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Historic adjacent concrete buildings strengthened by cable-ties under seismic pounding effects: A stochastic approach considering uncertain input parameters

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Abstract. Cultural Heritage structures include existing old industrial framed reinforced concrete (RC) buildings. The present study deals with a stochastic numerical treatment for the pounding problem concerning the seismic interaction between historic adjacent framed structures strengthened by cable-ties (tension-only bracings) when the input parameters are uncertain. This problem concerns here the unilateral contact between neighbouring structures during earthquakes and is considered as an inequality problem of dynamic structural contact mechanics. The Monte Carlo method is used for treating the uncertainty concerning input parameters. The purpose here is to estimate numerically and to control actively the influence of the cable-ties on the seismic response of the adjacent structures. Finally, in a practical case of two seismically interacting historic framed reinforced concrete (RC) structures, the effectiveness of the proposed methodology is shown.

Keywords: Historic RC Structures, Seismic pounding effects, Upgrading by Cable-ties, Seismic Sequences, Input Parameters Uncertainty, Monte Carlo method.

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, see e.g. [1]. In systems of such historic structures, the case of the seismic interaction (pounding) between adjacent structures or structural parts can become a crucial problem [2-9]. It is reminded that pounding concerns the seismic interaction between adjacent structures, e.g. neighboring buildings in city centers constructed in contact when the so-called "continuous" building system is allowed to be applied. On the common contact interface, during an earthquake excitation, appear at each time-moment either compressive stresses or relative removal displacements (separating gaps) only. These requirements result to inequality conditions in the mathematical problem formulation [10]. Moreover, pounding can cause significant strength degradation and damages on adjacent structures.

In order to overcome the above strength degradation effects, various repairing and strengthening procedures can be used for the seismic upgrading of existing RC buildings [8, 11-12]. Certainly this upgrading of Cultural Heritage structures must be realized by using materials and methods in the context of the sustainable structures [13]. Among the rehabilitation procedures, cable-like members (tension-only bracings) can be used as a first strengthening and repairing procedure [14-17].

Tention-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 (tension-only) ties can undertake tension but buckle and become slack and structurally ineffective when subjected to a sufficiently large compressive force. Thus, the governing conditions in the mathematical problem formulation take equality as well as an inequality form and the problem becomes a highly nonlinear one. As concerns the numerical treatment, nonconvex optimization algorithms are generally required, see details in [10, 18-21].

Concerning the numerical analysis of such existing old Cultural Heritage RC structural systems, many uncertainties for input parameters must be taken 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 [22-26].

As concerns the current 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 [27], 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 this is accumulated and so it is higher than that for single seismic events, see [7, 27-28].

In the present research study, a computational probabilistic approach is developed for the seismic analysis of Cultural Heritage adjacent existing industrial RC framed-buildings. These structures are subjected to seismic sequences and are to be strengthened by cable-ties elements in order to reduce the pounding effects. Special attention is given for the estimation of the uncertainties concerning structural input parameters. Souncertain-but-bounded input parameters [29] are considered and treated by using Monte Carlo techniques [30-32]. Damage indices are computed for the seismic assessment of such historic and industrial RC structures [33-34]. Finally, an application is presented for a simple typical example of an industrial RC system strengthened by bracing ties in order to reduce pounding effects under seismic sequences.

2 The Stochastic Method of Analysis

A stochastic seismic analysis of Cultural Heritage existing RC framed-buildings has been recently presented [26]. This methodology proposed in [26] is followed herein. As well-known, see e.g. [30-32], 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.

2.1 Numerical Treatment of the Deterministic Problem

The mathematical formulation and solution of the deterministic problem concerning the seismic analysis of existing RC adjacent frame-buildings strengthened by ties has been recently developed in [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 (tension-only). 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, the cable constitutive law and the unilateral contact, can be expressed mathematically by the subdifferential relation [18-19]:

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

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. $\hat{\partial}$ is the generalized gradient and S_i is the super-potential function, see Panagiotopoulos [18] and [19].

