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ON THE SEISMIC RISK OF INELASTIC REINFORCED CONCRETE FRAMES

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SUMMARY

In order to assess the seismic risk of a given type of structure subjected to earthquakes one needs not only proper seismological information but also a convenient approach for damage estimation. In this paper an efficient procedure for estimation of the expected damage in plane R/C frames is proposed which permits to calculate the expected losses. The same procedure enables us to see how the quality of the building materials influences the costs for the repairs of the structure needed.

INTRODUCTION

The seismic risk evaluation of a given type of building structures is bound to the problem of optimum design in regions with possible seismic activity and has a great importance from economical point of view. S. Kisliakov [1] proposed a method of damage risk and reliability assessment of nonlinear structures. The possibility $p(a_i)$ for an earthquake with peak ground acceleration (PGA) to occur will be given by the seismologists. It depends on the type of seismic excitation and on the available seismic information about the region. According to [1] the following equation gives the expected damage for a wide-band earthquake loading :

(1) $E[D] = \sum_i E[D/\alpha_i]\, p(\alpha_i)$,

here $E[D/a_i]$ is to be determined from each accelerogram of a given set with a corresponding duration of the strong part of the record. The expected damage for a given PGA and duration may be expressed in different terms, for instance in terms of relative costs. This latter criterion has been accepted by the authors. A good estimation of the damage level is given by the cost ratio (the costs of repairing the damaged structure divided by the price of the new structure). Obviously there are different technologies and prices in different countries and yet such a criterion is very convenient, The qualitative description of the possibie damages found through numerical analysis involving recorded or artificially generated accelerograms gives also an important information about the seismic risk.

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In this paper we propose an approach for damage calculation with some qualitative and quantitative information about the numerical analysis performed. This approach as shown below enables us to understand also how the quality of materials influences the possible damages.

MATHEMATICAL MODEL OF THE STRUCTURE

We shall examine the behaviour of multistorey plane R/C frame structures with rigid joints and rectangular cross section of the members Using the finite element philosophy the frame is thought to be a system of nodes, rigidly connected by beams and columns. The axial deformations are neglected. The mass is supposed to be concentrated in the nodes (Fig. 1). The structure is first computed according to the

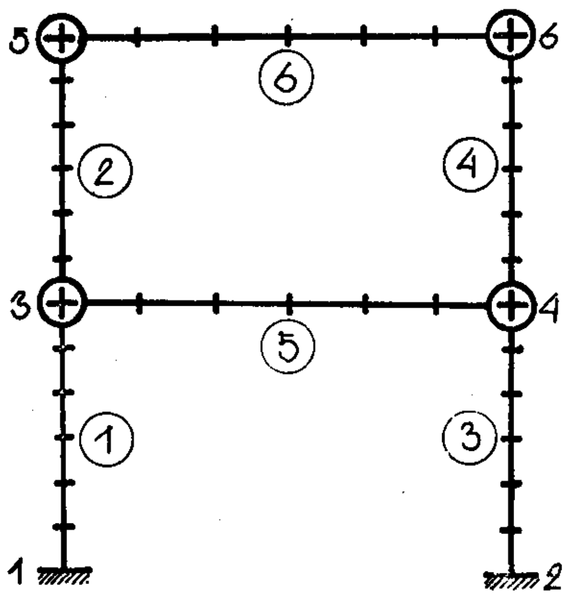


Fig. 1 Scheme of the examined frame

Bulgarian National Building Code for earthquake regions - the number forces and corresponding reinforcement are determined. For each examined section a trilinear hysteretic model is constructed depending on the section size. On the stress - strain curves of the concrete and of the reinforcement. and on the axial forces from the dead load (Fig. 2). The hvsteretic models (M—x) offer the opportunity

