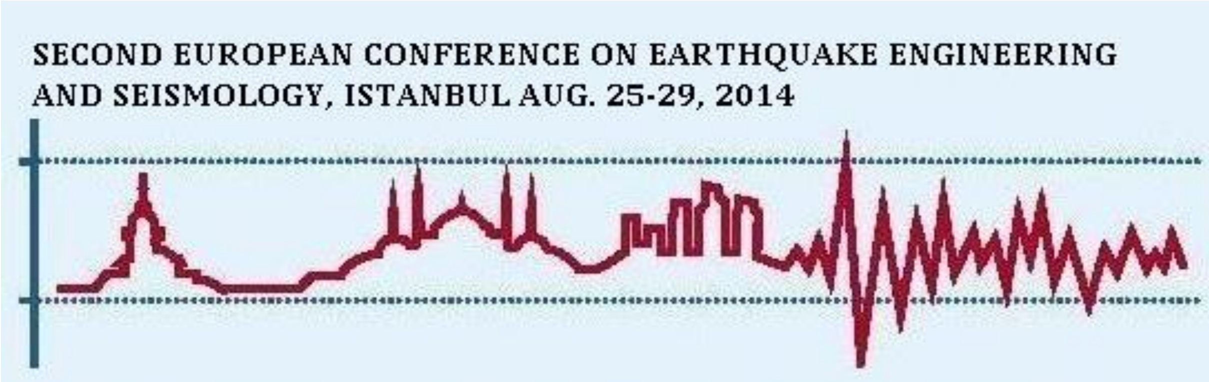

A NUMERICAL APPROACH FOR POUNDING EFFECTS ON THE RESPONSE OF ADJACENT RC STRUCTURES STRENGTHENED BY CABLE ELEMENTS UNDER MULTIPLE EARTHQUAKES

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ABSTRACT

A numerical approach for estimating the effects of seismic interaction (pounding) on the response of adjacent structures under multiple earthquakes is presented. Reinforced Concrete (RC) frames of existing buildings are considered, which after their seismic assessment are upgraded by using cable-elements . A double discretization, in space by the finite element method and in time by a direct integration approach, is used. Both, the unilateral behaviour of the cable-elements and of the contact interfaces, are strictly taken into account and result to inequality constitutive conditions. So, in each time-step, a non-convex linear complementarity problem is solved. Alternatively, an incremental approach is used by applying the Ruaumoko software. It is found that the sequences of ground motions have a significant effect on the response and, hence, on the design of adjacent reinforced concrete structures strengthened by cable-elements.

INTRODUCTION

As well known, pounding concerns the seismic interaction between adjacent structures, e.g. neighboring buildings in city centers constructed in contact. Pounding can cause significant strength degradation and damages on adjacent structures, see e.g. (Bertero, 1987), (Anagnostopoulos and Spiliopoulos, 1992), (Favvata et al, 2009), (Karayannis and Favvata, 2005), (Jankowski, 2012). On the common contact interface of the interacting structures, during an earthquake excitation, either compressive stresses or relative removal displacements (separating gaps) are developed at each time-moment. This behaviour results to inequality conditions in the mathematical problem formulation (Liolios, 1990).

To overcome such strength degradation effects, caused by pounding and generally by other environmental actions, various repairing and strengthening procedures can be used for the seismic upgrading of existing reinforced concrete (RC) buildings, see e.g. (Fardis, 2009), (Dritsos, 2005).. Among them, cable-like members (ties) can be used as a first strengthening and repairing procedure (Bertero and Whittaker, 1989), (Tegos et al, 2009), (Markogiannaki and Tegos, 2011). 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

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constitutive relations for cable-members are also inequality conditions, see (Liolios et al, 2012), (Liolios et al, 2013).

Due to above considerations, the full problem of the earthquake response of adjacent RC structures under pounding, 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 variational and/or hemivariational inequalities can be used and has been successfully applied (Panagiotopoulos, 1993). As concerns the numerical treatment, non-convex optimization algorithms are generally required (Liolios et al, 2014), .

