, expressing the Virtual Work Principle:

$$S_i^{\uparrow}(d_i, e_i - d_i) \geq s_i(d_i) \cdot (e_i - d_i), \tag{2.2}$$

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

Next, dynamic equilibrium for the two frames (A) and (B) of structural system, considered as uncoupled and unstrengthened, i.e. without pounding and without cables, is expressed by the usual matrix relations of Structural

Dynamics [17]:

$$\mathbf{M_A}\ddot{\mathbf{u}}_\mathbf{A} + \mathbf{C_A}(\dot{\mathbf{u}}_\mathbf{A}) + \mathrm{K}_\mathbf{A}(\mathbf{u_A}) = \mathbf{f_A}, \tag{2.3a}$$

$$\mathbf{M_B}\ddot{\mathbf{u}}_\mathbf{B} + \mathbf{C_B}(\dot{\mathbf{u}}_\mathbf{B}) + \mathbf{K_B}(\mathbf{u_B}) = \mathbf{f_B}. \tag{2.3b}$$

Here $\mathbf{u}$ and $\mathbf{f}$ are the displacement and the loading forces time dependent vectors, respectively. The damping and stiffness terms, $\mathbf{C}(\dot{\mathbf{u}})$ and $\mathbf{K}(\mathbf{u})$, respectively, concern the general non-linear case. When the linear-elastic case holds, these terms have the usual simple forms $\mathbf{C}\dot{\mathbf{u}}$ and $\mathbf{Ku}$. Dots over symbols denote derivatives with respect to time. For the case of ground seismic excitation $\mathbf{x}_g$, the loading history term $\mathbf{f}$ becomes:

$$\mathbf{f} = -\mathbf{Mr}\ddot{\mathbf{x}}_g, \tag{2.4}$$

where $\mathbf{r}$ is the vector of stereostatic displacements.

When both, cable-elements and pounding are taken into account, equations (2.3) for the assembled system of the two frames (A) and (B), considered as coupled and strengthened, become:

$$\mathbf{M_A}\ddot{\mathbf{u}}_\mathbf{A} + \mathbf{C_A}(\dot{\mathbf{u}}_\mathbf{A}) + \mathbf{K_A}(\mathbf{u_A}) = \mathbf{f_A} + \mathbf{T_A}\mathbf{s_A} + \mathbf{Bp}, \tag{2.5a}$$

$$\mathbf{M_B}\ddot{\mathbf{u}}_\mathbf{B} + \mathbf{C_B}(\dot{\mathbf{u}}_\mathbf{B}) + \mathbf{K_B}(\mathbf{u_B}) = \mathbf{f_B} + \mathbf{T_B}\mathbf{s_B} - \mathbf{Bp}, \tag{2.5b}$$

$$\mathbf{p} = \mathbf{p_N} + \mathbf{p_T}. \tag{2.5c}$$

Here, $\mathbf{s_A}$ and $\mathbf{s_B}$ are the cable elements stress vectors for frames (A) and (B), respectively; $\mathbf{p}$ is the contact elements stress vector and $\mathbf{T_A}$, $\mathbf{T}_B$, $\mathbf{B}$ are transformation matrices. The pounding stress vector $\mathbf{p}$ is decomposed to the vectors $\mathbf{p_N}$, of the normal, and $\mathbf{p_T}$ of the tangential interaction forces between frames (A) and (B).

The above relations (2.1)–(2.5), combined with the initial conditions, provide the problem formulation, where, for given $\mathbf{f}$ and/or $\ddot{\mathbf{x}}_g$, the vectors $\mathbf{u_A}$, $\mathbf{u_B}$, $\mathbf{p}$ and $\mathbf{s_A}$, $\mathbf{s_B}$ have to be computed.

From the strict mathematical point of view, using (2.1) and (2.2) and following [11, 12], we can formulate the problem as a hemivariation inequality one and investigate it about existence and uniqueness of solution.

2.2. Problem numerical treatment by optimization methods

A numerical treatment of the problem, compatible with the above strict mathematical formulation and suitable for civil engineering practical cases, has been proposed in [14, 18] and is followed here.

