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SOLID MECHANICS

A NUMERICAL APPROACH FOR ESTIMATING THE EFFECTS OF MULTIPLE EARTHQUAKES TO SEISMIC RESPONSE OF STRUCTURES STRENGTHENED BY CABLE-ELEMENTS*

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Dedicated to Professor Stefan Radev,
Corresponding Member of the Bulgarian Academy of Sciences, Sofia,
on the occasion of his 70-th anniversary.

Abstract. The behaviour of reinforced concrete (RC) frames, which have been strengthened by cable elements and are subjected to multiple earthquakes, is numerically investigated. The purpose is to estimate damage indices in order to compare the seismic response of the structures before and after the retrofit by cable element strengthening and to select the optimum strengthening version.
Key words: Computational solid mechanics, earthquake structural engineering, cable-braced structures, multiple earthquakes, damage indices.

1. Introduction

Civil engineering structures are often exposed to non-usual extremal actions (seismic, environmental etc.), which can cause significant strength degradation and damages. To overcome such defects, sometimes cable-like members

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are used as a first strengthening and repairing procedure against earthquake actions [1–4]. These cable-members can undertake tension, but buckle and become slack and structurally ineffective when subjected to a sufficiently large compressive force. Thus, the problem governing conditions take an equality as well as an inequality form and the problem becomes high nonlinear.

For the strict mathematical treatment of the problem, the concept of variational and/or hemivariational inequalities can be used and has been successfully applied [5, 6]. As concerns the numerical treatment, non-convex optimization algorithms are generally required [5–8].

On the other hand, current seismic codes [9–11] 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 [12–15].

This study presents a numerical approach for the analysis of reinforced concrete (RC) building frames, which have been strengthened by cable elements and are subjected to multiple earthquakes. The finite element method is used for space discretization in combination with a time discretization scheme. The procedure aims mainly to practical applications and uses the Ruaumoko structural engineering software [16]. The purpose is to compare various cable-bracing strengthening versions, in order the optimum one to be chosen.

2. Method of analysis

2.1. Problem formulation

As usually in structural dynamics [17], a double discretization, in space and time, is used. First, the structural system is discretized in space by using finite elements. Pin-jointed bar elements are used for the cables. The behaviour of these elements includes loosening, elastoplastic or/and elastoplastic-softening-fracturing and unloading – reloading effects. All these characteristics concerning the cable constitutive law can be expressed mathematically by the relation:

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

where s_i and d_i are the (tensile) force (in [kN]) and the deformation (elongation) (in [m]), respectively, of the i-th cable element, $\widehat{\partial}$ is the generalized gradient and S_i is the superpotential function, see Panagiotopoulos [5] and Stavroulakis

[6]. By definition, 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 assembled structural system without cables is expressed by the usual matrix relation:

$$\mathbf{M}\,\ddot{\mathbf{u}} + \mathbf{C}(\dot{\mathbf{u}}) + \mathbf{K}(\mathbf{u}) = \mathbf{p}. \tag{3}$$

Here, $\mathbf{u}$ and $\mathbf{p}$ are the displacement and the load 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 form $\mathbf{C}\,\dot{\mathbf{u}}$ and $\mathbf{K}\mathbf{u}$. Dots over symbols denote derivatives with respect to time. When cable-elements are taken into account, equation (3) becomes:

$$\mathbf{M}\,\ddot{\mathbf{u}} + \mathbf{C}(\dot{\mathbf{u}}) + \mathbf{K}(\mathbf{u}) = \mathbf{p} + \mathbf{A}\mathbf{s}. \tag{4}$$

Here, $\mathbf{s}$ is the cable stress vector and $\mathbf{A}$ is a transformation matrix. For the case of ground seismic excitation $\mathbf{x}_g$, the loading history term $\mathbf{p}$ becomes:

$$\mathbf{p} = -\mathbf{M}\mathbf{r}\,\ddot{\mathbf{x}}_g, \tag{5}$$

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

The above relations (1)–(5), combined with the initial conditions, consist the problem formulation, where, for given $\mathbf{p}$ and/or $\ddot{\mathbf{x}}_g$, the vectors $\mathbf{u}$ and $\mathbf{s}$ have to be computed.

From the strict mathematical point of view, using (1) and (2), we can formulate the problem as a hemivariational inequality one by following [5, 6] and investigate it.