On the other hand, an important limitation of current seismic design codes, e.g. EC8 (2005), is the exclusive adoption of the isolated and rare "design earthquake" while the influence of repeated earthquake phenomena is ignored. Despite the fact that the problem has been qualitatively acknowledged, very few studies have been reported in the literature regarding the multiple earthquake phenomena. Hatzigeorgiou and Beskos (2009) and Hatzigeorgiou and Liolios (2010) examined the influence of multiple earthquakes in numerous structural systems and found that seismic sequences lead to increased displacement demands in comparison with the "design earthquake". Similar conclusions are reported in Efraimiadou et al (2011, 2013).

The present study deals with a numerical approach for the seismic analysis of existing adjacent reinforced concrete (RC) building frames under multiple earthquakes. The RC frames can come in unilateral contact during the seismic sequence excitation and their seismic assessment has shown the need to be strengthened. For various reasons (architectural etc.) the strengthening by cable elements was chosen. 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. The investigation purpose is to compare various cable-bracing strengthening versions for existing RC structures, in order the optimum one to be chosen.

METHOD OF ANALYSIS

Problem formulation and numerical treatment

As usually in structural dynamics (Chopra, 2007), a double discretization, in space and time, is used. First, the structural system is discretized in space by using finite elements. As concerns the interfaces of unilateral contact, where pounding is expected to take place, unilateral constraints elements are used (Liolios, 1990). On the other hand, for the cable strengthening system, pin-jointed bar elements are used (Liolios et al, 2012, 2013).

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:

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

Here s_i and d_i are generalized stress and deformation quantities, respectively, $\hat{\partial}$ is the generalized gradient and S_i is the superpotential function, see Panagiotopoulos (1993). 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 Panagiotopoulos (1993)- the relation (1) is equivalent to the following hemivariational inequality, expressing the Virtual Work Principle:

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

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) is expressed by the usual matrix incremental relations:

$$\boldsymbol{M}_A \Delta\ddot{\boldsymbol{u}}_A+\boldsymbol{C}_A \Delta\dot{\boldsymbol{u}}_A+(\boldsymbol{K}_A+\boldsymbol{G}_A)\, \Delta\boldsymbol{u}_A = -\boldsymbol{M}_A \Delta\ddot{\boldsymbol{u}}_g+ \boldsymbol{T}_A \Delta\boldsymbol{s}_A + \boldsymbol{B}\Delta\boldsymbol{p}, \tag{3a}$$

$$\boldsymbol{M}_B \Delta\ddot{\boldsymbol{u}}_B+\boldsymbol{C}_B \Delta\dot{\boldsymbol{u}}_B+(\boldsymbol{K}_B+\boldsymbol{G}_B)\, \Delta\boldsymbol{u}_B = -\boldsymbol{M}_B \Delta\ddot{\boldsymbol{u}}_g+ \boldsymbol{T}_B \Delta\boldsymbol{s}_B - \boldsymbol{B}\Delta\boldsymbol{p}. \tag{3b}$$

$$\boldsymbol{p} = \boldsymbol{p}_N + \boldsymbol{p}_T\,. \tag{3c}$$

Here $\boldsymbol{u}$ and $\boldsymbol{p}$ are the displacement and the contact interface forces time dependent vectors, respectively. $\boldsymbol{M}$, $\boldsymbol{C}$ and $\boldsymbol{K}$ are the mass, damping and stiffness matrices, respectively. $\boldsymbol{G}_A$ and $\boldsymbol{G}_B$ are the geometric stiffness matrices, by which P-Delta effects are taken into account [13]. $\boldsymbol{s}_A$ and $\boldsymbol{s}_B$ are the cable elements stress vectors for frames (A) and (B), respectively. $\boldsymbol{T}_A$, $\boldsymbol{T}_B$, $\boldsymbol{B}$ are transformation matrices. According to (2.3c), the pounding stress vector $\boldsymbol{p}$ is decomposed to the vectors $\boldsymbol{p}_N$, of the normal, and $\boldsymbol{p}_T$ of the tangential interaction forces between frames (A) and (B).