The approach is fully described in [18] and is based on a piecewise linearization of the above constitutive relations as in elastoplasticity [19]. By using a time-integration scheme, in each time-step Δt a relevant non-convex linear complementarity problem of the following matrix form is eventually solved :

$$\mathbf{v} \geq \mathbf{0},\ \ \mathbf{A}\mathbf{v} + \mathbf{a} \leq \mathbf{0},\ \ \mathbf{v}^{\mathbf{T}}.(\mathbf{A}\mathbf{v} + \mathbf{a}) = 0. \tag{2.6}$$

Here, $\mathbf{v}$ is the vector of unknown unilateral quantities at the time-moment t, $\mathbf{v}^{\mathbf{T}}$ is the transpose of $\mathbf{v}$, $\mathbf{a}$ is a known vector dependent on excitation and results from previous time moments $(t - \Delta t)$, and $\mathbf{A}$ is a transformation matrix.

Thus, the detailed nonlinear Response Time-History (RTH) of the coupled system for a given seismic ground excitation can be computed by using available optimization algorithms.

In [18], the seismic response of cable-braced tall RC frames (having seven storeys) to earthquake excitation has been investigated. Relevant procedures using optimization methods have been also presented in [11-15] for unilateral contact problems. In the present analysis, the effects of both, pounding and cables-strengthening, are taking into account.

2.3. Incremental numerical treatment by Ruaumoko software

An alternative approach for treating numerically the problem is the incremental one. Now, relations (2.5), taking into account also second-order geometric effects (P-Delta effects), are written in incremental form:

$$\begin{aligned}\mathbf{M_A}\Delta\ddot{\mathbf{u}}_{\mathbf{A}} + \mathbf{C_A}\Delta\dot{\mathbf{u}}_{\mathbf{A}} + (\mathbf{K_A} + \mathbf{G_A})\Delta\mathbf{u_A} \\ = -\mathbf{M_A}\Delta\ddot{\mathbf{u}}_{\mathbf{g}} + \mathbf{T_A}\Delta\mathbf{s_A} + \mathbf{B}\Delta\mathbf{p},\end{aligned} \tag{2.7a}$$

$$\begin{aligned}\mathbf{M_B}\Delta\ddot{\mathbf{u}}_{\mathbf{B}} + \mathbf{C_B}\Delta\dot{\mathbf{u}}_{\mathbf{B}} + (\mathbf{K_B} + \mathbf{G_B})\Delta\mathbf{u_B} \\ = -\mathbf{M_B}\Delta\ddot{\mathbf{u}}_{\mathbf{g}} - \mathbf{T_B}\Delta\mathbf{s_B} - \mathbf{B}\Delta\mathbf{p}.\end{aligned} \tag{2.7b}$$

Here, $\mathbf{G_A}$ and $\mathbf{G_B}$ are the geometric stiffness matrices, by which P-Delta effects are taken into account [18].

The structural analysis software Ruaumoko [16] is based on such incremental approaches. Ruaumoko software uses the finite element method and permits an extensive parametric study on the inelastic response of structures. For practical applications, an efficient library of hysteretic behaviour models is available. Concerning the time-discretization, implicit or explicit approaches can be used. Here, the Newmark implicit scheme is chosen and Ruaumoko is used to provide results which are related to the following critical parameters: local or global structural damage, maximum displacements, interstorey drift ratios, development of plastic hinges and response using the incremental dynamic analysis (IDA) method [20].

Details of the approach are described in [21], where the seismic response of cable-braced RC systems to multiple earthquakes is investigated. As concerns multiple earthquakes, it is reminded that current seismic codes [22-24] 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 in seismically active regions, because, as it is shown in [25–27], real seismic sequences have accumulating effects on various damage indices.

