A computational approach for the seismic sequences induced response of Cultural Heritage structures upgraded by ties

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1 Introduction

In Civil Engineering praxis, recent 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 [1]. As concerns their global seismic behaviour of such RC structures, it often arises the need for seismic upgrading. For the recent Cultural Heritage, this upgrading must be realized by using materials and methods in the context of the Sustainable Construction [1, 2].

The use of cable-like members (tension-ties) can be considered as an alternative strengthening method in comparison to other traditional methods (e.g. RC mantles) [3]. As well-known, ties have been used effectively in monastery buildings and churches arches [1]. 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 equality as well as an inequality form and the problem becomes highly nonlinear [4].

As concerns the seismic upgrading of existing RC structures, modern seismic design codes adopt exclusively the use of the isolated and rare ‘design earthquake’, whereas the influence of repeated earthquake phenomena is ignored. But as the results of recent research have shown [5], multiple earthquakes generally require increased ductility design demands in comparison with single isolated seismic events. Especially for the seismic damage due to multiple earthquakes, this is accumulated and so it is higher than that for single ground motions.

The present research treats with a computational approach for the seismic analysis of Cultural Heritage existing industrial RC frame-buildings that have been strengthened by cable elements and are subjected to seismic sequences. Damage indices 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 it is presented for a simple typical example of a two-bay two-story industrial RC frame strengthened by bracing ties under multiple earthquakes.

2 The computational treatment of the problem

For the system of the partial differential equations (PDE) governing the problem mathematical formulation, a double discretization, in space and time, is used as usually in structural dynamics. Details of the developed numerical approaches are given in [6] and are briefly summarized herein. First, the structural system is discretized in space by using frame finite elements. Pin-jointed bar elements are used for the cable-elements. The behaviour of these elements includes loosening, elastoplastic or/and elastoplastic-softening-fracturing and unloading - reloading effects. Non-linear behaviour is considered as lumped at the two ends of the frame elements, where plastic hinges can be developped. All these non-linear characteristics, concerning the ends of frame elements and the cable constitutive law, can be expressed mathematically by the subdifferential relation [4]

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

Here $\mathrm{s_i}$ and $\mathrm{d_i}$ 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 $\mathrm{S_i}$ is the superpotential function, see Panagiotopoulos [4].

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

$$\mathbf{M\ddot{u}} + \mathbf{C(\dot{u})} + \mathbf{K(u)} = \mathbf{p} + \mathbf{As} \tag{2}$$

Here **u** and **p** are the displacement and the load time dependent vectors, respectively, and **s** is the cable stress vector. **M** is the mass matrix and **A** is a transformation matrix. The damping and stiffness terms, $\mathbf{C(\dot{u})}$ and $\mathbf{K(u)}$, respectively, concern the general non-linear case. Dots over symbols denote derivatives with respect to time. For the case of ground seismic excitation $\mathbf{x_g}$, the loading history term **p** becomes $\mathbf{p} = -\mathbf{M}\ \mathbf{r}\ \mathbf{\ddot{x}_g}$, where **r** is the vector of stereostatic displacements.

The above relations (1)-(5), combined with the initial conditions, consist the problem formulation, where, for given p and/or $\mathbf{\ddot{x}_g}$, the vectors **u** and **s** have to be computed. From the strict mathematical point of view, using (l) and (2), we can formulate the problem as a hemivariational inequality one by following [4] and investigate it.

In Civil Engineering practical cases, a numerical treatment of the problem is applied, based on a piecewise linearization of the above constitutive relations. By using a direct time-integration scheme, in each time-step a relevant non-convex linear complementarity problem [4,6] of the following matrix form is solved :

$$\mathbf{v} \geq \mathbf{0}\,, \qquad \mathbf{D}\,\mathbf{v} + \mathbf{a} \leq \mathbf{0}\,, \qquad \mathbf{v}^{T} . (\mathbf{D}\,\mathbf{v} + \mathbf{a}) = 0\,. \tag{3}$$