The above relations (2.1)-(2.3), combined with the initial conditions, provide the problem formulation, where, for given seismic ground excitation $\ddot{\boldsymbol{u}}_g$, the vectors $\boldsymbol{u}_A$, $\boldsymbol{u}_B$, $\boldsymbol{p}$ and $\boldsymbol{s}_A$, $\boldsymbol{s}_B$ have to be computed.

From the strict mathematical point of view, using (l) and (2) and following [10], we can formulate the problem as a hemivariational inequality one and investigate it about existence and uniqueness of solution.

As concerns the numerical treatment of the above formulated problem, an approache respecting the strict mathematical formulation has been proposed and applied in (Liolios, 2000). The approach is based on piece-wise linearization as in elastoplasticity (Maier, 1971), on optimization algorithmes and on double discretization, in space by finite elements and in time by a direct integration method. So, in each time-step the solution of a non-convex linear comprlementarity problem of the following form is required (Liolios et al, 2012, 2014):

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

Here $\mathbf{v}$ is the vector of unknown unilateral quantities at the time–moment t, $\mathbf{d}$ is a known vector dependent on excitation and results from previous time moments (t- Δt), and $\mathbf{D}$ is a transformation matrix.

Alternatively, based on the above incremental formulation of the problem, use can be made of the structural analysis software Ruaumoko (Carr, 2008). Ruaumoko software uses the finite element method and permits an extensive parametric study on the inelastic response of structures.

Estimators for the cable-strengthening verions

In order to decide about the necessity of a seismic upgrading, first the seismic assessment of the existing RC structure is realized (Fardis, 2009), (Kappos and Penelis, 2010), (Penelis and Penelis, 2014), (Strauss et al, 2010). In the positive case, to decide further which strengthening scheme is the optimal one, use is made here of the overall structural damage index (OSDI). This is due to the fact, that this parameter summarises all the existing damages on columns and beams of RC frames in a single value, which is useful for comparison reasons (Elenas et al, 2012).

In the OSDI model after Park and Ang (1985) and Ang (1988), 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:

$$DI_L = \frac{\mu_m}{\mu_u} + \frac{\beta}{F_y d_u} E_T \tag{5}$$

where, DI_L is the local damage index, μ_m 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.

The Park/Ang global damage index is a weighted average of the local damage indices and the dissipated energy is chosen as the weighting function. The global damage index is given by the following relation:

$$DI_G = \frac{\sum_{i=1}^{n} DI_L E_i}{\sum_{i=1}^{n} E_i} \tag{6}$$

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

NUMERICAL EXAMPLE

Description of the existing RC structural system

The system of the two reinforced concrete frames (A) and (B) of Fig. 1 is considered. The frames are of concrete class C40/45, and have been designed according to Greek building codes. Seismic assessment is obtained according to current European seismic codes (EC, 2005), (Fardis, 2009). 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, have 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 to be 10% for the columns and 50% for the beams (Paulay and Priestley, 1992).