2.2. Problem numerical treatment for multiple earthquakes

In civil engineering practical cases, a numerical treatment of the problem is applied. Such an approach, based on a piecewise linearization of the above constitutive relations as in elastoplasticity [18], is described in [19] for cable-braced RC systems. By using the Newmark time-integration scheme, in each time-step a relevant non-convex linear complementarity problem of the following matrix form is solved:

$$\mathbf{v} \geq \mathbf{0}, \quad \mathbf{A}\mathbf{v} + \mathbf{a} \leq \mathbf{0}, \quad \mathbf{v}^{\top}.(\mathbf{A}\mathbf{v} + \mathbf{a}) = \mathbf{0}. \tag{6}$$

So, the nonlinear Response Time-History (RTH) for a given seismic ground excitation can be computed. Similar procedures using optimization methods have been presented in [5, 6, 8].

An alternative approach for treating numerically the problem is the incremental one. On this approach is based the structural analysis software Ruaumoko [16], which uses the finite element method. For the time-discretization, the Newmark scheme is here chosen. Generally, the Ruaumoko software permits an extensive parametric study on the inelastic response of structures.

Especially for the case of multiple earthquakes, Ruaumoko has been applied for reinforced concrete planar frames under real seismic sequences [15, 20–22]. These sequences are recorded by the same station, in the same direction and in a short period of time, up to three days [23]. Comprehensive analysis of the created response databank is employed in order to derive significant conclusions. Ruaumoko provides 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 [24].

2.3. Comparison for the cable-strengthening versions

Among the several response parameters, the focus is on 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 reinforced concrete frames in a single value, which is useful for comparison reasons [25].

In the OSDI model after Park/Ang [26, 27], the global damage is obtained as a weighted average of the local damage at the section ends of each element or at each cable element. The local damage index is given by the following relation:

$$DI_L = \frac{\mu_m}{\mu_u} + \frac{\beta}{F_y d_u} E_T, \tag{7}$$

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:

$$DI_G = \frac{\sum_{i=1}^{n} DI_L E_i}{\sum_{i=1}^{n} E_i}, \tag{8}$$

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.

3. Numerical example

3.1. Description of the considered RC structure.

The reinforced concrete frame F0 of Fig. 1 is of concrete class C40/45, has dimensions L = 7 m and h = 3.5 m and was designed according to Greek building codes and to current European seismic codes [9–11]. The beams are of rectangular section 30/60 (width/height, in cm) and have a total vertical distributed load 30 KN/m (each beam). The columns have section dimensions, in cm: 40/40.

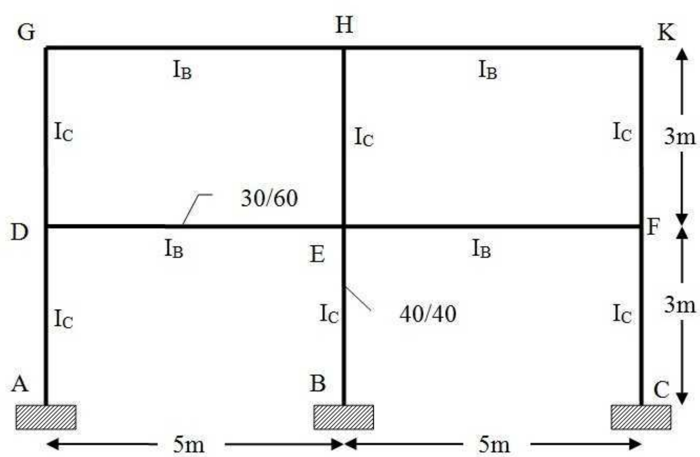

Fig. 1. The initial RC frame F0 without cable-strengthening

The frame was initially constructed without cable-bracings. Due to various extremal actions (environmental etc.), corrosion and cracking has been taken place, which has caused a strength and stiffness degradation. The so

