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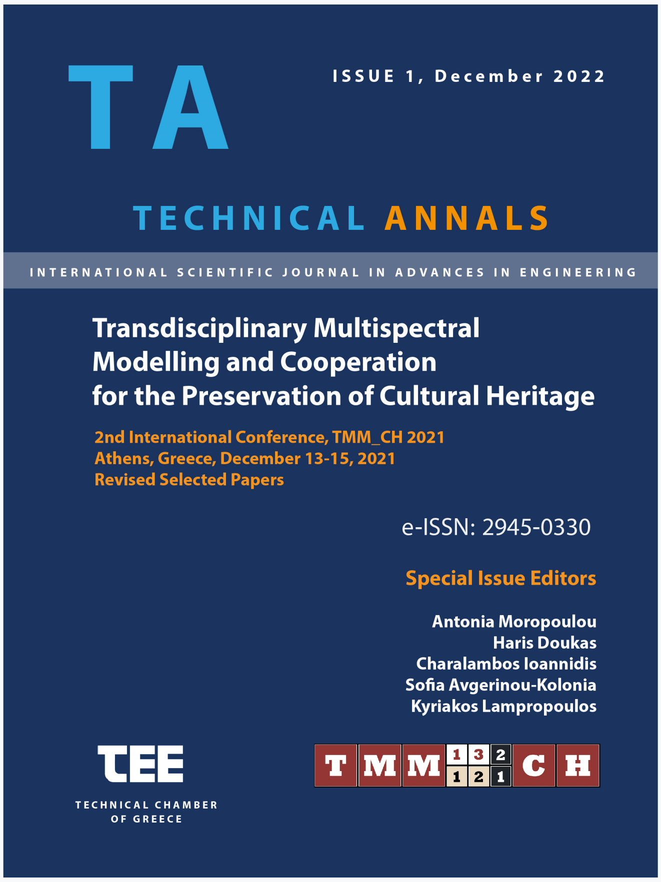


# A stochastic approach for upgrading Cultural Heritage framed concrete buildings by cable-ties to prevent progressive collapse under seismic sequences

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# A stochastic approach for upgrading Cultural Heritage framed concrete buildings by cable-ties to prevent progressive collapse under seismic sequences

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**Abstract.** Old industrial reinforced concrete (RC) structures included to built Cultural Heritage are subjected sometimes to obligatory removal of some structural element-members, e.g. columns. In such cases, a modification of the structural response and a redistribution of internal actions can result to a requirement for upgrading the remaining structure after the removal of the degraded elements in order to avoid a progressive collapse. The present study deals with such a case, which concerns the stochastic computational analysis of historic framed RC structures under the removal of some columns and the so-induced requirement of a strengthening by ties (tension only elements). The unilateral behaviour of these cable-ties, which can undertake only tension, is strictly considered. The response under seismic sequences of the remaining historic RC structure strengthened by ties is computed considering uncertain-but-bounded input parameters.

**Keywords:** Historic industrial RC Structures, Progressive Collapse, Upgrading by Cable-ties, Input Parameters Uncertainty, Seismic Sequences.

## 1 Introduction

The recent built Cultural Heritage (CH) includes, besides the usual historic monumental structures (churches, monasteries, old masonry buildings etc.), also existing old industrial buildings of reinforced concrete (RC), e.g. old factory premises framed structures. Such historic RC structures are subjected to various environmental actions, e.g. corrosion, earthquakes etc., which can often cause significant damages. A main such defect is the strength degradation, resulting into a reduction of the loads bearing capacity of some structural elements. For some of such degraded elements is sometimes obligatory to be removed, and so a further reduction of the whole structure capacity is caused, which can lead to a progressive collapse [1,2].