2.4. Comparative investigations for the cable-strengthening versions

The decision about a possible strengthening for an existing structural system of interacting structures, damaged by a seismic event, can be taken after a relevant evaluation of suitable damage indices. This is obtained by structural assessment and reliability analysis for the existing engineering structures based on identification [28–30]. For this purpose, a comparative investigation of structural responses due to various seismic excitations can be used. So, the system is considered for various cases: with or without pounding, with or without strengthening by cable-bracings, and combinations of them.

The focus is on the overall structural damage index (OSDI) [31–33] among the several response parameters. This is due to the fact, that this parameter summarises all the existing damages on columns and beams of reinforced concrete frames in a single value, which is useful for comparison reasons.

In the OSDI model after Park/Ang [32, 33], the global damage is obtained as a weighted average of the local damage at the section ends of each structural element or at each cable element. First, the local damage index DI_L is computed by the following relation:

$$DI_L = \frac{\mu_m}{\mu_u} + \frac{\beta}{F_y d_u} E_T, \tag{2.8}$$

where: μ_m is the maximum ductility attained during the load history, μ_u the ultimate ductility capacity of the section or element, β a strength degrading parameter, F_y the yield force of the section or element, E_T the dissipated hysteretic energy, and d_u the ultimate deformation.

Next, the dissipated energy E_T is chosen as the weighting function and the global damage index DI_G is computed by using the following relation:

$$DI_G = \frac{\sum_{i=1}^{n} DI_{Li} E_i}{\sum_{i=1}^{n} E_i}, \tag{2.9}$$

where: DI_{Li} is the local damage index after Park/Ang at location i, E_i is the energy dissipated at location i and n is the number of locations at which the local damage is computed.

3. Numerical example

3.1. Description of the considered RC structural system

The system of the two reinforced concrete frames (A) and (B) in Fig. 1 is considered. The frames are of concrete class C40/45, and have been designed according to Greek building codes and to current European seismic codes [22–24]. The beams are of rectangular section 30/60 (width/height, in cm), with section inertia moment I_B and have a total vertical distributed load 30 KN/m (each beam). The columns, with section inertia moment I_C, possess section dimensions, in cm: 30/30 for frame (A) and 40/40 for the frame (B). The frames are parts of two adjacent buildings, which initially were designed and constructed independently in different time periods. Due to connections shown in Fig. 1, pounding is expected to take place on columns FK (point G_1) and LN (point G_2) of frames (A) and (B), respectively. The gaps on G_1 and G_2 are taken initially as zero.

The system of the seismically interacting RC frames (A) and (B) has been subjected to various extremal actions (seismic, environmental etc.). So, corrosion and cracking have been taken place, which have caused a strength and stiffness degradation. The so resulted reduction for the section inertia moments I_C and I_B was estimated [34] to be 10% for the columns and 50% for the beams.

To overcome the above degradation, various strengthening schemes by cable-elements can be investigated. These schemes are here denoted as CISJ, where I is the number of the contacts and J is the number of the bracing-cables which are taken into account. So, the frame system of Fig. 1 is denoted as C2S0

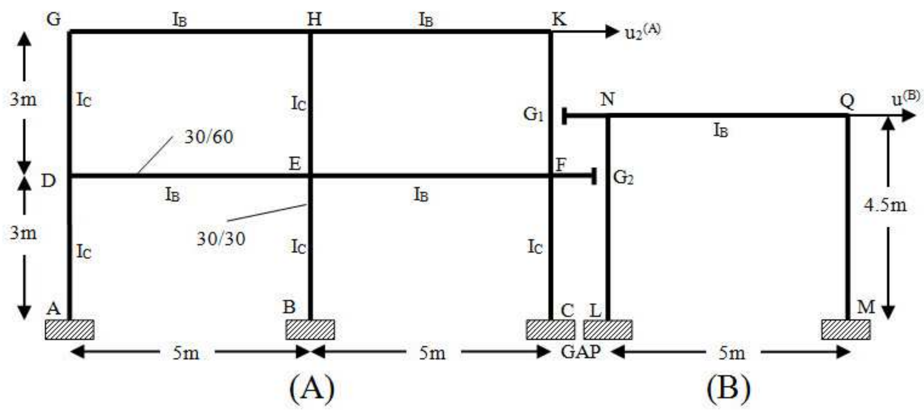

Fig. 1. The initial system of the RC frames (A) and (B), without cable-strengthening and with two possible unilateral contacts on G_1 and G_2 when the contacts are taken into account, the system is denoted as C2S0

