PROBABILISTIC METHODS FOR SEISMIC UPGRADING BY TIES OLD RC BUILDINGS CONSIDERING INPUT PARAMETERS UNCERTAINTY

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Abstract:
The probabilistic seismic response of existing old reinforced concrete (RC) structures is numerically investigated. These RC structures have been damaged due to extreme environment actions and are to be seismically upgraded by using cable elements (tension-ties). Special attention is given to uncertainty for the input parameters of the structural elements. For this purpose, use is made of Monte Carlo methods. A double discretization, in space by the Finite Element Method and in time by a direct incremental approach, is applied. The unilateral behaviour of the cable-elements, which can undertake tension stresses only, is strictly taken into account and results to inequality constitutive conditions. Finally, using damage indices, the probabilistic optimal strengthening version of cable-systems is chosen for the case of multi-storey RC frames under earthquakes sequences.

***Key words**: Seismic Upgrading, Strengthening by Ties, Uncertainty of Input Parameters, Monte Carlo methods, Multiple Earthquakes.*

1. Introduction

Old existing reinforced concrete (RC) structures, which are subjected to non-usual extremal actions (seismic, environmental etc.), appear usually a significant strength degradation and damages. To overcome such strength degradation effects, various repairing and strengthening procedures can be used for the seismic upgrading of existing RC buildings [1,2].

Among the seismic upgrading methods, cable-like members (ties) can be used as a first strengthening and repairing procedure [3]. It must be emphasized that the cable-members can undertake tension, but buckle and become slack and structurally ineffective when subjected to a sufficient compressive force. So, in the mathematical problem formulation, the constitutive relations for cable-members include also inequality conditions, and the problem becomes a high nonlinear one. For the strict mathematical

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treatment of the problem, the concept of variational and/or hemivariational inequalities can be used and has been successfully applied [4]. As concerns the numerical treatment, non-convex optimization algorithms are generally required [3,4].

Further, for the seismic analysis of such old RC structures, an estimation of the input parameters must be realized taking into account a lot of uncertainties [8,12-15]. These uncertainties mainly concern the holding properties of the old materials which 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.

The present study deals with a computational probabilistic approach for the seismic analysis of old RC frame-buildings, which have to be strengthened by cable elements and to be subjected to seismic sequences [6,7]. Special attention is given for the estimation of the uncertainties concerning structural input parameters. For this purpose, Monte Carlo techniques are used [8-11]. Damage indices [14] are computed for the seismic assessment 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. Method of analysis

The probabilistic approach for the seismic analysis of old existing RC frame-buildings may be obtained through Monte Carlo simulations. As well-known, see e.g. [8-13], 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 for each set of the random input variables and finally realizing a statistical analysis.

Details of the methodology concerning the deterministic problem and the probabilistic aspects are given shortly in the next sections.

2.1. Problem formulation and numerical treatment of the deterministic problem

Numerical methods for the problem treatment have been presented recently, see [3]. Briefly, the RC frame structural system is discretized in space by using finite elements. The usual frame elements are used for the reinforced concrete frame. On the other hand, for the cable strengthening system, pin-jointed bar elements are used. The behaviour of both, the cable elements and the non-linear RC elements, includes loosening, elastoplastic or/and elastoplastic-softening-fracturing and unloading - reloading effects as depicted in Fig. 1. All these characteristics can be expressed mathematically by non-convex relations of the general form:

(2.1) $$s_i\,(d_i) \in \hat{\partial}\, S_i(d_i)$$

Here s_i and d_i are generalized stress and deformation quantities, respectively, $\hat{\partial}$ is the generalized gradient andS_i is the superpotential function, see Panagiotopoulos [4].