To avoid such a collapse, a suitable upgrading and strengthening must be performed. Moreover, concerning their global seismic behavior, it often arises the need for their seismic upgrading. Certainly this upgrading must be realized by using materials and methods in the context of the Sustainable Construction and in the frame of the current Civil Engineering praxis [3,4]. Especially for RC structures which belong to recent built Cultural Heritage, some traditional methods for their seismic upgrading (e.g. RC mantles) are available, see [4,5].

Recently, the use of cable-like members (tension-only ties) has been considered as an alternative strengthening method [6,7,16,31-34]. As well-known, ties have been used effectively in monastery buildings and churches arches. The ties-strengthening approach has the advantages of "cleaner" and "more lenient" operation, avoiding as much as possible the unmaking, the digging, the extensive concreting and "nuisance" functionality of the existing building. These benefits hold also for Cultural Heritage RC structures. It is emphasized that the 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 a highly nonlinear one [6-10].

For the numerical analysis of such old RC structures, many uncertainties for input parameters must be taking into account. These mainly concern the holding properties of the old materials that had been used for the building of such structures, e.g. the remaining strength of the concrete and steel, as well as the cracking effects etc. Therefore, an appropriate estimation of the input parameters and use of probabilistic methods must be performed. For the quantification of such uncertainties, probabilistic methods have been proposed [11-16].

Moreover, 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 [17, 18], seismic sequences generally require increased ductility design demands in comparison with single isolated seismic events. Especially for the seismic damage due to multiple earthquakes and to pounding [17-19], this is accumulated and so it is higher than that for single seismic events.

The present research treats with a computational probabilistic approach for the seismic analysis of Cultural Heritage existing industrial RC framed-buildings, which are subjected to removal of some structural elements and are under seismic sequences. These structures are to be strengthened by cable-ties elements in order a progressive collapse to be prevented. Special attention is given for the estimation of the uncertainties concerning structural input parameters. So uncertain-but-bounded input parameters [20] are considered and treated by using Monte Carlo techniques [12-15,21- 23,35-37]. Damage indices are computed for the seismic assessment of such historic and industrial RC structures [24,25]. Finally, an application is presented for a simple typical example of an industrial RC frame strengthened by bracing ties in order to prevent progressive collapse under seismic sequences.

## 2 The Stochastic Computational Approach

The stochastic seismic analysis of Cultural Heritage existing RC framed-buildings is obtained herein through Monte Carlo simulations. As well-known, see e.g. [21-23], Monte Carlo simulation is simply a repeated process of generating deterministic solutions to a given problem. Each solution corresponds to a set of deterministic input values of the underlying random variables. A statistical analysis of the so obtained simulated solutions is then performed. Thus the computational methodology consists of solving first the deterministic problem any times for each set of the random input variables and finally realizing a statistical analysis. Details of the methodology are described in [16] and are given briefly in the next sections.

### 2.1 Numerical Treatment of the Deterministic Problem

The mathematical formulation and solution of the deterministic problem concerning the seismic analysis of Cultural Heritage existing RC frame-buildings strengthened by ties has been recently developed in [6, 7, 16]. Briefly, a double discretization, in space and time, is used. So, first, the structural system is discretized in space by using frame finite elements. Non-linear behavior is considered as lumped at the two ends of the RC frame elements, where plastic hinges can be developed. Pin-jointed bar elements are used for the cable-elements. The unilateral behavior of these tie-elements and the non-linear behavior of the RC structural elements can include loosening, elastoplastic or/and elastoplastic-softening-fracturing and unloading - reloading effects. All these non- linear characteristics, concerning the ends of frame elements and the cable constitutive law, can be expressed mathematically by the subdifferential relation [8,9]:

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

Here $s_i$ and $d_i$ are generalized stress and deformation quantities. For the case of tie-elements, these quantities are the tensile force (in [kN]) and the elongation (in [m]), respectively, of the i-th cable element. $\partial$ is the generalized gradient and $S_i$ is the superpotential function, see Panagiotopoulos [8] and [9,10].