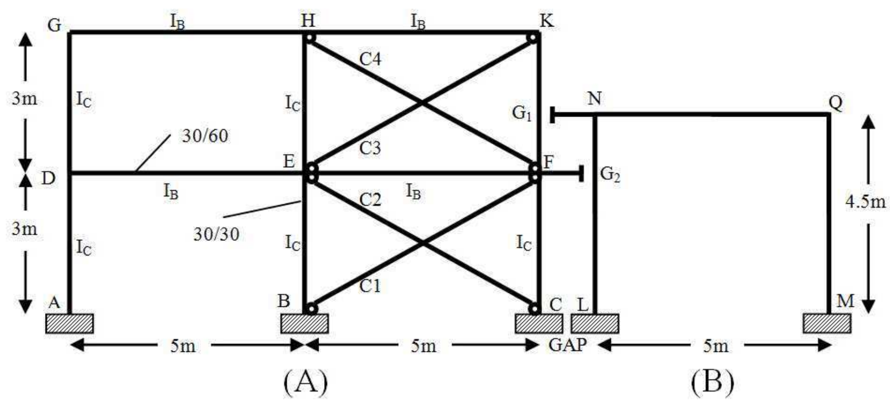

Fig. 2. The C2S4 system with diagonal cable-strengthening for frame (A) only

when the two possible unilateral contacts on G_1 and G_2 are taken into account as activated ones, whereas no strengthening by cable-bracings is considered. When both, unilateral contacts and cable-bracings are not taken into account, the system is denoted as C0S0.

Two cable-bracing systems, shown in Fig. 2 and Fig. 3, have been in-

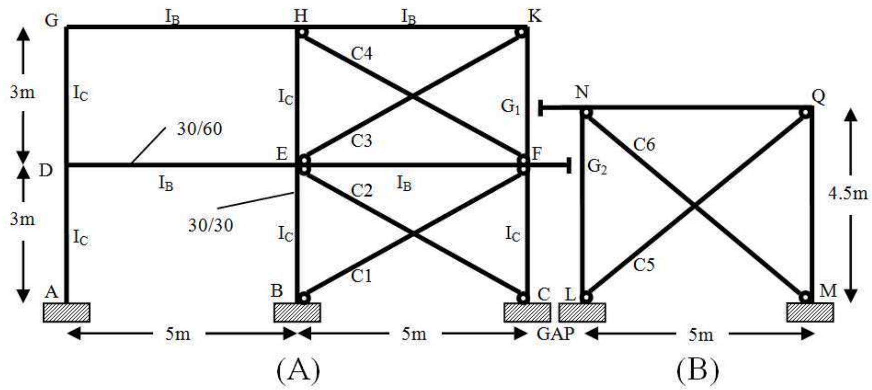

Fig. 3. The C2S6 system with diagonal cable-strengthening for both frames (A) and (B)

vestigated in order to upgrade seismically the damaged frame system C2S0. The purpose was to choose the optimal one. The first cable-bracing system in Fig. 2, with 4 cable-elements in frame (A), is denoted as C2S4. The second cable-bracing system in Fig. 3, denoted as C2S6, has X-bracing 6 diagonal cable-elements in the two frames (A) and (B).

The cable elements have a cross-sectional area $F_c = 18cm^2$ and they are of steel class S220 with yield strain $\varepsilon_y = 0.2\%$, fracture strain $\varepsilon_f = 2\%$ and elasticity modulus $E_c = 200$ GPa. The cable constitutive law, relevant to the piece-wise linearized form of eq. (2.1), is depicted in Fig. 4. The general unilateral (slackness), hysteretic, fracturing, unloading-reloading etc. behaviour of cable-elements is shown. So, segments UO and OA concern the slack-linear elastic behaviour, according to which the cables can not undertake compressive stresses (branch UO). Segments AB and DC concern the plastic and fracture behaviour, respectively. The cable yield resistance is $S_y = 438.3$ kN. The yield deformation is $d_y = 11.7$ mm for cables C1 – C4 in frame (A), and $d_y = 13.5$ mm for cables C5 – C6 in frame (B). Denoting the ultimate cable elongation by d_u, the cables ductility is $d_u/d_y = 10$. Paths FQF or FPRPF concern unloading-reloading cases.

