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SEISMIC STRENGTHENING BY TIES OF HISTORIC REINFORCED CONCRETE BUILDINGS UNDER ENVIRONMENTAL AND SHEAR EFFECTS

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Dedicated to the memory of Yordan MILEV, (15.2.1960 – 08.1.2017), Late Professor at the University of Architecture, Civil Engineering and Geodesy (UACEG), Sofia, Bulgaria

Keywords: *historic reinforced concrete buildings, computational structural mechanics, seismic strengthening by tie-elements, earthquakes sequences, shear effects*

ABSTRACT

The seismic strengthening of existing historic industrial reinforced concrete (RC) structures by using cable elements (tension-ties) is numerically investigated. Special attention is given to environmental and shear-effects, common to old RC structures, which have been designed and constructed before the use of current aseismic codes. The unilateral behaviour of the cable-elements is strictly taken into account and results in inequality constitutive conditions. Finally, using damage indices, the optimal strengthening version of cable-systems is chosen for the case of multi-storey RC frames under seismic sequences of multiple earthquakes.

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1. Introduction

As is well-known, see e.g. [1], the recent built cultural heritage includes, besides the usual historic monumental structures (churches, old masonry buildings, etc.), also existing industrial buildings of reinforced concrete (RC), e.g. old factory premises. These old RC structures have been usually designed and constructed before the use of current aseismic codes, and many of them are vulnerable to shear-effects [2, 25]. Considering their global seismic behaviour, it often arises the need for seismic strengthening, which must be realized by using materials and methods in the context of the sustainable construction [1, 19, 20].

The seismic rehabilitation of such historic RC structures makes use of many well-known repairing and strengthening techniques, see e.g. [1 – 2, 5, 8, 9, 25, 27]. One of the simple, low cost and efficient methods for strengthening existing RC frames against lateral induced earthquake loading is the use of cross X-bracings [5, 8, 12 – 15]. The use of cable-like members (tension-ties) instead of traditional RC mantles can be considered as an alternative strengthening method for inadequate RC frame structures under lateral seismic actions.

Tention-ties have been used effectively in historic buildings, as well as in monastery and churche arches. Cable restrainers are also used for concrete and steel superstructure movement joints in bridges [26]. 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 both an equality and inequality form, and so the problem becomes a highly nonlinear one [17, 21].

In the present study, a numerical approach for the seismic analysis of existing industrial RC frame-structures strengthened by cable elements is presented taking into account shear effects. The approach is based on an incremental formulation and uses the Ruaumoko structural engineering software [3]. Damage indices [18, 22] are computed, for the seismic assessment of historic and industrial structures and in order the optimum cable-bracing strengthening version to be chosen. Finally, an application is presented for a two-bay two-story RC frame strengthened by bracing ties under multiple earthquakes.

2. The computational Approach

2.1. Problem Formulation

For the problem formulation and solution, a double discretization, in space and time, is applied [15]. The structural system is discretized in space by using frame finite elements [4]. Pin-jointed bar elements are used for the cable-elements. The unilateral behaviour of these elements, as well as the behaviour of RC elements, can in general include loosening, elastoplastic or/and elastoplastic-softening-fracturing and unloading – reloading effects, as shown in Fig. 1. 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 [17, 21].

The incremental dynamic equilibrium for the assembled structural system with cables is expressed by the matrix relation:

$$M\Delta\ddot{u} + C\Delta\dot{u} + (K_T + G)\Delta u = f - M\Delta\ddot{u}_g + T\Delta s, \tag{1}$$

where $\boldsymbol{u}(t)$ and $\boldsymbol{p}(t)$ are the displacement and the load time dependent vectors, respectively, and $\boldsymbol{C}$ and $\boldsymbol{K_T}\,\boldsymbol{(u)}$, are the damping and the tangent stiffness matrix, respectively. $\boldsymbol{G}$ is the geometric stiffness matrix, by which P-Delta effects are taken into account. Dots over symbols denote derivatives with respect to time. By $\boldsymbol{f}(t)$ and $\boldsymbol{s}(t)$ the load and cable stress vectors, respectively, are denoted. $\boldsymbol{T}$ is a transformation matrix and $\boldsymbol{u}_g$ the ground seismic excitation.