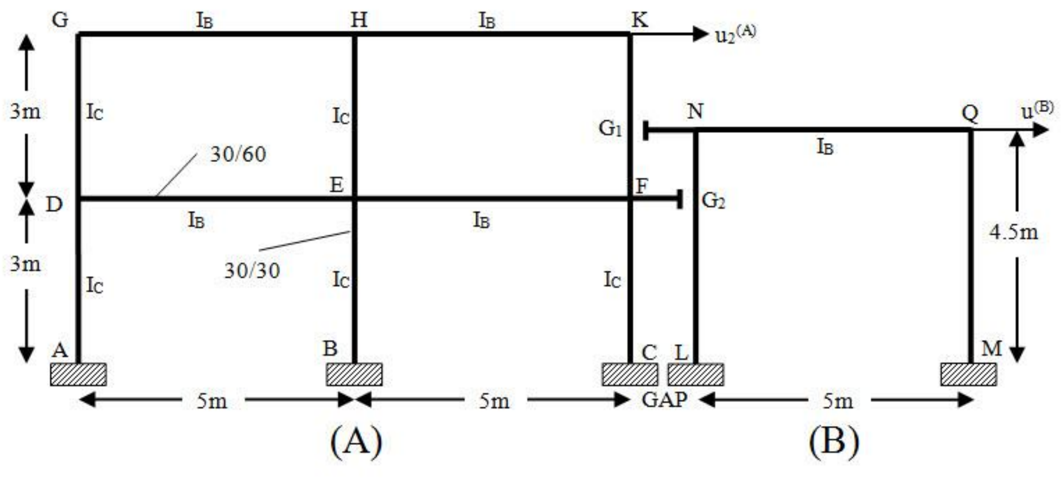

Figure 1. The initial system of the RC frames (A) and (B), without cable-strengthening and with two possible unilateral contacts on G1 and G2. When the contacts are taken into account, the system is denoted as C2S0.

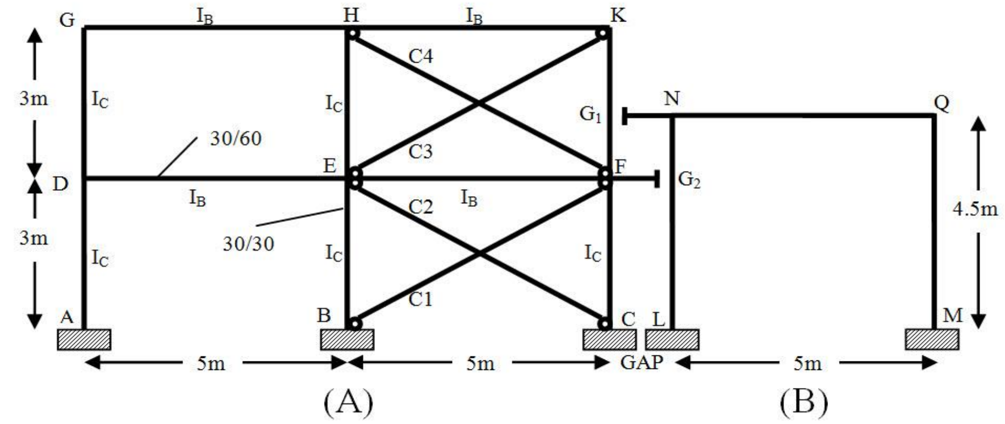

Figure 2. The C2S4 system with diagonal cable-strengthening for frame (A) only.

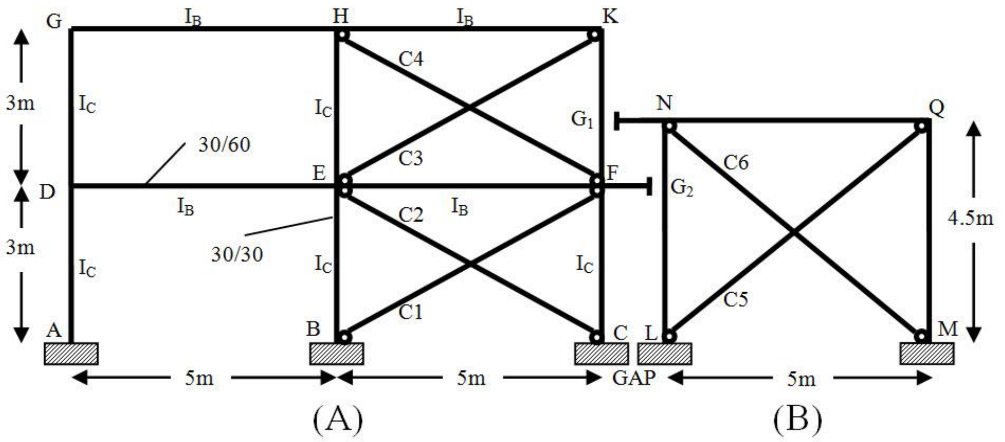

Figure 3. The C2S6 system with diagonal cable-strengthening for both frames (A) and (B).