3.2. Earthquakes input

The systems C2S0, C2S4 and C2S6 in Figs 1–3 are considered to be subjected to a multiple ground seismic excitation presented in the paper [21]. The

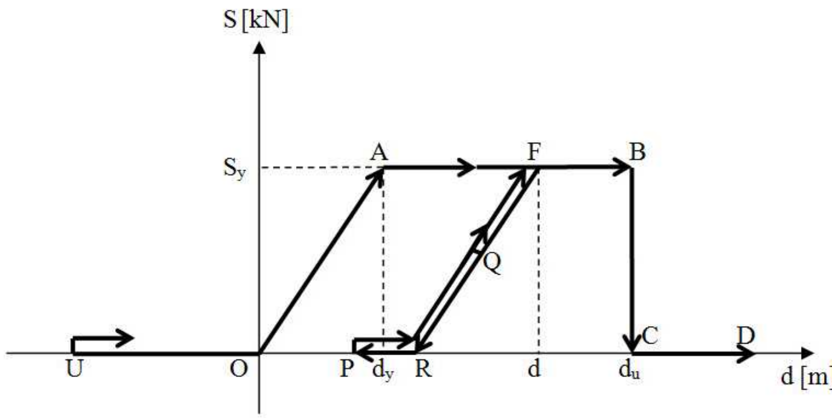

Fig. 4. The constitutive law of cable-elements

complete list of these earthquakes, which were downloaded from the strong motion database of the Pacific Earthquake Engineering Research (PEER) Center [35], appears in Table 1.

The strong ground motion database consists of five real seismic sequences, which have been recorded during a short period of time (up to three days), by the same station, in the same direction, and almost at the same fault distance. These seismic sequences are namely: Mammoth Lakes (May 1980 – 5 events), Chalfant Valley (July 1986 – 2 events), Coalinga (July 1983 – 2 events), Imperial Valley (October 1979 – 2 events) and Whittier Narrows (October 1987 – 2 events) earthquakes. For more details, see [21].

3.3. Representative results

Representative results, concerning only the isolated Coalinga case of Recorded PGA = 0.605 g shown in Table 1., are presented in next Table 2. The investigated response quantities are shown in table column (1). For comparison reasons, results concerning the uncoupled case C0S0, when pounding and cable strengthening are not taken into account, are shown in column (2). The results concerning the numerical investigation for the systems C2S0, C2S4 and C2S6 are shown in columns (3), (4) and (5), respectively.

The representative response quantities shown in column (1) in Table 2, are:

- $u_2^{(A)}$ and $u_1^{(A)}$ the absolutely maximum horizontal displacements of the second (GHK) and first (DEF) floor, respectively, of frame (A) – see Fig. 1.

Table 1. Sequential earthquakes data

No	Seismic sequence	Station	Comp	Date (Time)	Magnitude (M_L)	Recorded PGA(g)	Normalized PGA(g)
1	Mammoth Lakes	54099 Convict Creek	N-S	1980/05/25 (16:34) 1980/05/25 (16:49) 1980/05/25 (19:44) 1980/05/25 (20:35) 1980/05/27 (14:51)	6.1 6.0 6.1 5.7 6.2	0.442 0.178 0.208 0.432 0.316	0.200 0.081 0.094 0.195 0.143
2	Chalfant Valley	54428 Zack Brothers Ranch	E-W	1986/07/20 (14:29) 1986/07/21 (14:42)	5.9 6.3	0.285 0.447	0.128 0.200
3	Coalinga	46T04 CHP	N-S	1983/07/22 (02:39) 1983/07/25 (22:31)	6.0 5.3	0.605 0.733	0.165 0.200
4	Imperial Valley	5055 Holtville P.O.	HPV 315	1979/10/15 (23:16) 1979/10/15 (23:19)	6.6 5.2	0.221 0.211	0.200 0.191
5	Whittier Narrows	24401 San Marino	N-S	1987/10/01 (14:42) 1987/10/04 (10:59)	5.9 5.3	0.204 0.212	0.192 0.200