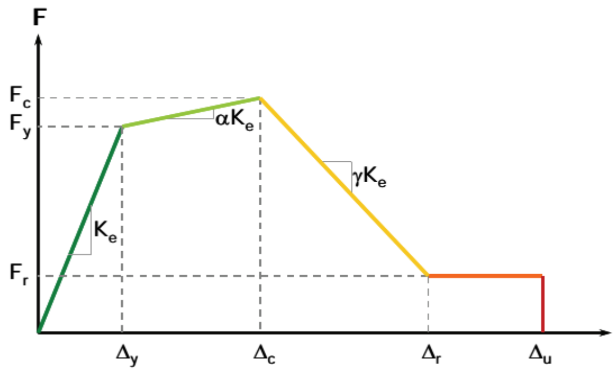

Figure 1. Piecewise linearized constitutive diagram (backbone) for generalized force-generalized displacement of RC elements

The above equation (1) combined with the initial conditions contains the problem formulation, where, for given $\boldsymbol{f}$ and/or $\ddot{u}_g$, the vectors $\boldsymbol{u}$ and $\boldsymbol{s}$ have to be computed. Regarding the strict mathematical point of view, we can formulate the problem as a hemi-variational inequality one by following [17, 21] and investigating it.

For the numerical treatment of the problem, the structural analysis software Ruaumoko [3] is used. Here, for the time-discretization, the Newmark scheme is chosen. Ruaumoko uses the finite element method [5] and provides results which concern, among others, the following critical parameters: local or global structural damage, maximum displacements, inter-storey drift ratios, development of plastic hinges and various response quantities, which allow the using of the incremental dynamic analysis (IDA) method [15].

Ruaumoko has been applied successfully for earthquakes sequences concerning the cases of concrete planar frames [3] and RC frames strengthened by cables [15]. 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 [10, 15].

2.2. Shear Effects

For old RC structures, during a seismic excitation, the RC structural elements can appear flexural, shear or combined flexural-shear failure mode. Considering shear effects, the RC structural elements are classified [25] according to their shear span-to-depth ratio:

$$a_s = \frac{L_s}{d} = \frac{M}{Vd}, \tag{2}$$

where $L_s = M/V$ is the *shear length* corresponding to the considered member end, M the bending moment, V the shear force and d the section depth (height). In the case of RC frame columns or beams, the shear length L_s is equal to one-half of the clear length of the examined structural element according to EC8 [7 – 9].

Due to shear effects, the RC structural elements display one of the three following behaviours [25] according to their shear span-to-depth ratio:

1. For $a_s \geq 7{,}0$, bending prevails, and in this case bending failure occurs before any shear failure, no matter if shear reinforcement exists or not.
2. For $2{,}0 \leq a_s \leq 7{,}0$, the failure mode depends on the shear reinforcement of the web.
3. For $a_s \leq 2{,}0$, the case of short R/C columns is considered, where a special design procedure must be followed so that an explosive cleavage failure of the short column is avoided.

As has been remarked for bridge structural systems [26], in general the shear effects can reduce the available plastic curvature ϕ_p of critical sections. Due to shear effects in plastic hinges, the flexural ductile behaviour can be modified to a flexural-brittle one. Thus a shear failure, and a so-caused reduction of the curvature ductility $\mu_\varphi = \varphi_u / \varphi_y$, can be eventually displayed or not. This must be checked continuously during the seismic excitation, so that the real response of the historic structure can be correctly estimated.

Under a seismic excitation, the RC structural elements have linear-elastic behaviour until one or two of their critical end-regions, after cracking, enter to the yielding state and plastic hinges appear. These effects of cracking on columns and beams are estimated by applying the guidelines of Eurocode 8, part 3 [7, 8] and Greek Retrofitting Code [9]. So, the effective flexural stiffness $E_c I_{eff}$ is given by the EC8-formula [7 – 9, 28]:

$$E_c I_{eff} = \frac{M_y L_s}{3\theta_y} . \tag{3}$$

Here M_y and θ_y are the flexural moment and the chord rotation at yield, respectively, which are calculated by EC8 formulas given in [7 – 9].

On the other hand, the available cyclic shear strength V_R (in MN), corresponding to the considered member end, is decreased with the incremental demand plastic chord rotation $\theta_{p,d}$ according to the following semi-empirical experimental expression given by Eq. (A.12) of Eurocode EC8, see [7 – 9, 16]:

$$V_R = \frac{1}{\gamma_{el}} \left[\frac{(h-x)\lambda_1}{2L_s} + \left(1-0{,}05\lambda_2\right)\left[0{,}16\lambda_3\left(1-0{,}16\lambda_4\right)A_c\sqrt{f_{cm}/CF} + V_w\right]\right] . \tag{4}$$