For the numerical treatment of the problem, the cable-elements and the unilateral-contact are taken into account. Thus, the dynamic equilibrium for the structural system of two adjacent structures (A) and (B) is written in matrix notation:

$$\mathbf{M_A}\ddot{\mathbf{u}}_\mathbf{A} + \mathbf{C_A}(\dot{\mathbf{u}}_\mathbf{A}) + \mathbf{K_A}(\mathbf{u_A}) = \mathbf{f_A} + \mathbf{T_A}\mathbf{s_A} + \mathbf{Bp}\,. \qquad (2A)$$

$$\mathbf{M_B}\ddot{\mathbf{u}}_\mathbf{B} + \mathbf{C_B}(\dot{\mathbf{u}}_\mathbf{B}) + \mathbf{K_B}(\mathbf{u_B}) = \mathbf{f_B} + \mathbf{T_B}\mathbf{s_B} - \mathbf{Bp}\,. \qquad (2B)$$

$$\mathbf{p} = \mathbf{p}_N + \mathbf{p}_T. \qquad (3)$$

Here $\mathbf{s}_A$ and $\mathbf{s}_B$ are the cable elements stress vectors for the two adjacent structures (A) and (B), respectively; $\mathbf{p}$ is the contact elements stress vector and $\mathbf{T_A}$, $\mathbf{T_B}$ and $\mathbf{B}$are transformation matrices. The pounding stress vector $\mathbf{p}$ is decomposed to the vectors $\mathbf{p}_N$, of the normal, and $\mathbf{p}_T$ of the tangential interaction forces between structures (A) and

(B). By $\mathbf{u}_L$ and $\mathbf{f}_L$ are denoted the displacement vector for the structure $L=A$, B and the load time dependent vector, respectively. 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. For the case of ground seismic excitation $\mathbf{x}_g$, the loading history terms $\mathbf{f}_L$ become

$$\mathbf{f}_L = -\mathbf{M}_L \; \mathbf{r}_L \; \ddot{\mathbf{x}}_g . \tag{4}$$

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

The above relations (1)-(4), combined with the initial conditions, provide the problem formulation, where, for given $\mathbf{f}_L$, the vectors $\mathbf{u}_A$, $\mathbf{u}_B$, $\mathbf{p}$ and $\mathbf{s}_A$, $\mathbf{s}_B$ have to be computed.

For the computational treatment of the problem, the structural analysis software Ruaumoko [35]is applied hereafter as in details 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 evaluating suitable damage indices. The focus herein is on the overall structural damage index $\mathrm{DI_G}$ after Park/Ang, as in details is described in [33, 34].

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 modified [34] *local* damage index $\mathrm{DI_L}$ is computed by the following relation:

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

where: μ_m is the maximum ductility attained during the load history, μ_u the ultimate ductility capacity of the section or element, μ_y the yield ductility, 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 generalized deformation.

Next, the dissipated energy E_T is chosen as the weighting function and the *global* damage index $\mathrm{DI_G}$ is computed by using the following relation:

$$\mathrm{DI_G} = \frac{\sum_{i=1}^{n} \mathrm{DI_{Li}} E_i}{\sum_{i=1}^{n} E_i} \tag{6}$$

where: $\mathrm{DI_{Li}}$ 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

In order to calculate the random characteristics of the considered cultural Heritage RC system, the Monte Carlo simulation is used following [26]. As well-known, see e.g. [30-32], 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) [23-25]. 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 [29] for the stochastic analysis, these are estimated here by using available upper and lower bounds, denoted as U_B and L_B respectively. So, a mean value (average) is estimated as $(U_B + L_B)/2$ and a deviation amplitude 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 [24]. 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) μ_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 μ_u. The normalized rotation is the rotational ductility $\mu=\theta/\theta^{yield}$.