- $u^{(B)}$ the absolutely maximum horizontal floor (NQ) displacement of frame (B) – see Fig. 1.
- DI_G is the global damage index.
- DI_{BE} and DI_{FE} are the local damage indices for bending behaviour at the ends B and F of the column BE and of the beam FE, respectively, of frame (A).
- DI_{G1F} and DI_{G1K} are the local damage indices for bending behaviour in the column FK of frame (A).
- DI_{G2L} and DI_{G2N} are the local damage indices for bending behaviour in the column LN of frame (B).
- FG_2 and NG_1 are the maximum (absolutely) compressive forces (in [kN]) developed on unilateral contacts G_2 and G_1.
- SC1 to SC6 are the maximum tension forces (in [kN]) developed in the cable elements.

As the table values of the above response quantities show, pounding – see column (3) for system C2S0- generally increases the maximum values of response quantities of the uncoupled system C0S0. On the other hand, the strengthening by X-bracings of both frames (A) and (B) – see column (5) for system C2S6- improves the response values in comparison to strengthening by X-bracings of only frame (A) – see column (4) for system C2S4. This is concluded by comparing the corresponding values of the damage indices, especially that of the global damage index DI_G. The values SC2 = 438.3 kN concerning the maximum tension force of the cable-element C2 for the cases C2S4 and C2S6 are equal to yield resistance S_y, and thus show that this element is overstressed beyond the linear elastic limits.

In conclusion, the table values prove that the system C2S6 in Fig. 3, i.e. strengthening by X-bracings of both frames with two pounding contacts, is the optimal one for seismic upgrading the damaged system C2S0.

Table 2. Representative results for the Coalinga case of Recorded PGA = 0.605 g

Response Quantities	C0S0	C2S0	C2S4	C2S6
(1)	(2)	(3)	(4)	(5)
$u_2^{(A)}$ [mm]	27.62	29.78	16.67	14.08
$u_1^{(A)}$ [mm]	19.63	17.04	10.21	8.01
$u^{(B)}$ [mm]	20.84	23.79	16.08	6.45
DI_G	0.114	0.132	0.092	0.068
DI_{BE}	0.191	0.248	0.142	0.117
DI_{FE}	0.605	0.627	0.381	0.171
DI_{G1F}	0.102	0.186	0.127	0.018
DI_{G1K}	0.088	0.197	0.170	0.017
DI_{G2L}	0.137	0.183	0.102	0.005
DI_{G2N}	0.159	0.149	0.180	0.007
FG_2 [kN]	0	−327.8	−258.5	−232.4
NG_1 [kN]	0	−455.2	−386.1	−371.1
SC1 [kN]	–	–	+312.1	+271.8
SC2 [kN]	–	–	+438.3	+438.3
SC3 [kN]	–	–	+343.4	+322.8
SC4 [kN]	–	–	+285.1	+274.9
SC5 [kN]	–	–	–	+252.4
SC6 [kN]	–	–	–	+232.2

4. Concluding remarks

The inelastic seismic behaviour of adjacent existing RC frames, strengthened by cable elements, has been numerically investigated by the herein presented numerical approach. Pounding effects and the unilateral behaviour of cable-elements are strictly taken into account. As the results of a numerical example have shown, the optimal strengthening version of cable-bracings can be decided by computing necessary damage indices.

Generally it is concluded that pounding has significant effects on the earthquake response of adjacent structure. Hence, cable strengthening can be effectively used for the seismic upgrading of existing adjacent RC structures.