For the numerical treatment of the problem, the cable-elements are taken into account and the dynamic equilibrium for the structural system is written in incremental matrix notation::

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

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, $\mathbf{C}(\dot{\mathbf{u}})$ and $\mathbf{K(u)}$, respectively, concern the general non-linear case. Dots over symbols denote derivatives with respect to time.

The above relations (1)-(2), combined with the initial conditions, consist the problem formulation, where, for given p, 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 dynamic hemivariational inequality one by following [8,9] and investigate it.

For the computational treatment of the problem, the structural analysis software Ruaumoko [26] is applied hereafter as described in [16]. The decision about a possible strengthening for an existing RC structure, damaged by a seismic event, can be taken after a relevant assessment. This can be obtained by using in situ structural identifications [13] and evaluating suitable damage indices. The focus herein is on the overall structural damage index $DI_G$ after Park/Ang, as in details is described in [16,24,25].

The global damage assessment index is obtained as a weighted average of the local damage index 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:

$$DI_L = \frac{\mu_m}{\mu_u} + \frac{\beta}{F_y d_u} E_T \tag{3}$$

where: $\mu_m$ is the maximum ductility attained during the load history, $\mu_u$ 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:

$$DI_G = \frac{\sum_{i=1}^{n} DI_{Li} E_i}{\sum_{i=1}^{n} E_i} \tag{4}$$

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.

## 2.2 Numerical Treatment of the Probabilistic Problem

As mentioned, the Monte Carlo simulation is used [21-23] to calculate the random characteristics of the response of the considered cultural Heritage RC buildings. The·main element of a Monte Carlo simulation procedure is the generation of random numbers from a specified distribution. Systematic and efficient methods for generating such random numbers from several common probability distributions are available. The random variable simulation is implemented herein by using the technique of Latin Hypercube Sampling (LHS) [12-15,35-37]. The generated basic design variables are treated as a sample of experimental observations and used for the system deterministic analysis to obtain a simulated solution as in subsection 2.1. is described. As the generation of the basic design variables is repeated, more simulated solutions can be determined. Finally, a statistical analysis of the obtained simulated solutions is performed.

In more details, a set of values of the basic design input variables can be generated according to their corresponding probability distributions by using statistical sampling techniques. As concerns the uncertain-but-bounded input parameters [20] for the stochastic analysis, these are estimated here by using available upper and lower bounds, denoted as $U_B$ and $L_B$ respectively. So, the mean values are estimated as $(U_B + L_B)/2$.

Such design variables for the herein considered RC buildings are the uncertain quantities describing the backbone diagrams of non-linear constitutive laws, e.g. plastic-hinges behavior, and the spatial variation of input parameters for old building materials. Concerning the plastic hinges in the end sections of the frame structural elements, a typical normalized moment- normalized rotation backbone is shown in Figure 1, see [14]. This backbone hardens after a yield moment $M_y$, having a non- negative slope of $a_h$ up to a corner normalized rotation (or rotational ductility) $\mu_c$ where the negative stiffness segment starts. The drop, at a slope of $a_c$, is arrested by the residual plateau appearing at normalized height $r$ that abruptly ends at the ultimate rotational ductility $\mu_u$. The normalized rotation is the rotational ductility $\mu=\theta/\theta^{yield}$.