The above six backbone parameters in Fig. 1, namely a_h, a_c, μ_c, *r*, μ_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 [24] 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 [22], concrete strength and elasticity modulus follow the Normal distribution, whereas the steel strength follows the Lognormal distribution.

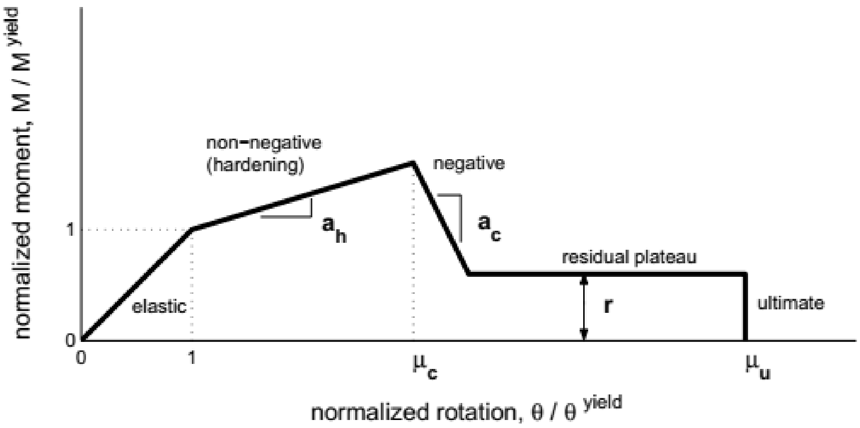

Fig. 1. Representative moment-rotation backbone diagramme for plastic hinges [24].

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%	0.80	1.20	Normal-tr.
a_h	0.1	40%	0.06	0.14	Normal-tr.
μ_c	3.0	40%	1.80	4.20	Normal-tr.
a_c	-0.5	40%	-0.70	-0.30	Normal-tr.
r	0.5	40%	0.30	0.70	Normal-tr.
μ_u	6.0	40%	3.60	8.40	Normal-tr.

3 Numerical Example

3.1 Description and modelling of the considered Cultural Heritage RC structural system

The investigated Cultural Heritage old industrial reinforced concrete systems shown in Fig. 2. This system is a 2-D "mixed" system of two adjacent reinforced concrete (RC) structures, the frame (A) and the shear wall (B). The frame (A) is to be upgraded by ties. The system will be subjected to a multiple ground seismic excitation.

The shear wall (B) has an orthogonal opening of 2mx3m. The frame 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 frame columns, with section inertia moment I_C, have section dimensions, in cm: 30/30. The thickness of the shear wall (B) is 20cm. The structures are parts of two adjacent buildings, which initially were designed and constructed independently in different time periods.

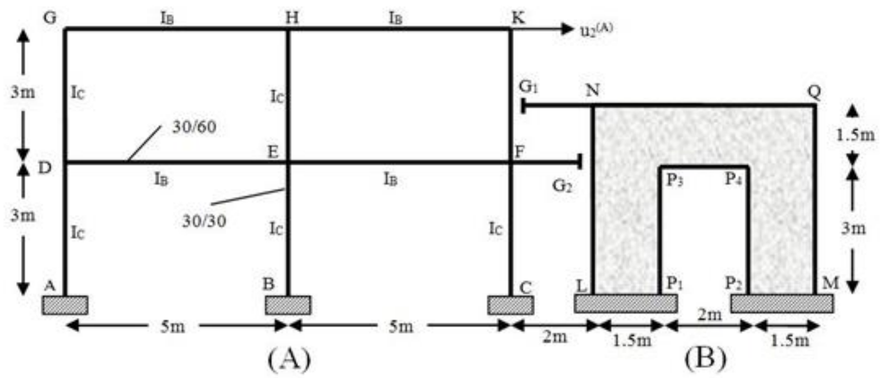

Fig. 2. The initial system of the RC structures (A) and (B), without cable-strengthening and with two possible unilateral contacts on G_1 and G_2**.**

Due to connections shown in Fig. 1, pounding is expected to take place on frame column FK (point G_1) and on shear wall part LN (point G_2) of structures (A) and (B), respectively. The gaps on G_1 and G_2 are taken initially as zero. The system of the seismically interacting RC structures (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 effective stiffness of the concrete members are estimated according to [36-37]. The so resulted reduction for the section inertia moments I_C and I_B was estimated to be 20% for the internal column BH and the shear wall (B), 40% for the external columns AG and CK, and 60% for the frame beams.