The herein presented approach can be also used for a detailed parametric study of pounding buildings under multiple earthquakes. This case generally indicates the need for strengthening, because increased displacement demands are required in comparison to single seismic events. Furthermore, the seismic damage for multiple earthquakes is higher than that for single ground motions. These characteristics, computed by the herein approach, are very important and should be taken into account for the seismic design of new and the repairing and strengthening of existing structures. The case of multiple earthquakes for existing structures, seismically interacting (pounding) and strengthened by cable-tie elements, is now under investigation. Relevant research results, concerning also the case of tall buildings (having storeys more than two), will be presented in a next paper.

REFERENCES

[1] Bertero, V. V. Observations on Structural Pounding, Proc. Intern. Conf. The Mexico Earthquakes, New York, ASCE, 1987, 264–278.

[2] Anagnostopoulos, S., K. Spiliopoulos. An Investigation of Earthquake induced Pounding between Adjacent Buildings. *Earthquake Engineering and Structural Dynamics*, **21** (1992), 289–302.

[3] Karayannis, C. G., M. J. Favvata. Earthquake-induced Interaction between Adjacent Reinforced Concrete Structures with Non-equal Heights. *Earthquake Engineering and Structural Dynamics,* **34** (2005), No. 1, 1–20.

[4] Jankowski, R. Non-linear FEM Analysis of Pounding-involved Response of Buildings under Non-uniform Earthquake Excitation. *Eng. Struct.*, **37** (2012), 99–105.

[5] Mahmoud, S., A. Abd-elhamed, R. Jankowski. Earthquake-Induced Pounding between Equal Height Multi-storey Buildings Considering Soil-structure Interaction, Bulletin of Earthquake Engineering, 2013, 1–28.

[6] Bertero, V. V., A. S. Whittaker. Seismic Upgrading of Existing Buildings. *5as Jornadas Chilenas de Sismología e Ingeniería Antisísmica*, **1** (1989), 27–46.

[7] Dritsos, S. E. Repair and Strengthening of Reinforced Concrete Structures, Greece, University of Patras, 2005, (in Greek).

[8] Tegos, I., G. Liakos, S. Tegou, G. Roupakias, K. Stilianidis. An Alternative Proposal for the Seismic Strengthening of Existing R/C Buildings through Tensionties, Proceedings, 16th Pan-Hellenic Concrete Conference, Cyprus, 2009, vol. II, 1–18.

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

[10] Stavroulaki, M. E., G. E. Stavroulakis, B. Leftheris. Modelling Prestress Restoration of Buildings by General Purpose Structural Analysis Optimization Software. *Computers and Structures*, **62** (1997), No. 1, 81–92.

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

[12] Stavroulakis, G. E. Computational Nonsmooth Mechanics: Variational and Hemivariational Inequalities. *Nonlinear Analysis. Theory, Methods and Applications*, **47** (2001), No. 8, 5113–5124.

[13] Mistakidis, E. S., G. E. Stavroulakis. Nonconvex Optimization in Mechanics. Smooth and Nonsmooth Algorithmes, Heuristic and Engineering Applications, London, Kluwer, 1998.

[14] Liolios, A. A Linear Complementarity Approach for the Non-convex Seismic Frictional Interaction between Adjacent Structures under Instabilizing Effect. *Journal of Global Optimization*, **17** (2000), 259–266.

[15] Liolios, A., A. Boglou. Chaotic Behaviour in the Non-linear Optimal Control of Unilaterally Contacting Building Systems during Earthquakes. *Chaos, Solitons & Fractals,* **17** (2003), 493–498.

[16] Carr, A. J. Ruamoko: Inelastic Dynamic Analysis Program, New Zealand, Christchurch, University of Canterbury, Department of Civil Engineering, 2008.

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

[18] Liolios, Ang., K. Chalioris, Ast. Liolios, St. Radev, Kon. Liolios. A Computational Approach for the Earthquake Response of Cable-braced Reinforced Concrete Structures under Environmental Actions. In: Margenov, S. et al (Eds): Large-Scale Scientific Computations. Berlin Heidelberg, Springer-Verlag, *LNCS*, **7116** (2012), 590–597.