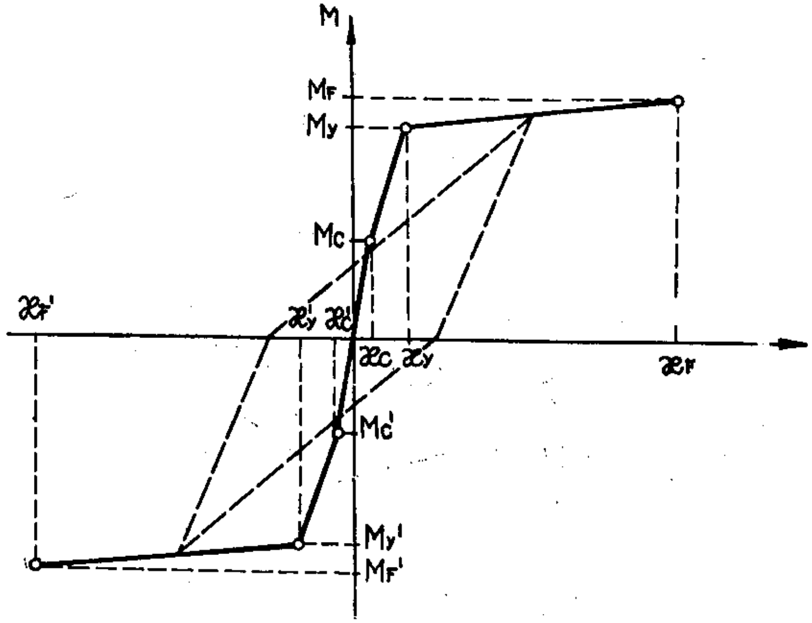
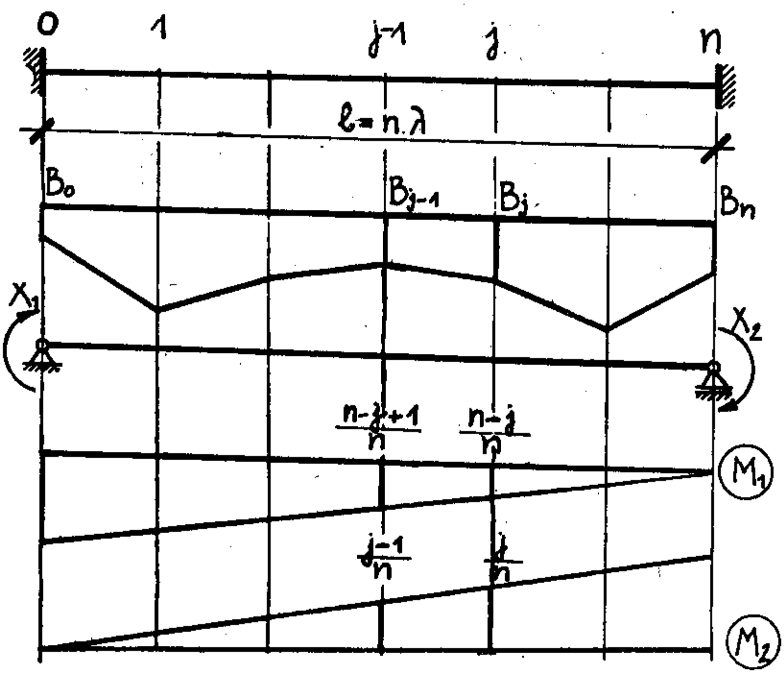


Fig. 2 Trilinear hysteretic model

for calculating the sectional stiffness in the time domain. Such a model reoresents the moment-curvature relationship, and has a svmmetric or unsymmetric trilinear primary curve with typical points at cracking, yielding and failure in the positive and negative ranges. The computation of coordinates of these points i. e. the design of the hvstetic models and the rules of the stiffness determination are published in [2]. The experiments with dynamically loaded columns carried out in the Laboratory of reinforced concrete structures at Higher Institute of Civil Engineering and Architecture in Sofia, showed negligible difference between the theoretically and experimentally determined stiffnesses. The analysis of the experimental work enables us to accept that if the failure moment in a section is reached, the stiffness becomes equal to zero and in that section a hinge appears. In such a case the number of the frame degrees of freedom is to be changed and after balancing the nodal forces the computation continues as long as the structure becomes a mechanism (or to the end of seismic action). Between any two cross-sections the stiffness B(x)is supposed to be linear function. (See Fig. 3)

(2) $B(x) = \frac{n}{l}(B_j - B_{j-1})\,x + B_j - j\,(B_j - B_{j-1})$

If the stiffness in n sections of an element is known, the element stiffness matrix can be easily calculated. (Fig 3) The stiffness matrix of the whole structure can be formed by means of the well



Fie. 3. Diagrams used in element stiffness calculation

known Finite element techniques for every point of time from the beginning of the seismic action.