As concerns the discretization in space by using finite elements, for the RC frame (A) the usual 2-D frame elements are used (see the Manual of Ruaumoko code, [35]). For the shear RC wall (B), use is made of the displacement-compatible plane stress model proposed and applied in [38]. This model is a quadrilateral plane stress one with 8 nodes totally. Of them, the 4 nodes are the corner ones and the 4 others on the side middles. Each node has three degrees of freedom. So, the displacement vector of each node i has two translational components, u_{ix} and u_{iy} , and one rotational component θ_{iz}. This formulation allows the connection of the plane stress elements with the frame elements. Concerning the shear wall (B), 6 square elements with dimensions 1.5mx1.5m and one orthogonal element with dimensions 2.0mx1.5m are used.

In order to rehabilitate seismically the system, the initial RC frame (A) of Fig. 2 is strengthened by four (4) steel cables (tension-only bracing elements) as shown in Fig. 3. The cable-bracing scheme of Fig. 3, with 4 cable-elements in frame (A), is denoted as S4. The 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 been developed in [16].

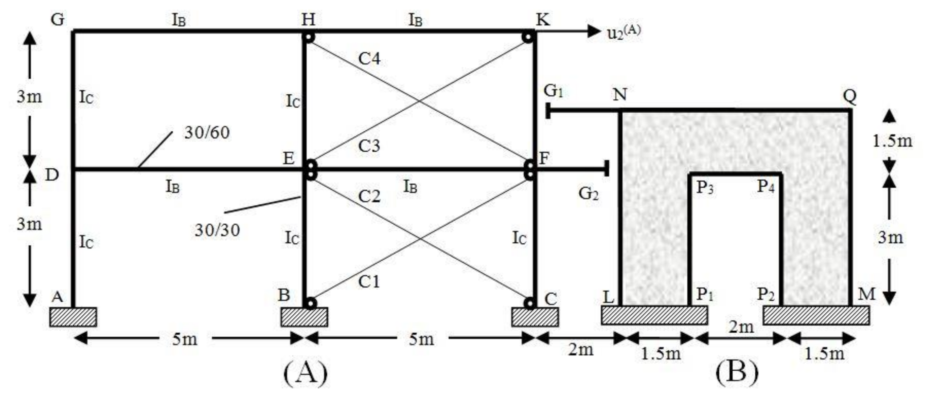

Fig.3. The S4 system with 4 diagonal strengthening cables in frame (A).

Using Ruaumoko software [35], the columns and the beams of the frame are modelled 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. Interaction curves (M-N) for the critical cross-sections of the examined RC frame have been computed.

For the modelling of the cable (tension-only bracing) elements, the Ruaumoko“Bi-linear with Slackness Hysteresis Rule”IHYST = 5 shown in Fig. 4 is considered (see Fig. 33 in the Manual of Ruaumoko code, [35]), taking into account also the Ruaumoko“Degrading Strength Rule” shown in Fig. 5 (see Fig. 48 in the Manual of Ruaumoko code, [35]).

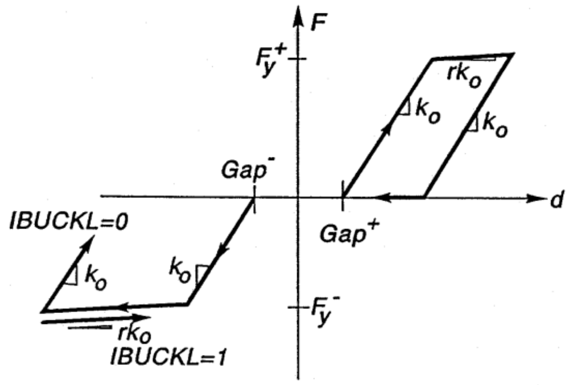

Fig.4. The Bilinear with Slackness Hysteresis RuleIHYST = 5
|in Ruaumoko code, see Carr [35].