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

[20] Vamvatsikos, D., C. A. Cornell. Incremental Dynamic Analysis. *Earthquake Engineering and Structural Dynamics*, **31** (2002), 491–514.

[21] Liolios, Ang., Ast. Liolios, G. Hatzigeorgiou. 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** (2013), No. 3, 21–32.

[22] EC1: EN 1991-1-1. Eurocode 1: Actions on Structures, Part 1-1: General Actions-densities, Self-weight, Imposed Loads for Buildings, European Committee for Standardization, Brussels, 2002.

[23] EC2: EN 1992-1-1. Eurocode 2: Design of Concrete Structures, Part 1-1: General Rules and Rules for Buildings, European Committee for Standardization, Brussels, 2004.

[24] EC8: EN 1998-1. Eurocode 8: Design of Structures for Earthquake Resistance; Part 1: General Rules, Seismic Actions and Rules for Buildings, European Committee for Standardization, Brussels, 2005.

[25] Hatzigeorgiou, G., A. A. Liolios. Nonlinear Behaviour of RC Frames under Repeated Strong Ground Motions. *Soil Dynamics and Earthquake Engineering*, **30** (2010), 1010–1025.

[26] Efraimiadou, S., G. Hatzigeorgiou, A. Liolios. Inelastic Pounding of Adjacent Reinforced Concrete Structures under Multiple Earthquakes, in: Boudouvis, A. and Stavroulakis, G. (eds), Proceedings of 7th GRACM Intern. Conf. Computational Mechanics, paper No. 19, Athens, 30.6.–2.7.2011.

[27] Efraimiadou, S., G. D. Hatzigeorgiou, D. E. Beskos. Structural Pounding Between Adjacent Buildings Subjected to Strong Ground Motions. Part I: The Effect of Different Structures Arrangement. *Earthquake Engng. Struct. Dyn.*, **42** (2013), No. 10, 1509–1528.

[28] Strauss, A., D. M. Frangopol, K. Bergmeister. Assessment of Existing Structures based on Identification. *Jnl Struct. Eng. ASCE*, **136** (2010), No. 1, 86–97.

[29] Strauss, A., K. Bergmeister, P. Pukl, V. Cervenka, D. Novak. SARA-A Tool within Life Span Analysis of Concrete Structures, In: Frangopol, D. M. et al (Eds.), Life-cycle Performance of Deteriorating Structures, ASCE Publications, 2004, 192–400.

[30] Bergmeister, K., D. Novak, R. Pukl, V. Cervenka. Structural Assessment and Reliability Analysis for Existing Engineering Structures, Theoretical Background. *Struct. Infrastruct. Eng.*, **5** (2009), No. 4, 267–275.

[31] Elenas, A., Ast. Liolios, L. Vasiliadis, M. Favvata, Ang. Liolios. Seismic Intensity Parameters as Damage Potential Descriptors for Life-cycle Analysis of Buildings. In: Strauss, A., Frangopol, D., Bergmeister, K. (Eds), Proceedings of the Third International Symposium on Life-Cycle and Sustainability of Civil Infrastructures Systems, IALCCE'12, Held in Vienna, Austria, October 3–6, 2012, London, CRC Press, Taylor & Francis, Abstracts Book, p. 130.

[32] Ang, A. H. S. Seismic Damage Assessment and Basis for Damage-Limiting Design. *Probabilistic Engineering Mechanics,* **3** (1988), 146–150.

[33] Park, Y. J., A. H. S. Ang. Mechanistic Seismic Damage Model for Reinforced Concrete. *Journal of Structural Division ASCE*, **111** (1985), No. 4, 722–739.

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

[35] Pacific Earthquake Engineering Research Center. PEER Strong Motion Database. `http://peer.berkeley.edu/smcat`.

[36] Fardis, M. N. Seismic Design Assessment and Retrofitting of Concrete Buildings, Based on EN-Eurocode 8. Berlin, Springer, 2009.