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

An alternative approach for treating numerically the problem is the incremental one. So, the matrix incremental dynamic equilibrium is expressed by relation:

$$\boldsymbol{M}\,\Delta\ddot{\boldsymbol{u}} + \boldsymbol{C}\,\Delta\dot{\boldsymbol{u}} + \boldsymbol{K_T}\,\Delta\boldsymbol{u} \;=\; -\boldsymbol{M}\,\Delta\ddot{\boldsymbol{u}}_{\boldsymbol{g}} + \boldsymbol{A}\,\Delta\boldsymbol{s} \tag{4}$$

where $\boldsymbol{K_T}\,(\boldsymbol{u})$, is the damping and the tangent stiffness matrix, respectively. On such incremental approaches is based the structural analysis software Ruaumoko, which uses the finite element method and is applied hereafter. See [5, 6] for details.

After the seismic assessment of the existing RC structure [3], the choice of the best strengthening cable system can be realized by using local and global damage indices [5, 6]. 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 generalized force of the section or element, E_T the dissipated hysteretic energy, d_u the ultimate generalized displacement. For the global damage index DI_G, which is a weighted average of the local damage indices, the dissipated energy is chosen as the weighting function- see [5, 6] for details .

3 A numerical example and concluding remarks

The Cultural Heritage industrial RC frame of Fig. 1a was initially without the cables C1-C6. After seismic assessment, the initial frame, denoted as F_0, was decided to be strengthened by the shown cable-system and is denoted as system F_6. The cable constitutive law, concerning the unilateral (slackness), hysteretic, fracturing, unloading-reloading etc. behavior, is depicted in Fig. 1b. The system F_6 is considered to be subjected to the multiple ground seismic excitation of the Coalinga case [5,6].

Some representative results are shown in Table 1. In column (1) of the Table 1, the Event E_1 corresponds to Coalinga seismic event of 0.605g PGA, and Event E_2 to 0.733g PGA, (g=9.81m/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.

As the values in the Table 2 show, multiple earthquakes generally increase response quantities, especially the damage indices. On the other hand, the strengthening of the frame F0 by X-bracings (system Frame F6 of Fig. 3) improves the response behaviour, e.g. u_{top} is reduced from 3.517 cm to 1.422 cm.

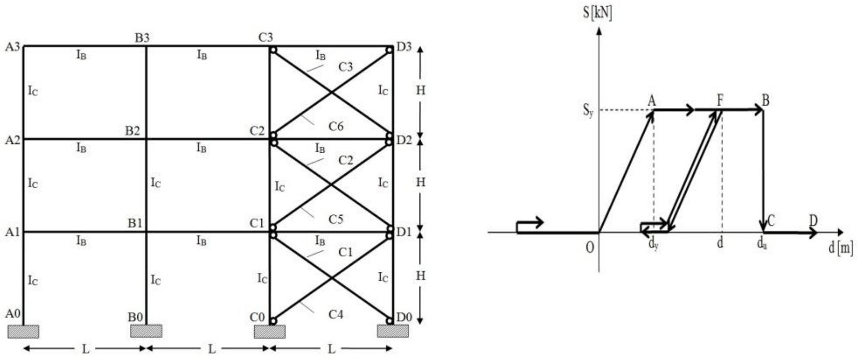

Fig. 1. The frame system and the constitutive law for the cable-elements.

Table 1. Representative response quantities for the systems F0 and F6.

SYSTEM	EVENTS	DI_G	DI_L	u_{top} [cm]
(0)	(1)	(2)	(3)	(4)
F0	Event E_1	0.2534	0.179	2.087
	Event E_2	0.508	0.522	2.984
	Event (E_1+ E_2)	0.5474	0.588	3.517
F6	Event E_1	0.108	0.037	1.126
	Event E_2	0.202	0.114	1.278
	Event (E_1+ E_2)	0.228	0.125	1.422

The results of the numerical example show that the herein presented computational approach can be effectively used for the numerical investigation of the seismic inelastic behaviour of Cultural Heritage industrial RC frames strengthened by cable elements and subjected to multiple earthquakes.

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