Next, when the cable-elements are taken into account, the dynamic equilibrium for the structural system is written in incremental matrix notation:

(2.2) $$\boldsymbol{M}\ \Delta\ddot{\boldsymbol{u}} + \boldsymbol{C}\ \Delta\dot{\boldsymbol{u}} + (\boldsymbol{K_T} + \boldsymbol{G})\ \Delta\boldsymbol{u} = -\boldsymbol{M}\,\Delta\ddot{\boldsymbol{u}}_{\boldsymbol{g}} + \boldsymbol{T}\ \Delta\boldsymbol{s}$$

Here **u** and **f** are the displacement and the loading forces time dependent vectors, respectively. The damping and tangent-stiffness terms, ***C*** and $\boldsymbol{K_T(u)}$, respectively, concern the general non-linear case. By **s** is denoted the cable elements stress vector and ***T*** is a transformation matrix. ***G*** is the geometric stiffness matrix, by which P-Delta effects are taken into account. Dots over symbols denote derivatives with respect to time.

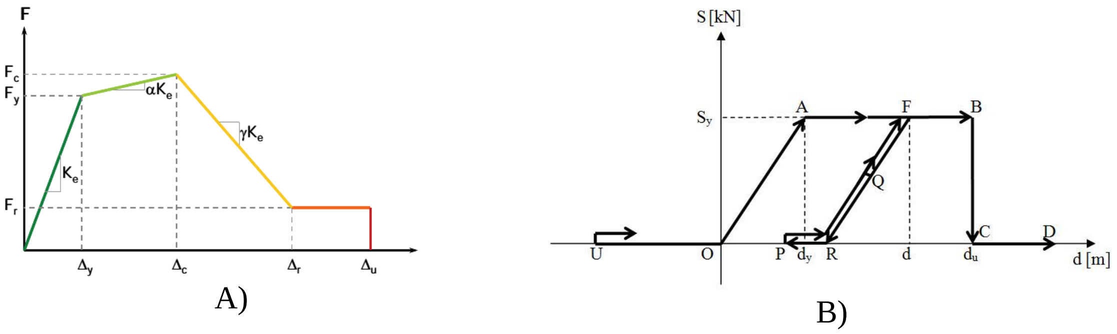

Fig. 1. The constitutive laws: A) generalized force- generalized displacement piecewise linearized constitutive diagramme (backbone), B) unilateral behavior of the cable-elements.

The system of the above relations (2.1)-(2.2), combined with the initial conditions, provide the problem formulation, where, for given **f** and/or $\ddot{x}_g$, the vectors **u** and **s** have to be computed. Relevant computational approaches are described in [3]. After piecewise linearization of the above constitutive relations (2.1), in each time-step Δt a relevant non-convex linear complementarity problem of the following matrix form is eventually solved :

(2.3) $\quad \mathbf{v} \geq \mathbf{0}\,, \qquad \mathbf{A\,v + a} \leq \mathbf{0}\,, \qquad \mathbf{v}^{\mathbf{T}}\,.\,(\mathbf{A\,v + a}) = 0.$

Here **v** is the vector of unknown unilateral quantities at the time –moment t, $\mathbf{v}^{\mathbf{T}}$ is the transpose of **v**, **a** is a known vector dependent on excitation and results from previous time moments (t- Δt), and **A** is a transformation matrix.

Concerning the incremental approach, see eq.(2.2), on this is based the structural analysis software Ruaumoko [5]. This software uses the finite element method and permits an extensive parametric study on the inelastic response of structures.

The decision about a possible strengthening for an existing RC structural system can be taken after an assessment realization [1-3]. Here the assessment is based on a relevant evaluation of suitable damage indices [3, 14]. After Park/Ang [14], the *local* damage index DI_L is computed by the following relation:

(2.9a) $$DI_L = \frac{\mu_m}{\mu_u} + \frac{\beta}{F_y d_u} E_T$$

where: μ_m is 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, and d_u the ultimate generalized deformation.

The Park/Ang *global* damage index is given by the following relation:

(2.9b) $$DI_G = \frac{\sum_{i=1}^{n} DI_{Li} E_i}{\sum_{i=1}^{n} E_i}$$

where DI_{Li} is the local damage index and E_i the energy dissipated at location *i* , and *n* 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 response of the considered RC buildings, the Monte Carlo simulation is used [8-13, 15-18]. As mentioned, 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 using the technique of Latin Hypercube Sampling (LHS), see [16-18]. The LHS is a selective sample technique by which, for a desirable accuracy level, the number of the sample size is significantly smaller than the direct Monte Carlo simulation.