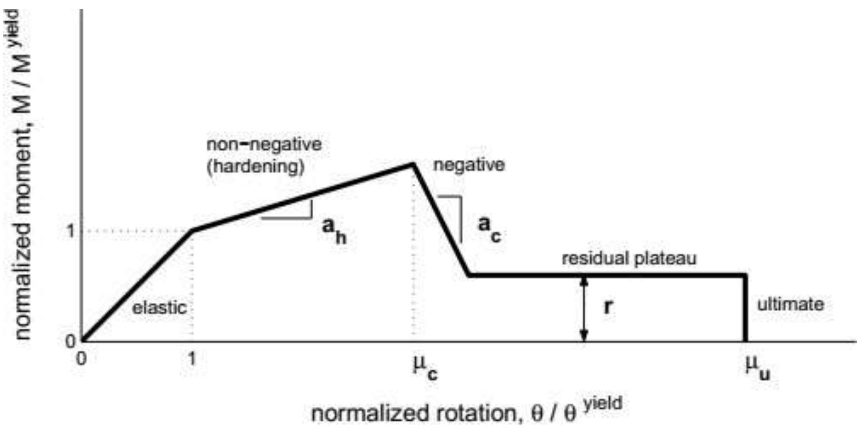


**Fig. 1**. Representative moment-rotation backbone diagramme for plastic hinges [14].

**Table 1.** Uncertain-but-bounded parameters for a typical plastic hinge

| | Mean | COV | $L_B$ (min) | $U_B$ (max) | Distr. type |
|---|---|---|---|---|---|
| $a_{My}$ | 1.0 | 20% | 0.70 | 1.30 | Normal-tr. |
| $a_h$ | 0.1 | 40% | 0.04 | 0.16 | Normal-tr. |
| $\mu_c$ | 3.0 | 40% | 1.20 | 4.80 | Normal-tr. |
| $a_c$ | -0.5 | 40% | -0.80 | -0.20 | Normal-tr. |
| $r$ | 0.5 | 40% | 0.20 | 0.80 | Normal-tr. |
| $\mu_u$ | 6.0 | 40% | 2.40 | 9.60 | Normal-tr. |

The above six backbone parameters in Fig. 1, namely $a_h$, $a_c$, $\mu_c$, $r$, $\mu_u$ and $a_{My},= M/M_y$ are assumed to vary independently from each other according to a truncated Normal distribution. Typical distribution properties for these uncertain-but-bounded parameters concerning plastic hinges according to [14] are given in Table 1. The table values concern the mean value, the coefficient of variation (COV) and the upper and lower bounds of the truncated Normal distribution.

As regards the random variation of input parameters for the old materials, which had been used for the building of old RC structures, their input estimations concern mainly the remaining strength of the concrete and the steel and the elasticity modulus. According to JCSS (Joint Committee Structural Safety), see [11], concrete strength and elasticity modulus follow the Normal distribution, whereas the steel strength follows the Lognormal distribution.

# 3 Numerical Example

## 3.1 Description of the considered Cultural Heritage RC Structural System

The Cultural Heritage old industrial reinforced concrete frame F0 of Fig. 2 is considered to be upgraded by ties in order to avoid progressive collapse and will be subjected to a multiple ground seismic excitation. This system F0 had been designed and constructed according to old Greek building codes, having initially two more internal columns in the ground floor. These columns are shown as dashed lines and have been removed due to degradation caused by environmental actions. Following [1, 27], the axial loads, which were initially undertaken by these two columns, are now shown as the two applied vertical concentrated loads of 180 kN and 220 kN. The loading system shown in Fig.2 is the critical one taken into account the “equivalent static” loading according to Greek codes, see [28].