Here, γ_{el} is a safety factor that is taken equal to 1,15 for primary seismic structural elements (due to scattering of the experimental values) and is taken 1,00 for secondary seismic members. x is the compression zone depth (in meters) that is known by the sectional analysis, *CF* is the Confidence Factor according to Table 3.1 of EC8 [7]; V_W is the contribution of the transverse reinforcement to shear strength, taken as being equal to $V_W = \rho_w b_w z f_{yw,m} / CF$ for cross-section with rectangular web of width b_w; ρ_w is the transverse reinforcement ratio that is given by $\rho_w = \left(A_{sw} l_w\right)/\left(h_c b_c s_h\right)$, where l_w is the total length of the stirrups, A_{sw} is the steel section area of the stirrup, h_c and b_c the dimensions of the confined core of the section and s_h is the centreline spacing of stirrups.

The other parameters in eq. (4) are: $\lambda_1 = \min\left(N,\ 0{,}55A_c\ f_{cm}/CF\right)$; $\lambda_2 = \min\left(5,\ \mu_\Delta^p\right)$; $\lambda_3 = \max\left(0{,}5,\ 100\rho_{tot}\right)$; $\lambda_4 = \min\left(5,\ a_s\right)$, where N is the axial force in [MN], which is positive for compression, while when the axial force is tensional then it is taken zero; $A_c = b_w d$ for rectangular sections with b_w as width of compression zone and d is the depth of the tension reinforcement in meters; f_{cm} is the concrete compressive strength (mean value) in [MPa]; $\mu_\Delta^p = \theta_p/\theta_y$; ρ_{tot} is the total longitudinal reinforcement ratio (tensional, compression and intermediate; and a_s is the contemporary shear ratio.

In order to define the final elastic-plastic diagram of Moment-Chord Rotation (M-θ) of the critical section at the considered member end, it must be checked which type of failure precedes, the flexure-failure or the shear-failure [16]. Thus, using the known value of the shear strength V_R from eq. (6), the moment $M_{u,y}$ at the critical section due to V_R is calculated:

$$M_{u,v} = L_s V_R\,. \tag{5}$$

When $M_{u,y} > M_y$, i.e. when $M_{u,y}$ is greater than the flexural yielding moment M_y, then the flexural failure precedes the shear one. In that case, the final elastic-plastic diagram of Moment-Chord Rotation (M-θ) is given by Figure 2(a).

On the contrary, when $M_{u,y} > M_y$, i.e. when $M_{u,y}$ is smaller than the flexural yielding moment M_y, then the shear failure precedes the flexural one. In this case, the final elastic-plastic diagram of Moment-Chord Rotation (M-θ) diagram of the considered member end is given as the curve OABCD of Figure 2(b) according to sect. 7.2.4.2 of KANEPE [9].

After the above check, the final (corrected) values of M_y and θ_y are used in eq. (3).

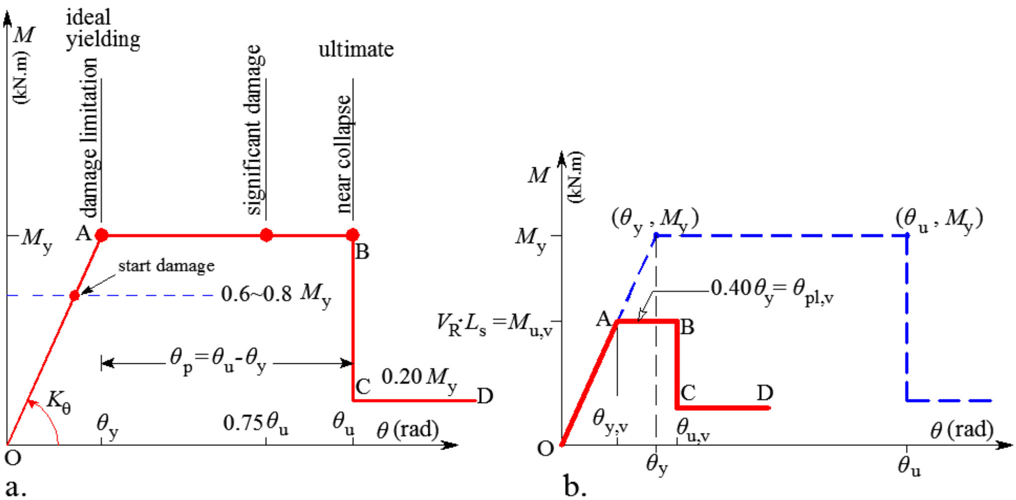

Figure 2. Moment-Chord Rotation Diagram
(a) for ductile failure and (b) for shear failure (from Makarios [16])

2.3. Damage Indices for the Assessment and the Best Ties System

The seismic assessment of the existing RC structure [8], as well as the choice of the best strengthening cable system can be realized by using damage indices [6, 18, 22]. In this study 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.