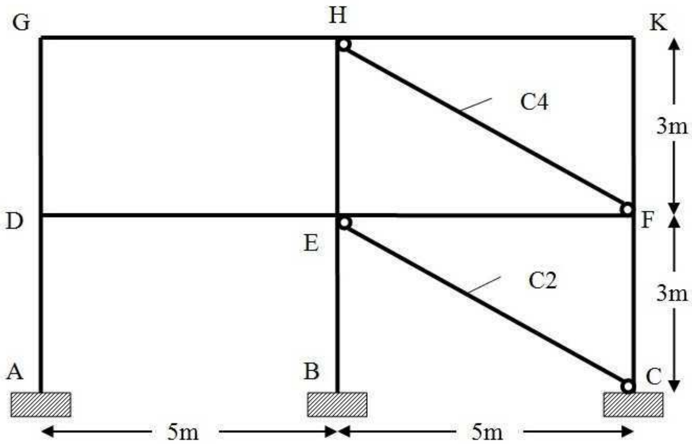

Fig. 2. The RC frame F1 with diagonal cable-strengthening

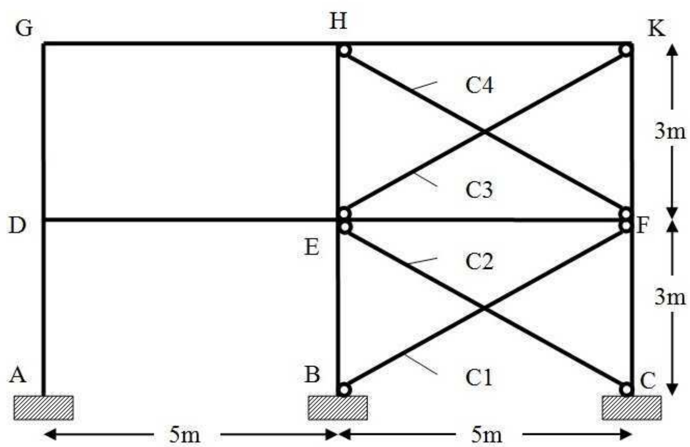

Fig. 3. The RC frame F2 with X-braces cable-strengthening

resulted reduction for the section inertia moments was estimated [28] to be 10% for the columns and 50% for the beams. Two cable-braces systems, shown in Figs 2 and 3, have been investigated in order the optimal one to be chosen. The first, denoted as F1, has descending diagonal cable-elements. The second, denoted as F2, has X-bracing diagonal cable-elements.

The cable elements have a cross-sectional area $F_c = 18$ cm^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, concerning the unilateral (slackness), hysteretic, fracturing, unloading-reloading etc. behaviour, is depicted in Fig. 4.

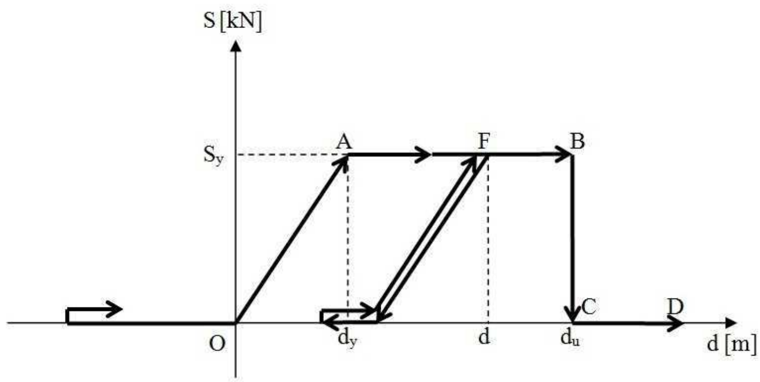

Fig. 4. The diagram for the constitutive law of cable-elements

3.2. Multiple earthquakes input

The system is subjected to a multiple ground seismic excitation, received from PEER database [23] and concerning the Coalinga case of next 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. The complete list of these earthquakes, which were downloaded from the strong motion database of the Pacific Earthquake Engineering Research (PEER) Center [23], appears in Table 1.

Every sequential ground motion records becomes a single ground motion record (serial array), where between two consecutive seismic events a time gap is applied, which is equal to 100 sec. This gap is enough to cease the moving of any structure due to damping. Figure 5 shows the time histories of two such simulated seismic sequences. For compatibility reasons with the design process, the seismic sequences are normalized to have PGA = 0.2 g.