The structure damping is expressed as recommended in [3] according to the theory of Ravleigh as a linear combination of the mass and initial stiffness matrices and scalar coefficients.

ANALYSIS PROCEDURE

The dynamic equilibrium of the frame structure is described by set of ordinary differential equations following from D' Alembert principle. We solve this system in the time domain. For the increments of the kinematic quantities displacements $\Delta \tau$, velocities $\Delta \dot{\tau}$ accelerations $\Delta \ddot{\tau}$ and the ground accelerations $\Delta \ddot{\tau}_g$ this system of equations can be written as follows :

(3) $M \Delta \ddot{\tau}_i + C \Delta \dot{\tau}_i + K_i \Delta \tau_i = -M \Delta \ddot{\tau}_{gi}$,

where the subscript i denotes the number of the time step. At the actual computer realisation of this procedure some different possibilities for step-by-step integration can be used. One can choose among the methods of Newmark, Hilber, Wilson, the methods of the Constant or Linear acceleration. All of them are implicit and unconditionally stable. Similar to [4] we can write Eq. (3) in the form :

(4) $M\Delta \ddot{\tau}_i + C\Delta\dot{\tau}_i + (1+\alpha) K\Delta\tau_i + \alpha K\Delta \tau_{i-1} = -M\Delta \ddot{\tau}_{gi}$.

This equation can be expreseed with respect to the increments of the disolacements. They can be found from the linear svstem of equations :

(5) $\bar{K}\Delta\tau_i = \Delta \bar{R}_i$,

where

(6) $\bar{K} = a_1 M + B_1 C + C_1 K$

and

(7) $\Delta \bar{R}_i = M (a_2\dot{\zeta}_{i-1} + a_3\ddot{\zeta}_{i-1} - \Delta \ddot{\zeta}_{gi}) + C (B_2\dot{\zeta}_{i-1} + B_3\ddot{\zeta}_{i-1}) + C_2 K \Delta\zeta_{i-1}$

The coefficients a_i, b_i and c_i are to be calculated with the formulae which match best the step-by-step integration method chosen. The increments of the velocities and the accelerations will be :

(8) $\Delta\dot{\zeta}_i = B_1\Delta\zeta_i - B_2\dot{\zeta}_{i-1} - B_3\ddot{\zeta}_{i-1}$

(9) $\Delta\ddot{\zeta}_i = a_1\Delta\zeta_i - a_2\dot{\zeta}_{i-1} - a_3\ddot{\zeta}_{i-1}$,

respectively. An especially important step in the computational procedure is the determination of the disolacements through successive appoximations. The so-called "state determination", which is to be performed during the structure analysis under long term loadings (in the beginning of the procedure) and, if necessary, at each time step as well. For the static load or for a separate time step a constant stiffness successive scheme is used. To accelerate the iterations by the step-by-step analysis the stiffness can be chosen in the beginning of the step. We are going to describe the i-th iteration from the i-th time step (Fig. 4)