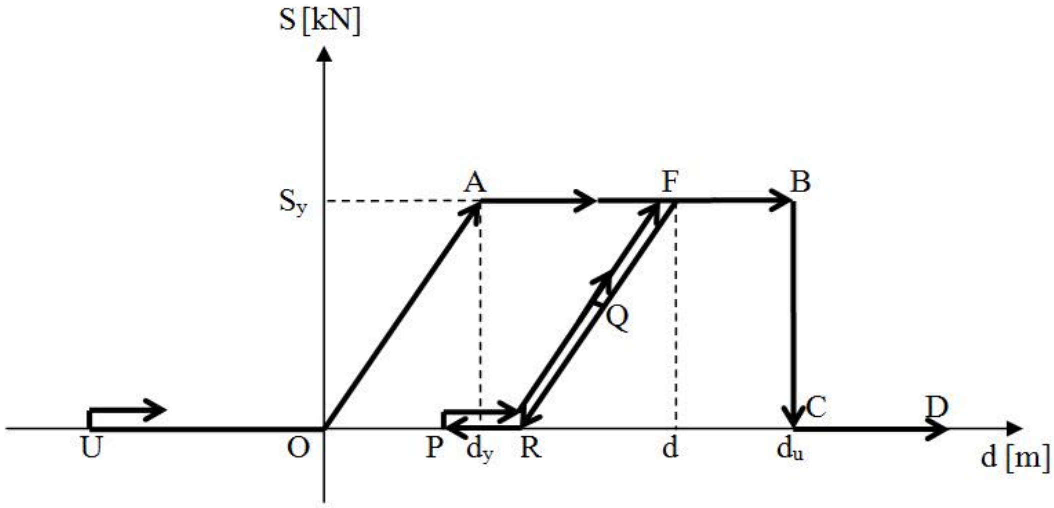

Figure 4. The constitutive law of cable-elements.

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 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.

In order to upgrade seismically the damaged frame system C2S0, two cable-bracing systems, shown in Fig. 2 and Fig. 3, have been investigated. The purpose was to chose the optimal one. The first cable-bracing system of Fig. 2, with 4 cable-elements in frame (A), is denoted as C2S4. The second cable-bracing system of 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 = 18 cm^2 and they are of steel class S220with yield strain ε_y = 0.2 %, fracture strain ε_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.

Multiple Earthquakes Input

The systems C2S0, C2S4 and C2S6 of Figures 1- 3 are considered to be subjected to a multiple ground seismic excitation presented in Hatzigeorgiou and Liolios (2010). The complete list of these earthquakes, which were downloaded from the strong motion database of the Pacific Earthquake Engineering Research Center (PEER), appears in Table 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)	6.1	0.442	0.200
				1980/05/25 (16:49)	6.0	0.178	0.081
				1980/05/25 (19:44)	6.1	0.208	0.094
				1980/05/25 (20:35)	5.7	0.432	0.195
				1980/05/27 (14:51)	6.2	0.316	0.143
2	Chalfant Valley	54428 Zack Brothers Ranch	E-W	1986/07/20 (14:29)	5.9	0.285	0.128
				1986/07/21 (14:42)	6.3	0.447	0.200
3	Coalinga	46T04 CHP	N-S	1983/07/22 (02:39)	6.0	0.605	0.165
				1983/07/25 (22:31)	5.3	0.733	0.200
4	Imperial Valley	5055 Holtville P.O.	HPV	1979/10/15 (23:16)	6.6	0.221	0.200
			315	1979/10/15 (23:19)	5.2	0.211	0.191
5	Whittier Narrows	24401 San Marino	N-S	1987/10/01 (14:42)	5.9	0.204	0.192
				1987/10/04 (10:59)	5.3	0.212	0.200

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.