3.3 Representative results

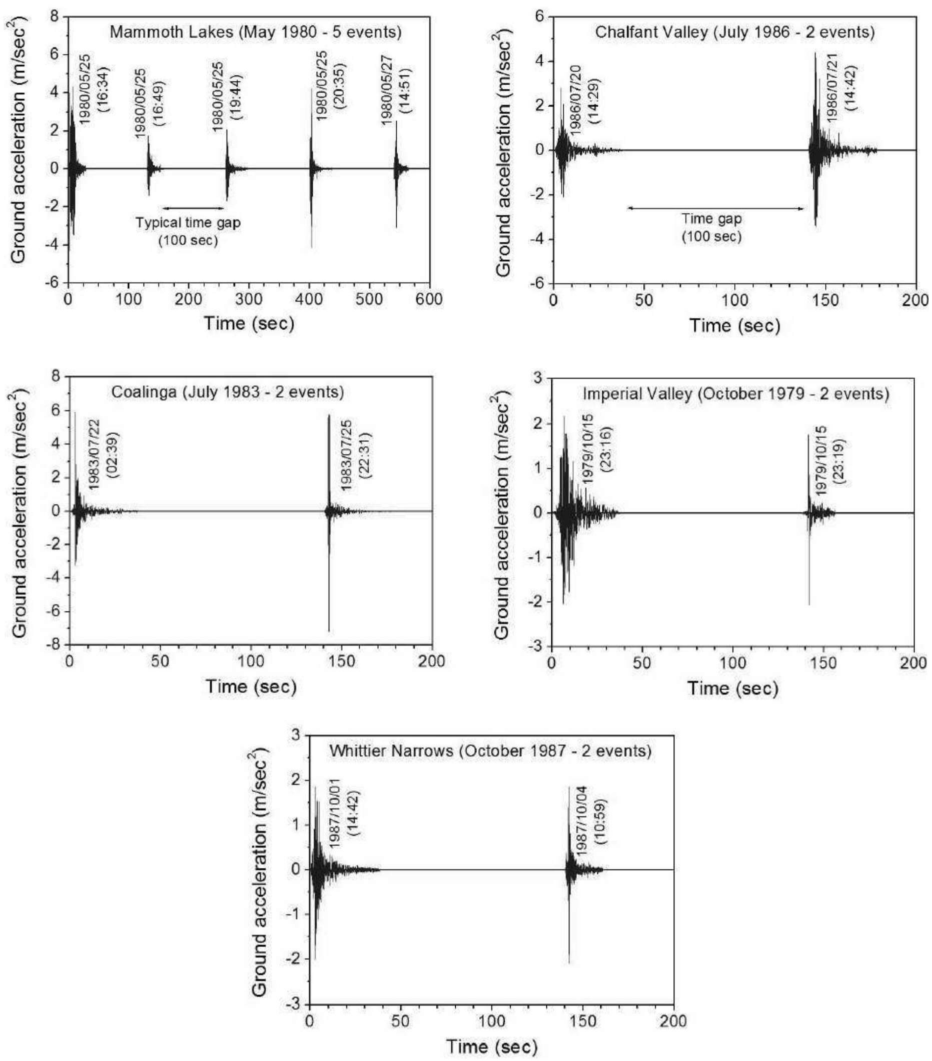

Fig. 5. Ground acceleration records of the simulated seismic sequences

Table 1. Multiple 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 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	NS	1987/10/01 (14:42)	5.9	0.204	0.192
				1987/10/04 (10:59)	5.3	0.212	0.200

Representative results of the numerical investigation are presented in next Table 2. In column (1), Event E_1 corresponds to Coalinga seismic event of 0.165 normalized PGA of Table 1 and Event E_2 to 0.200 normalized PGA. The sequence of events E_1 and E_2 is denoted as Event (E_1+E_2). In 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 maximum horizontal top displacement u_{top} is presented.

As the above table values show, multiple earthquakes generally increase, in an accumulative way, the response quantities. This holds especially for the

Table 2. Representative response quantities for the frames F0, F1 and F2

FRAMES	EVENTS	DI_G	DI_L	u_{top} [cm]
(0)	(1)	(2)	(3)	(4)
F0	Event E_1	0.134	0.179	2.227
	Event E_2	0.301	0.474	3.398
	Event $(E_1 + E_2)$	0.334	0.481	3.410
F1	Event E_1	0.133	0.185	1.715
	Event E_2	0.256	0.354	3.149
	Event $(E_1 + E_2)$	0.317	0.385	3.813
F2	Event E_1	0.068	0.007	1.126
	Event E_2	0.097	0.136	1.447
	Event $(E_1 + E_2)$	0.108	0.154	1.471

damage indices. On the other hand, the strengthening of the frame F0 by X-bracings, i.e. Frame F2, improves the response values and, in comparison to F1, is the optimal one.

3. Conclusions

A numerical approach for the inelastic behaviour of planar RC frames, strengthened by cable elements, under sequential strong ground motions has been presented. As the results of a numerical example have shown, multiple earthquakes generally indicate the need for strengthening. Increased displacement demands are required in comparison with 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 and strengthening of structures.

The presented approach can be also used for a detailed parametric study of the problem in order to obtain the optimum strengthening configuration. So, it seems to be a useful numerical tool for the optimal design of civil engineering structures strengthened by cable-elements against earthquake actions.

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