In the OSDI model after Park/Ang [22] 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:

$$DI_{L,PA} = \frac{\mu_m}{\mu_u} + \frac{\beta}{F_y d_u} E_T \, , \tag{6}$$

where $DI_{L,PA}$ 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 generalized force of the section or element, E_T the dissipated hysteretic energy, d_u the ultimate generalized displacement.

For 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:

$$DI_G = \frac{\sum_{i=1}^{n} DI_{Li} E_i}{\sum_{i=1}^{n} E_i} \, , \tag{7}$$

where DI_G is the global damage index, DI_{Li} 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. Numerical Example

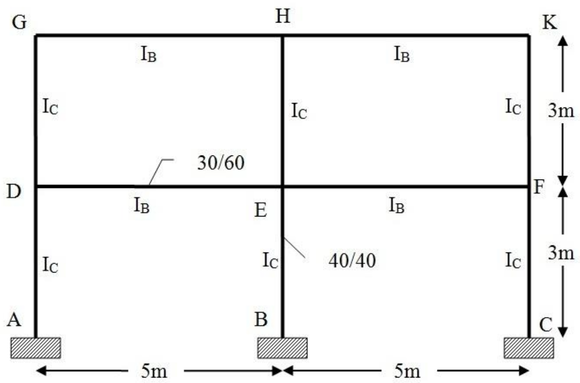

Figure 3. System F0: the industrial RC frame without cable-strengthening

The industrial reinforced concrete frame F0 of Fig. 3 is considered to be rehabilitated by using tention-ties. This frame was designed and constructed according to old Greek antiseismic code of 1959. The concrete quality was C16 (old B160), and for the other various example data (longitudinal reinforcement, stirrups etc.), due to space limitations here, see [15].

After the seismic assessment [8], the X-cable-bracings system, shown in Fig. 4, has been proposed as the optimal one so that the frame F0 can be seismically upgraded. The cable elements have a cross-sectional area F_c = 18 cm^2 and they are of steel class S220. Their constitutive law diagram concerning the unilateral behaviour is shown in Fig. 5.

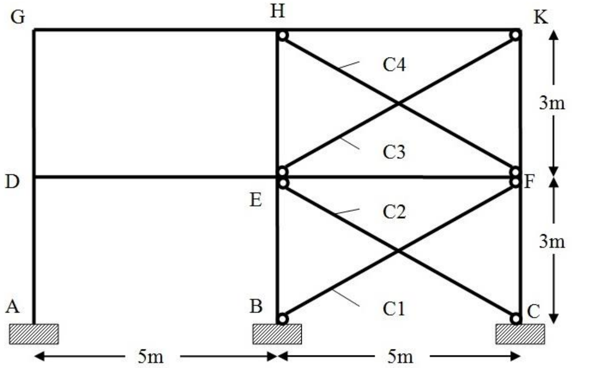

Figure 4. System F2: the industrial RC frame cable-strengthened with x-bracings

The systems of Figures 3 and 4 are subjected to the multiple ground seismic excitation of Coalinga, received from PEER database [10, 24]. This seismic sequence is shown in Table 1.

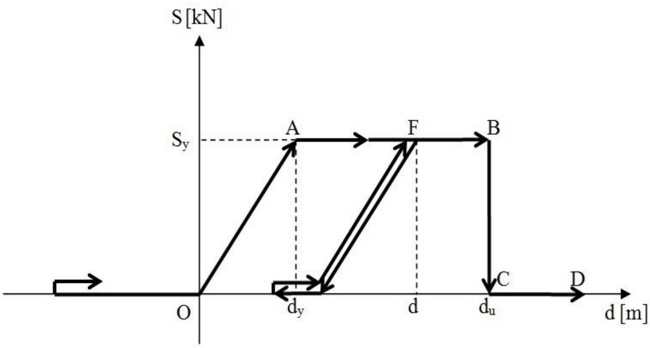

Figure 5. The piecewise linearized constitutive law of the unilateral behaviour of the cable elements

Concerning the Coalinga case of seismic sequence [10, 24], some representative results are shown in Table 2.

In column (1) of the Table 2, the Event E_1 corresponds to Coalinga seismic event of 0,605 g PGA, and Event E_2 to 0,733 g PGA, (g = 9,81 m/sec^2). The sequence of events E_1 and E_2 is denoted as Event (E_1+ E_2). In table column (2) the Global Damage Indices DI_G and in column (3) the Local Damage Index DI_L for the bending moment at the left fixed support A of the frames are given. Finally, in the column (4), the absolutely maximum horizontal top displacement u_{top} is given.