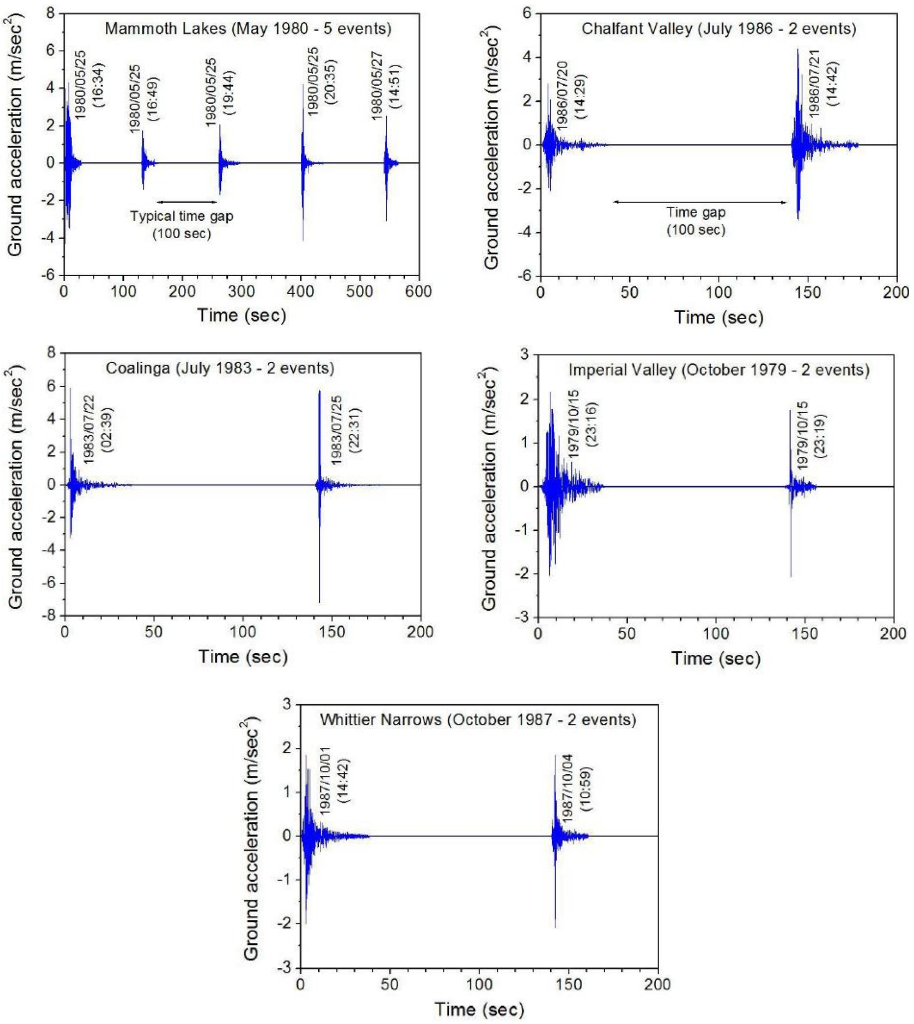

Figure 5. Ground acceleration records of the examined seismic sequences

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 (Hatzigeorgiou and Liolios, 2010). Their simulation is shown in Figure 5.

Representative Results for Multiple Earthquakes

The Coalinga case of the seismic sequence with Recorded PGA equals to 0.605g and 0.733g of Table 1 is investigated and representative results are given in next Table 2.

The representative response quantities are shown in the column (1) of Table 2. For comparison reasons, results concerning the uncoupled case C0S0, when pounding and cable strengthening are not taken into account, are shown on column (2). The results concerning the numerical investigation for the systems C2S0, C2S4 and C2S6 are shown on columns (3), (4) and (5), respectively.