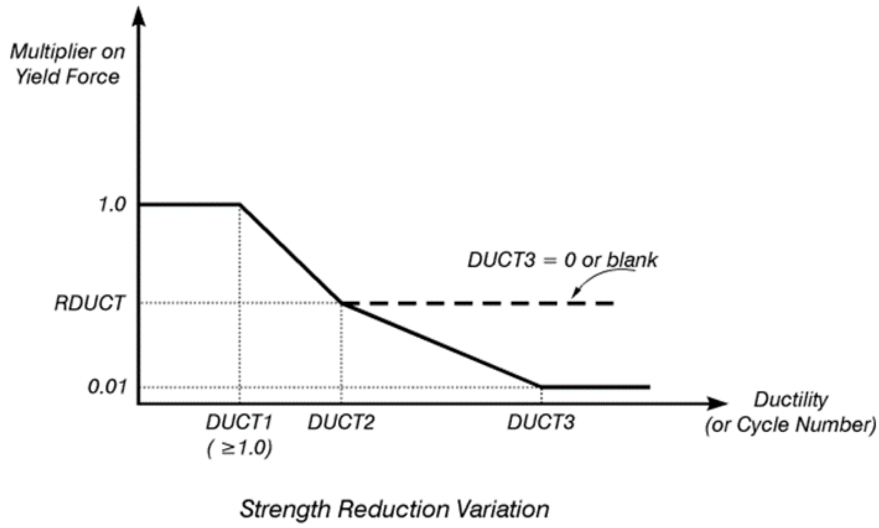

Fig.5. The RuaumokoDegrading Strength Rule, see Carr [35].

Further, investigations presented in [39, 40] and shown in Figures 6, 7 and 8 are taken into account. In more details, in the paper [39], concerning the seismic behaviour of cross-braced frames, the diagonal tension-only bracings were taken as being effective only when in tension and were modelled as ‘bilinear with slackness’ with a large value of slackness being given for the compressive direction so that a compressive stiffness would never occur, see Figure 6. In the paper [40], representative shake table test results for the El Centro 1940 N-Searthquake excitation include the Fig. 7 concerning “Stress-strain hysteresis loops for tension-only braces” and the Fig. 8 concerning a typical force time-history for tension-only braces.

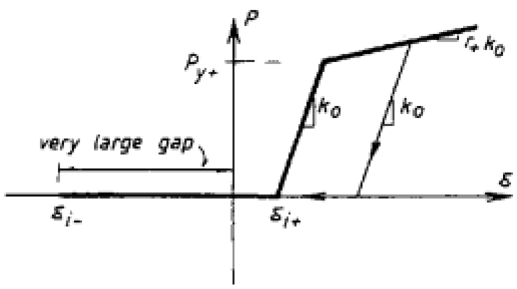

Fig.6. Bilinear hysteresis model for tention-ties with a large value of initial slackness in compression, see [39].