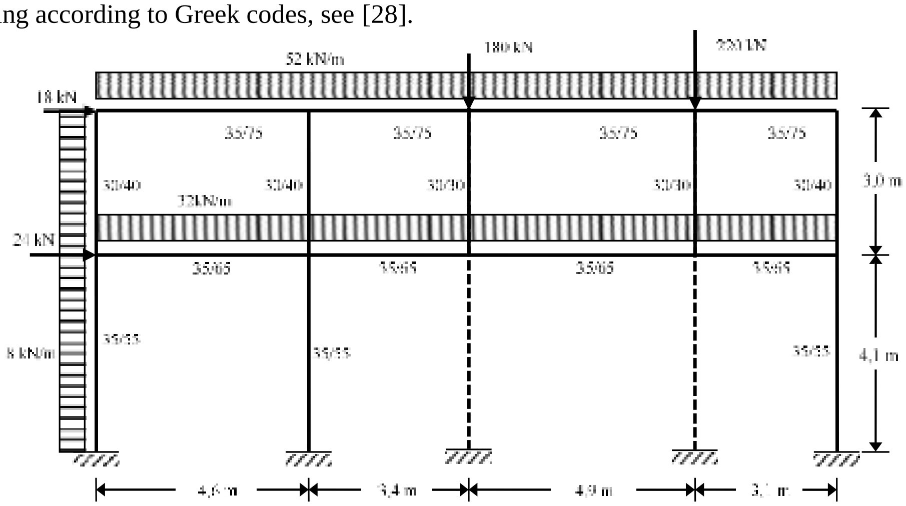


**Fig. 2.** System F0: The initial RC frame (without cables-strengthening).

Due to removal of the above two columns, the behavior of the horizontal beams connected with them changes drastically: These beams are not working further as “continuous beams”, although they had been designed and constructed as such ones. So, after a structural assessment by a “push-over” methodology [4,5,13,15,26] and incremental dynamic analysis (IDA) procedures [14,33-37] of the system F0 under uncertainty and the shown critical loading system, it is concluded that the initial RC frame F0 of Fig. 2 is under a significant risk for a progressive collapse. Indeed, as concerns the *global* damage index $DI_G$, a value greater than one is computed. This holds even more when seismic events and/or seismic sequences are activated.

In order to prevent such a progressive collapse, the initial RC frame F0 of Fig. 2 is strengthened by ten (10) steel cables (tension-only bracing elements) as shown in Fig.3. These strengthening cable members have a cross-sectional area $F_r = 20\ cm^2$ and are of steel class S1400/1600 with elasticity modulus $E_s = 210$ GPa. The cable constitutive law concerning the unilateral (slackness), hysteretic, fracturing, unloading-reloading etc. behavior, has the diagram depicted in Fig. 4.