$\bar{K}^T_{i,i-1}$ —Tangent stiffness at the beginning of the iteration

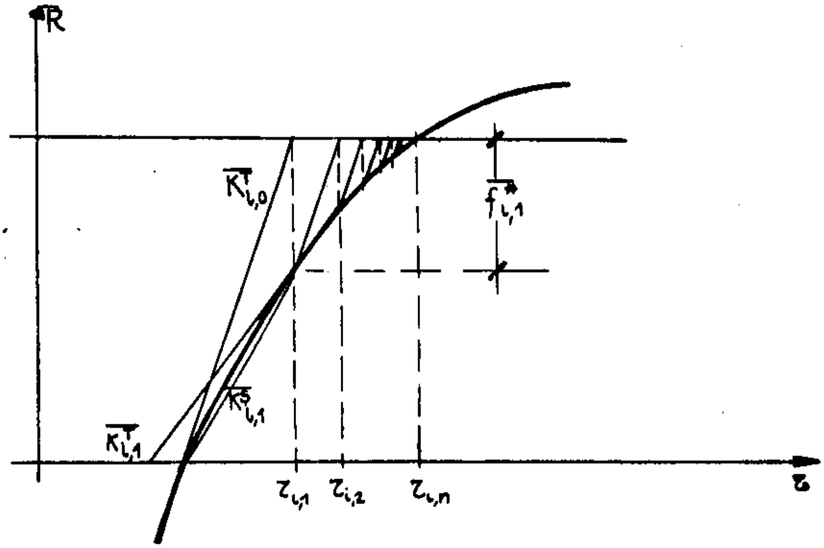


Fig. 4. Iterations in the step i of the step-by-step analysis

—Tangent stiffness at the end of the iteration $\bar{K}^T_{i,j}$

—Transversal stiffness matrix :

(10) $$\bar{K}^s_{i,j} = 0,5\left(\bar{K}^T_{i,j-1} + \bar{K}^T_{i,j}\right)$$

The three matrices mentioned above are effective. They are formed according to Eq. (6).

– The increments of the displacements at the end of the iteration are computed from the linear set of equations :

(11) $$\bar{K}^T_i \, \Delta\zeta_{i,j} = \bar{f}^*_{i,j-1},$$

where $\bar{K}^T_i$ is constant for all the iterations of the i-th time step. This matrix is chosen depending on the iteration number in the (i-1) - th step and on the ground acceleration increment $\Delta \ddot{r}_{gi}$, $\bar{K}_{iT}$ may be equal to $\bar{K}_{T\,i-1,o}$. Usually the step size t is very small (0.02 to 0.05 sec) and so is the incremental earthquake load $M \Delta \ddot{r}_{yi}$. As a result the number of iterations does not require an acceleration of the procedure. In the opposite case the authors provide an algorithm for acceleration of the iterations which will not be discused here.

- The unbalanced internal forces $F_{i,j}$ are :

(12) $\bar{F}^{*}_{i,j} = \bar{F}^{*}_{i,j-1} - \bar{K}^{S}_{i,j} \Delta X_{i,j}$,

Where

(13) $\bar{F}^{*}_{i,0} = \Delta \bar{R}_{i}$

are computed at the end of the last iteration. The iterations in the considered time step are stopped when $\bar{F}^{*}_{in} \rightarrow O$, i, e, when the equilibrium of external and internal forces is reached.

The increments of the displacements for the whole step are :

(14) $\Delta \tau_{i} = \sum_{j=1}^{n} \Delta \tau_{i,j}$

After that the velocity and acceleration increments are computed according to Eqs. (8) and (9) for the end of the time step. The bending moments in the examined crss-sections are specified at each iteration, and at the end of the time step the nodes have to be in equilibrium. These moments are used for the computation of the section stiffnesses from the hysteretic models and for the following arrangement of the matrices $\bar{K}^{T}_{i,j}$ and $\bar{K}^{S}_{ij}$

NUMERICAL EXAMPLE FOR DAMAGE ASSESSMENT OF A PLANE FRAME STRUCTURE

According to the structural idealisation and the computational procedure a computer program has been developed which permits a a nonlinear analysis of the structure under earthquake load to be performed. The section damages are assessed for each time step. We are going to describe briefly a numerical example :

The frame structure from Fig. 1 was computed after Bulgarian National Seismic Code for VIII-th degree input of the Medvedev-Sponheuer-Karnik (MSK) scale, which contains 12 degrees. First the reinforcement was determined. For each member of the frame the stiffness was examined for 7 cross-sections. For this purpose the corresponding hysteretic models (trilinear primary curves) were constructed. Then the frame was computed for seismic loads, specified by accelerograms of the following earthquakes (Ei-conditional identifying number of the accelerogram accepted) :

– Ei—Parkfield, California, earthquake of 27. July 1966, PGA=3.478 m/s*s

E—2—Imperial Valley earthquake, E1 Centro, California, 18 May 1940 PGA=3.417 m/s*s

—E3—San Fernando, California, earthquake of 9 February 1971, PGA=11.841 m/s*s

– E4—Brzece, Yugoslavia, earthquake of 23 May 1980, PGA=1.064 m/s*s.