3. Numerical example

3.1. Description of the considered RC structural system.

The old industrial reinforced concrete frame F0 of Fig. 2 is considered to be upgraded by ties and subjected to a multiple ground seismic excitation [6,7].

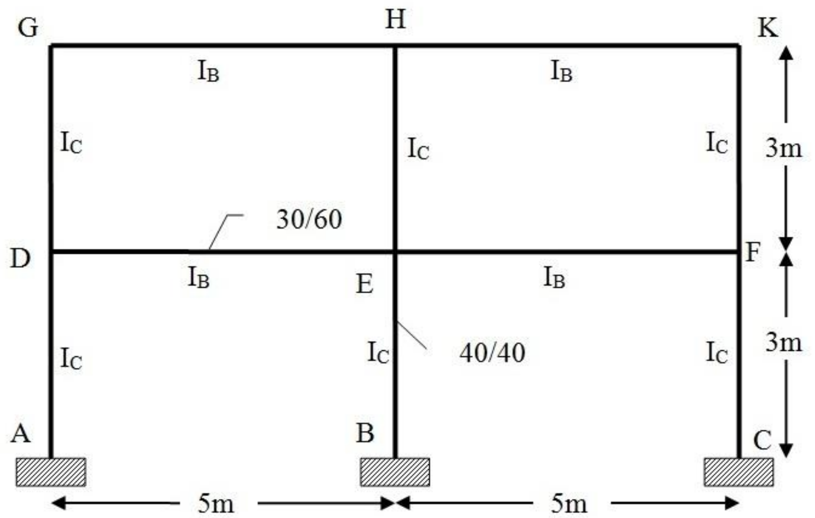

Fig. 2. System F0: the old RC frame without cable-strengthening

The estimated concrete class is C12/15 and the steel class is S220. As mentioned, according to JCSS (Joint Committee Structural Safety), see [15], concrete strength and elasticity modulus follow Normal distribution and the steel strength follows the Lognormal distribution. So the statistical characteristics of the input random variables concerning the building materials are estimated to be as shown in Table 1. The mean/median values of the random variables correspond to the best estimates employed in the deterministic model according to Greek Building Concrete Code EKOS2000.

Further, for the frame assessment, probabilistic section models are derived by using the data of the Table 1. The procedure is shown qualitatively in Fig. 3.

Table 1. Statistical data for the building materials treated as random variables

	Disribution	mean	COV
Compressive strength of concrete	Normal	8.0 MPa	15%
Yield strength of steel	Lognormal	191.3 MPa	10%
Initial elasticity modulus, concrete	Normal	26.0 GPA	8%
Initial elasticity modulus, steel	Normal	200 GPA	4%

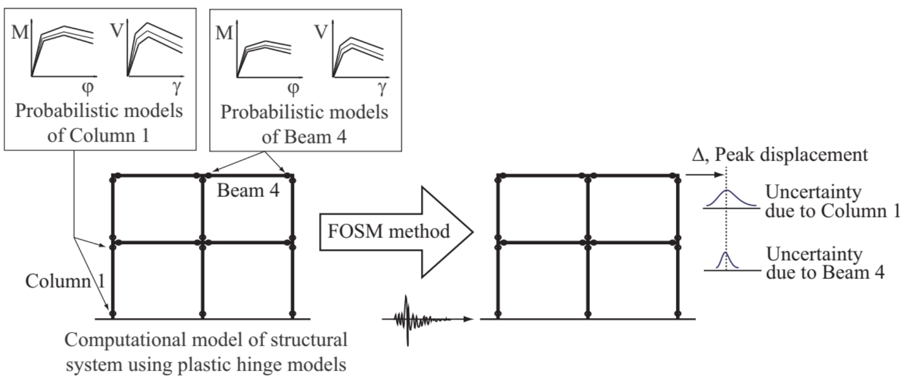

Fig. 3. System evaluation procedure using probabilistic section models

Due to various extreme actions (environmental etc.), corrosion and cracking has been taken place, which has caused a strength and stiffness degradation. The stiffness reduction due to cracking [1-3] results to effective stiffness of 0.60 I_g for the two 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 [5], the columns and the beams are modeled using prismatic frame elements [3]. Nonlinearity at the two ends of RC members is idealized 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.