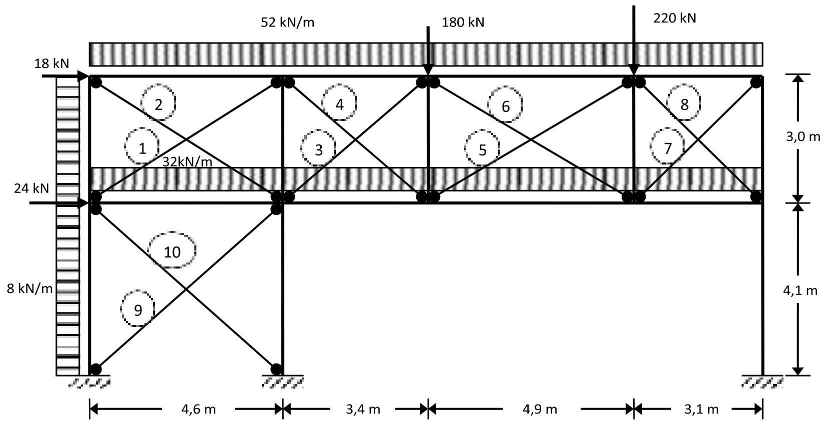


**Fig. 3.** System F10: The RC frame strengthened by 10 cables--strengthening.

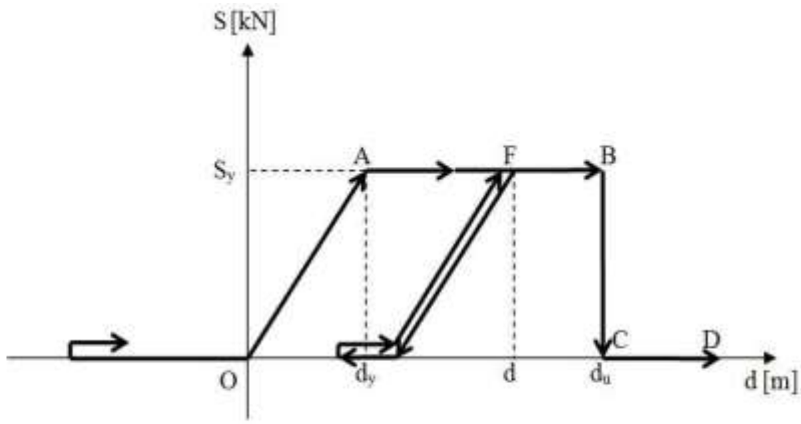


**Fig. 4.** The diagramme for the constitutive law of cable-elements.

Due to various extreme actions (seismic, environmental etc.), corrosion and cracking has been taken place, which has caused a strength and stiffness degradation estimated by insitu investigations. The effects of cracking on columns and beams are simulated by applying the guidelines of [28,29]. So, the stiffness reduction due to cracking results to effective stiffness with mean values of 0.60 $I_g$ for the external columns, 0.80 $I_g$ for the internal columns and 0.40 $I_g$ for the beams, where $I_g$ is the gross inertia moment of their cross-section.

Using Ruaumoko software [26], the columns and the beams of the frame are modeled by prismatic frame RC elements. Nonlinearity at the two ends of the RC frame structural elements is idealized by using one-component plastic hinge models, following the Takeda hysteresis rule [26]. Interaction curves (M-N) for the critical cross-sections of the examined RC frame have been computed. The Ruaumoko Bi- linear with slackness hysteresis element is used for the simulation of the cable (tension- only) elements.

The concrete class of the initial old frame is estimated to be C12/15. According to JCSS (Joint Committee Structural Safety), see [11,35-37], concrete strength and elasticity modulus follow a Normal probability density distribution (pdf) and the steel strength follows the Lognormal distribution. So the statistical characteristics of the input random variables concerning the old building materials are estimated to be as shown in Table 2. By COV is denoted the coefficient of variation. The mean/median values of the random variables correspond to the best estimates employed in the deterministic model according to Greek codes, see KANEPE [28]. On the contrary, the input variables concerning the steel of the bracing ties (new material) are considered as deterministic ones.

**Table 2.** Statistical data for the old building materials treated as random variables

| | Distribution | mean | COV |
|---|---|---|---|
| Compressive strength of concrete | Normal | 12.0 MPa | 15% |
| Yield strength of steel | Lognormal | 191.3 MPa | 10% |
| Initial elasticity modulus of concrete | Normal | 26.0 GPA | 8% |
| Initial elasticity modulus of steel | Normal | 200 GPA | 4% |

### 3.2 Seismic Sequences Input and some Representative Probabilistic Results

In Table 3 three typical real seismic sequence are reported, which have been downloaded from the strong motion database of the Pacific Earthquake Engineering Research (PEER) Center [30], see also [17,18].

The system F10 with cable elements of Fig. 3 is considered to be subjected to the Coalinga seismic sequence of the Table 3. The application of the proposed numerical procedure by using 250 Monte Carlo samples gives the following representative results concerning some dynamic response characteristics:

In column (2) of the Table 4, 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 the table column (3) the mean value and in column (4) the coefficient of variation COV of the Global Damage Indices $DI_G$. are given. Similarly, in the columns (5) and (6) the mean value and the coefficient of variation COV of the absolutely maximum vertical displacement $U_{y220}$, (under the concentrated load of 220 kN, see Fig. 2), respectively, are given.