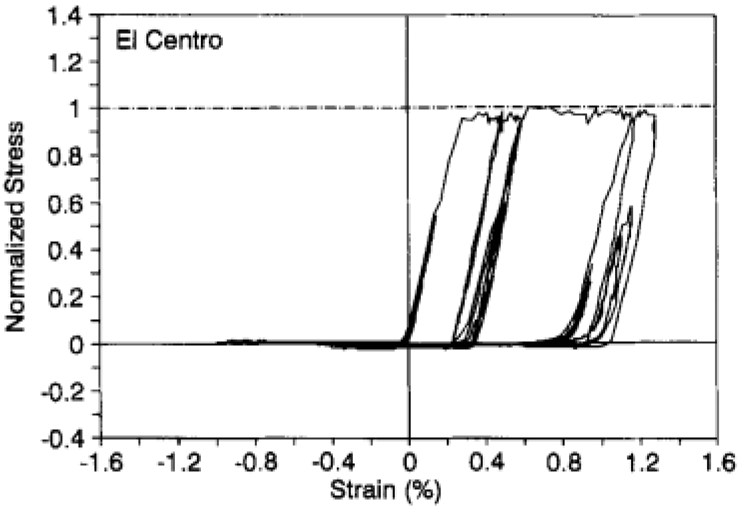

Fig. 7. Stress-strain hysteresis loops for tension-only braces, see [40].

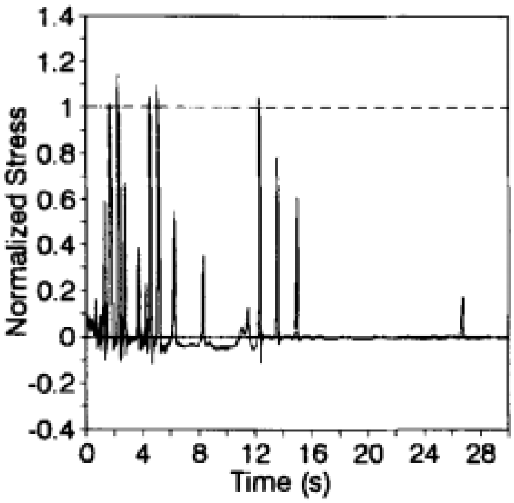

Fig. 8. Typical force time-history fortension-only braces, see [40].

Taking into account all the above considerations and [41], the constitutive law of cable-elements presented in [16] and shown in Fig. 9 is finally used herein.

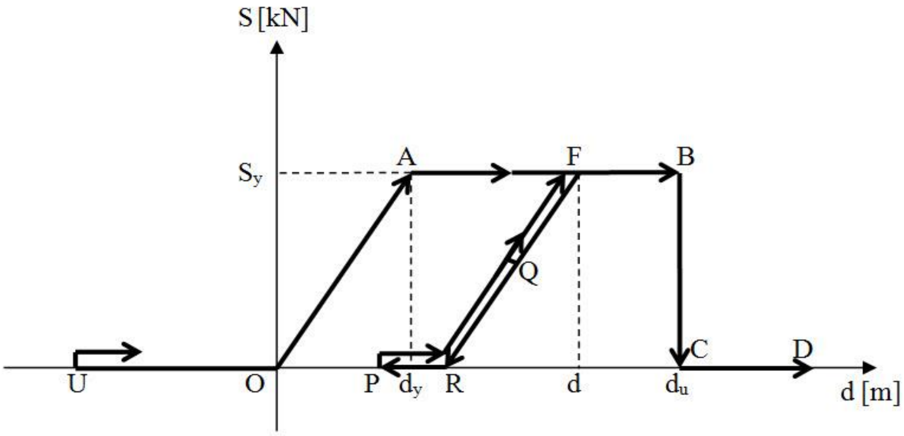

Fig.9. The diagramme for the constitutive law of cable-elements, see [16].

The unilateral contacts in G1 and G2 are modelled by using the Contact-Element of the Ruaumoko library [35], and by applying the procedure presented in [7], see Fig. 10.