The frame-system F0 of Fig. 2 was initially without cable-bracings. After the seismic assessment [3, 14], it was decided the frame F0 to be strengthened by ties. The X-cable-bracings system, shown in Fig. 4, has been proposed as the optimal one in order the frame F0 to be seismically upgraded. The system of the frame with the X-bracing diagonal cable-elements shown in Figure 4 is denoted as system F4.

The cable constitutive law, concerning the unilateral (slackness), hysteretic, fracturing, unloading-reloading etc. behavior, is depicted in Fig. 1(B). The cable elements have a cross-sectional area F_c = 18 cm^2 and they are of steel class S220 with elasticity modulus E_c = 200 GPa.

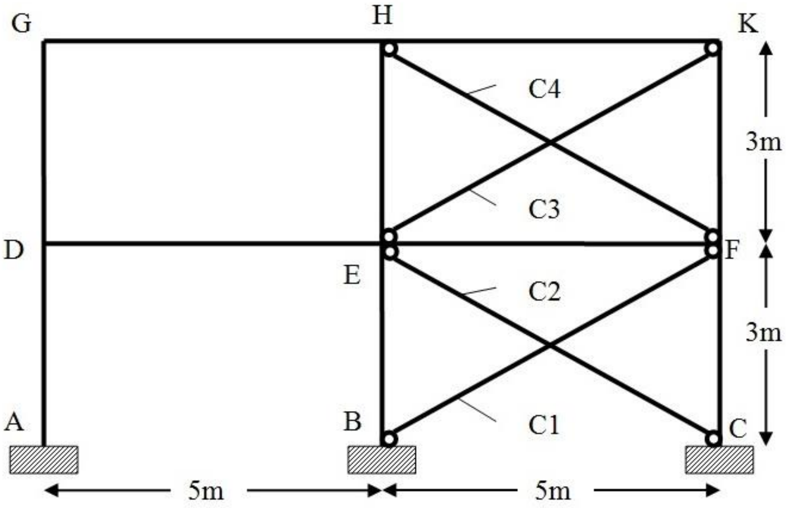

Fig. 4. System F4: the old RC frame cable-strengthened with X-bracings

3.2. Earthquakes Sequences Input

A list of multiple earthquakes, downloaded from the strong motion database of the Pacific Earthquake Engineering Research (PEER) Center [6, 7], appears in Table 2.

Table 2. 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

3.3. Representative probabilistic results

After application of the herein proposed computational probabilistic approach, some representative results are shown in Table 3. These results concern the Coalinga case of the seismic sequence of Table 2.

Table 3. Representative probabilistic response quantities for the systems F0 and F4

SYSTEM	EVENTS	DI_G		u_{top} [cm]	
		Mean value	COV	Mean value	COV
(1)	(2)	(3)	(4)	(5)	(6)
F0	Event E_1	0.176	14.8%	2.48	12.7%
	Event E_2	0.385	13.7%	3.87	11.8%
	Event (E_1+E_2)	0.442	15.7%	4.64	13.4%
F4	Event E_1	0.082	14.4%	1.36	12.9%
	Event E_2	0.107	12.8%	1.74	11.8%
	Event (E_1+E_2)	0.128	15.2%	1.88	14.7%

In column (2) of the Table 3, 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 column (5) and in the column (6) the mean value and the coefficient of variation COV of the absolutely maximum horizontal top roof displacement u_{top} , respectively, are given.

As the values in the Table 5 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 F4 of Fig. 4) improves the response behaviour.

4. Concluding remarks

Imput parameters uncertainty is taking into account in the herein presented numerical approach concerning the probabilistic seismic analysis of old RC structures. As the results of a numerical example have shown, the probabilistic optimal strengthening version of cable-bracings can be decided by computing damage indices. Thus, the proposed methodology can be used for the effective probabilistic investigation of the inelastic seismic behaviour of existing old RC building systems, which are to be seismically strengthened by cable elements under seismic sequences.

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