**Table 3.** Multiple earthquakes data

| No | Seismic sequence | Date (Time) | Magnitude (ML) | Recorded PGA(g) | Normalized PGA(g) |
|---|---|---|---|---|---|
| 1 | Coalinga | 1983/07/22 (02:39) | 6.0 | 0.605 | 0.165 |
| | | 1983/07/25 (22:31) | 5.3 | 0.733 | 0.200 |
| 2 | Imperial Valley | 1979/10/15 (23:16) | 6.6 | 0.221 | 0.200 |
| | | 1979/10/15 (23:19) | 5.2 | 0.211 | 0.191 |
| 3 | Whittier Narrows | 1987/10/01 (14:42) | 5.9 | 0.204 | 0.192 |
| | | 1987/10/04 (10:59) | 5.3 | 0.212 | 0.200 |

**Table 4.** Representative probabilistic dynamic response quantities for the system F10.

| SYSTEM | EVENTS | $DI_G$ | | $U_{y220}$ [cm] | |
|---|---|---|---|---|---|
| | | Mean value | COV | Mean value | COV |
| (1) | (2) | (3) | (4) | (5) | (6) |
| F10 | Event $E_1$ | 0.128 | 16.8% | -1.32 | 14.2% |
| | Event $E_2$ | 0.187 | 17.2% | -1.48 | 15.1% |
| | Event ($E_1$+ $E_2$) | 0.248 | 18.4% | -1.73 | 16.8% |

As the table values show, multiple earthquakes generally increase, in an accumulative way, the response quantities, e.g. critical vertical displacements and damage indices. On the other hand, the strengthening of the frame F0 by X-bracings (system Frame F10 of Fig. 3) improves the response behaviour against seismic sequences. So, the values of the Global Damage Indices $DI_G$ show that the progressive collapse has been avoided.

Especially for the sequence of events $E_1$ and $E_2$, i.e. Event ($E_1$+ $E_2$), the following mean-value results for the maximum response tension are computed concerning the critically active cable-elements of the stress vector $\underline{s}$, where: $\underline{s} = [S_1, S_2, \ldots, S_{10}]^T$:

$S_1 = 13.53$ kN, $S_4 = 698.24$ kN, $S_5 = 10.72$ kN,
$S_7 = 607.84$ kN, $S_9 = 71.25$ kN.

The relevant mean coefficient of variation is COV=21.84%.

### 3.3 Some Comments concerning the Representative Results

Obviously, by a suitable parametric investigation concerning the increase of Monte Carlo samples number, a further improved study of the predicted behavior for the ties-strengthened system F10 can be obtained, because the values of the coefficients of variations are reduced.

Similarly, by a suitable parametric investigation concerning the characteristics of the cable-elements, e.g. increase of sectional area $F_r$, etc., an improved upgrading of the initial structure F0 can be obtained and a further risk reduction of progressive collapse can be achieved for the system F10.

As reported in section 3.1, only the parameters concerning the old building materials, concrete and steel, in the existing RC frame F0 are considered as input random variables, which have a probability density function (pdf) with a symmetric statistic distribution. On the contrary, the input variables concerning the steel of the added bracing ties (new material) are considered as deterministic ones. So, the unilateral behavior of these tie-elements has no influence on the treatment of the probabilistic problem.

Moreover, as the above reported results for the maximum response tension of the activated cable-ties in the numerical example show, the numerical methodology presented herein in section 2.1 for the deterministic problem takes strictly into account the cable unilateral behavior. Thus, this methodology is an effective and reliable one. In the relevant earlier research studies [31,32] concerning strengthening by ties, the cable unilateral behavior is taken strictly into account only when the cable-ties can be placed in symmetric geometrical arrangements.

## 4 Concluding Remarks

The herein presented computational approach can be effectively used for the probabilistic numerical investigation of the seismic inelastic behaviour of Cultural Heritage old RC framed structures strengthened by cable elements in order to prevent progressive collapse. This is proven by the results of a typical numerical example concerning the seismic response of a system subjected to multiple earthquakes. The probabilistic treatment of the uncertain-but-bounded input parameters is effectively realized by using Monte Carlo simulation. Finally, the optimal cable-bracing scheme to avoid progressive collapse can be selected in a parametric way among investigated alternative ones by using computed damage indices.

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