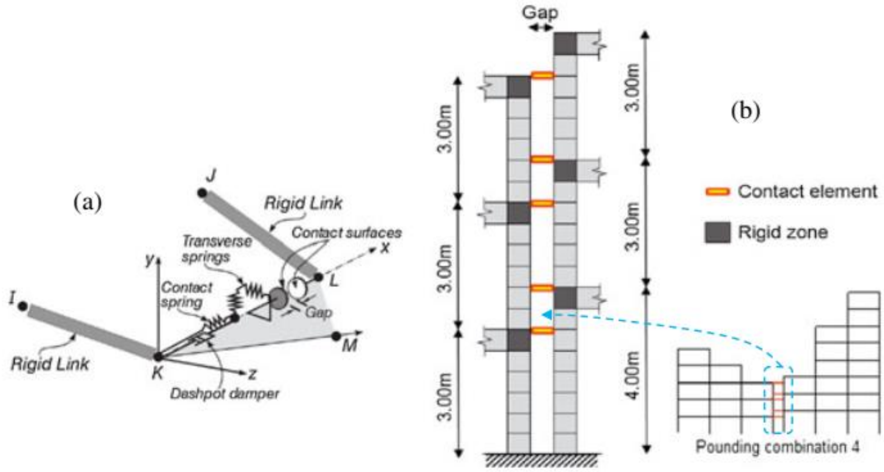

Fig. 10.(a) RuaumokoContact Element (from [35]),
(b) Modelling of structures interface , see [7].

The concrete class of the initial old system is estimated to be C12/15. According to JCSS (Joint Committee Structural Safety), see [22], 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 [36]. 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, see [27, 28]. The systemsS0 and S4 are considered to be subjected to the Coalinga seismic sequence of the Table 3.

Table 3. Multiple earthquakes data

No	Seismic sequence	Date (Time)	Magnitude (M_L)	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

The proposed numerical procedure is applied by using 250 Monte Carlo samples. Some representative results of the numerical investigation concerning the systems S0 and S4, for the sequence of Coalinga seismic events only, are presented in Table 4.

Table 4. Representative response quantities for the systems S0 and S4

SYSTEM	EVENTS	DI_G	DI_L	IMPACT-G_1 [kN]	IMPACT-G_2 [kN]	u_{top} [mm]
(1)	(2)	(3)	(4)	(5)	(6)	(7)
S0	E_1	0.204	0.238	-116.7	-42.8	-36.8
	E_2	0.288	0.264	-170.8	-85.7	-51.8
	E_1+E_2	0.394	0.378	-363.7	-159.8	-75.7
	COV	27.8%	32.1%	27.2%	29.2%	31.4%
S4	E_1	0.028	0.119	-221.4	-324.8	-17.3
	E_2	0.108	0.137	-262.8	-329.1	-29.1
	E_1+E_2	0.110	0.149	-337.3	-348.2	-33.2
	COV	22.8%	25.2%	23.8%	26.1%	28.4%

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). The coefficient of variation COV concerns the Event (E_1+ $E_{2.}$).

In table columns (3)-(7) the mean values of the shown quantities and the COV concerning the Event (E_1+ E_2) are given. So, in table column (3) the Global Damage Indices DI_G and in table column (4) the Local Damage Index DI_L for the bending behavior of the element FK in frame (A) are given. Next, the maximum compressive impact-contact forces on the pounding regions G1 and G2 are given in the table columns (5) and (6), respectively. Finally, in the table column (7), the maximum horizontal top displacement $u_{top} = u_2^{(A)}$ of the second frame floor is given.

As the table values show, multiple earthquakes generally increase, in an accumulative way, the response quantities, e.g. critical displacements and damage indices. On the other hand, the strengthening of the frame (A) by 4 X-tie bracings (system S4 of Fig. 3) improves the response behaviour against seismic sequences. So, the mean values of the maximum horizontal top displacement $u_{top} = u_2^{(A)}$ of the second frame floor in S4 are smaller in comparison to ones of S0. These values can be further reduced by a parametric investigation of the cable-ties characteristics, e.g. by increasing their cross-sectional area $F_r$or investigating alternate cable-strengthening schemes.

4 Concluding Remarks

A stochastic computational approach has been presented, which can be effectively used for the probabilistic numerical investigation of the seismic inelastic behaviour of adjacent Cultural Heritage old RC framed structures. These structures are strengthened by cable elements in order to reduce pounding effects. 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, by using computed damage indices, the optimal cable-bracing scheme to reduce pounding effects can be selected in a parametric way among investigated alternative cable-strengthening schemes.

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