Table 1. Sequential earthquakes data

No	Seismic sequence	Station	Comp	Date (Time)	Magn. M_L	Record. $PGA(g)$	Norm. PGA
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 315	1979/10/15 (23:16)	6,6	0,221	0,200
				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

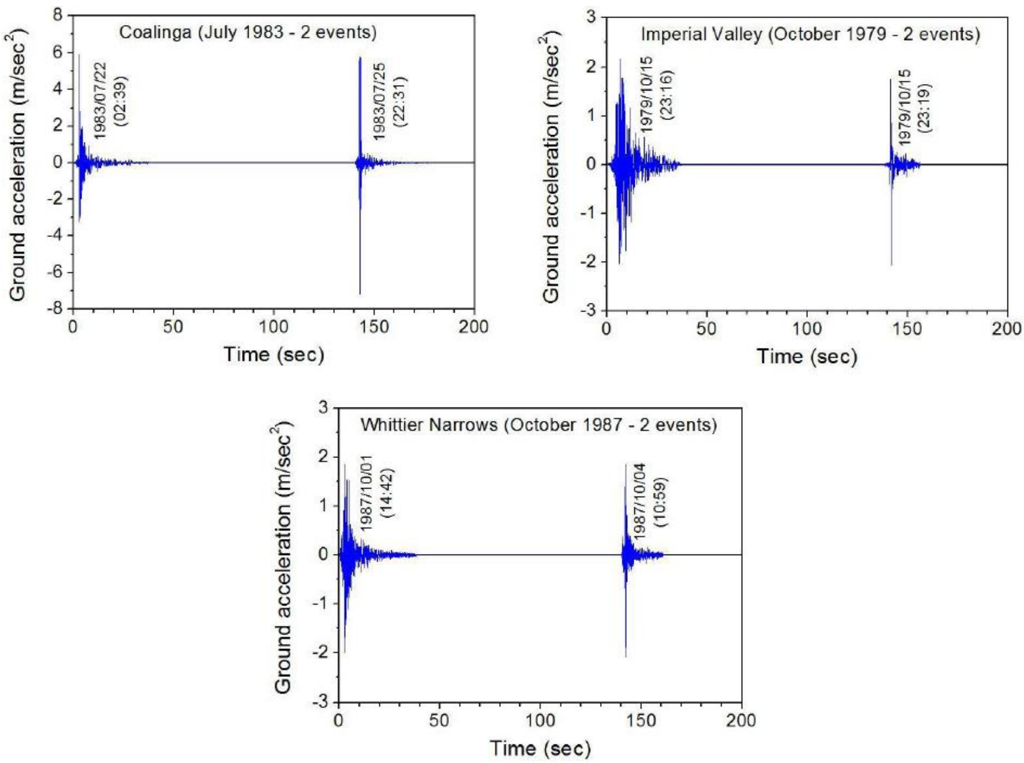

Figure 6. Simulation of the seismic sequences Coalinga, Imperial Valley and Whittier Narrows from PEER database [10, 15]

As the values in Table 2 below show, multiple earthquakes generally increase response quantities, especially the damage indices. Due to shear effects, the values concerning the system F0 are higher than those computed in [15], where such effects were not taken into account. On the other hand, the strengthening of the frame F0 by X-bracings (system Frame F2 of Fig. 8) improves the response behaviour, e.g. u_{top} is reduced from 3,84 cm to 1,48 cm.

Table 2. Representative response quantities for the systems F0 and F2

SYSTEM	EVENTS	DI_G	DI_L	u_{top} [cm]
(0)	(1)	(2)	(3)	(4)
F0	Event E_1	0,147	0,182	2,24
	Event E_2	0,321	0,484	3,48
	Event ($E_1 + E_2$)	0,348	0,497	3,84
F2	Event E_1	0,068	0,007	1,13
	Event E_2	0,097	0,136	1,45
	Event (E_1 + E$_2$)	0,108	0,154	1,48

4. Concluding Remarks

The computational approach presented herein can be effectively used for the numerical investigation of the seismic inelastic behaviour of historic RC frames strengthened by cable elements and subjected to multiple earthquakes. The shear effects, common to such old RC structures, are taken into account, and this has been shown in the presented numerical example. Finally, the computed damage indices can be used effectively so that an optimal cable-bracing scheme can be selected among investigated alternative ones.

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