The representative response quantities shown on column (1) of Table 2, are as follows:

1. $u_2^{(A)}$ and $u_1^{(A)}$ are the absolutely maximum horizontal displacements of the second (GHK) and first (DEF) floor, respectively, of frame (A)-see Fig. 1.
2. $u^{(B)}$ is the absolutely maximum horizontal floor (NQ) displacement of frame (B) -see Fig. 1.
3. DI_G is the global damage index.

4. DI_{BE} and DI_{FE} are the local damage indices for bending behavior at the ends B and F of the column BE and of the beam FE, respectively, of frame (A).
5. DI_{G1F} and DI_{G1K} are the local damage indices for bending behavior in the column FK of frame (A).
6. DI_{G2L} and DI_{G2N} are the local damage indices for bending behavior in the column LN of frame (B).
7. FG_2 and NG_1 are the maximum compressive forces (in [kN]) developed on unilateral contacts G_2 and G_1.
8. SC1 to SC6 are the maximum tension forces (in [kN]) developed in the cable elements.

Table 2. Representative results for the Coalinga case multiple earthquakes

Response Quantities	C0S0	C2S0	C2S4	C2S6
(1)	(2)	(3)	(4)	(5)
$u_2^{(A)}$ [mm]	54.82	54.88	32.57	30.84
$u_1^{(A)}$ [mm]	41.17	41.08	21.70	20.68
$u^{(B)}$ [mm]	29.84	30.87	47.68	12.70
DI_G	0.247	0.282	0.242	0.145
DI_{BE}	0.405	0.374	0.342	0.217
DI_{FE}	0.693	0.691	0.427	0.378
DI_{G1F}	0.145	0.194	0.027	0.128
DI_{G1K}	0.008	0.199	0.070	0.197
DI_{G2L}	0.192	0.190	0.192	0.163
DI_{G2N}	0.282	0.268	0.308	0.004
FG_2 [kN]	0	-327.8	-258.5	-277.9
NG_1 [kN]	0	-455.2	-386.1	-465.3
SC1 [kN]	-	-	+438.3	+438.3
SC2 [kN]	-	-	+438.3	+438.3
SC3 [kN]	-	-	+348.4	+324.8
SC4 [kN]	-	-	+295.7	+287.5
SC5 [kN]	-	-	-	+438.3
SC6 [kN]	-	-	-	+391.7

As the values of the above response quantities in Table 2 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 438.3 kN concerning the maximum tension force of the cable-elements C1, C2 and C5 for the cases C2S4 and C2S6 are equal to yield resistance S_y, and thus show that these elements are overstressed beyond the linear elastic limits. The table values prove that the system C2S6 of Fig. 3, i.e. strengthening by X-bracings of both frames (having two pounding contacts), is the optimal one for seismic upgrading the damaged system C2S0.

Finally, the response values of Table 2 are generally higher in comparison to the corresponding ones computed for the single isolated seismic event concerning the Coalinga case of Recorded PGA equal to 0.605g. This case has been investigated in Liolios et al (2014), where details are given. The above comparison proves that multiple earthquakes have accumulative effects on the damage indices of the seismic structural response.

CONCLUSIONS

The herein presented numerical approach can be used for the effective investigation of the inelastic seismic behaviour of adjacent existing RC frames, strengthened by cable elements and subjected to multiple earthquakes. 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 chosen by computing necessary damage indices.

The general conclusion is that pounding has significant effects on the earthquake response of adjacent structure under multiple earthquakes. Cable strengthening can be effectively used as a first solution alternative to traditional ones 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 can indicate the need for strengthening, because increased displacement demands are required in comparison to single seismic events. Furthermore, the seismic damage indices for multiple earthquakes are higher than those 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 structures and the assessment, repairing and strengthening of existing adjacent structures.

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