These accelerograms represent wide-band earthquake loadings. Obviously the E3 accelerogram should cause a considerably higher damage or even a failure, and E4 should simulate a slight earthquake. If we wish to assess the damage in terms of spectral characteristics of the accelerograms we should normalize all them to a PGA suitable for the VIII-th degree of the MSK scale. H. C, Shah [5] points out that the VIII-th degree of the MSK-scale corresponds to PGA from 1.0 to 2.0 m/s*s. Because of this all the accelerograms used were normalized to a PGA=2.0 m/s*s, so that under E1, E2, E3 and E4 below we shall understand the normalized accelerograms. The frame from Fig. 1 is computed for each accelerogram for two values of the concrete strength Case I-design strength and Case II-the strength of the concrete is calculated from experimental data. The Newmark's method has been used for the step-by-step integration with $\beta=0.36$ and $\gamma=.70$ (See [4]). The integration step was $\Delta t=0.04$ sec. One can obtain from Table 1 a output of the numerical damage analysis. The number of sections is given (42 examined sections altogether) where cracking of concrete has appeared and further the number of sections with yielded reinforcement ("C" means cracking and "Y"—yielding).

Table : 1

accelerogram	Case I	Case II
Z1	24 C	26C + 1Y
Z2	31C + 3Y	30C + 4Y
Z3	30C + 2Y	29C + 2Y
Z4	26 C	27C

Of great interest is the quantitative assessment of the damages. It is best displayed by the expenses of the damage repairs. These expenses could be calculated if we assume the damage in one section to refer to a whole zone around the section. Further, it is necessary to choose a technology for the repairs. The expenses of the restoring depend on the technology and on the prices of the materials and the workman ship. The quantity of MDR is defined as :

$$(15)\quad MDR = \frac{\text{price af repairs}}{\text{Price of the new structure}}$$

seems to be a good quantitative measure of damage. Table 2 shows the values of MDR for both cases of concrete strength. The same results are displayed in Fig.5.

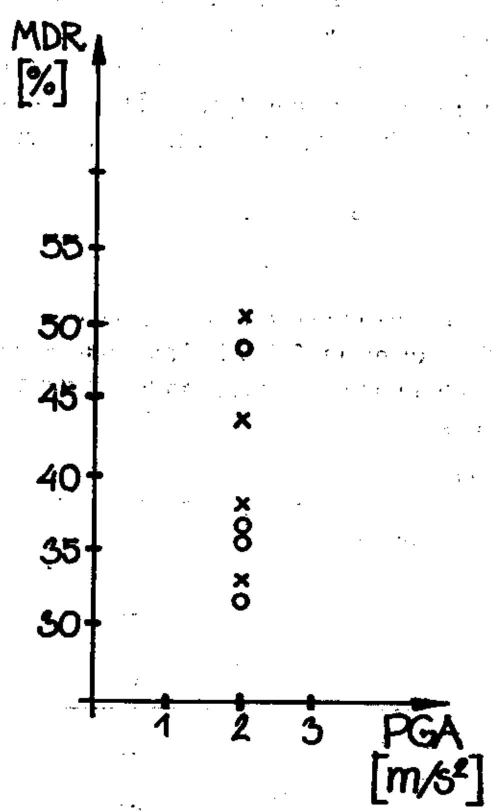


Fig. 5. MDR values in two cases of concrete strength

Table : 2

accelerogram	Case I	Case II
Z1	36.8%	37.6%
Z2	48.6%	50.7%
Z3	36.4%	43.8%
Z4	31.6%	32.6%

ON THE INFLUENCE OF THE QUALITY OF BUILDING MATERIALS

The proposed procedure of damage risk assessment by numerical analysis allows to study the influence of the quality of building materials on the seismic risk. The results for Case II in Table 1 and 2 refer to experimentally calculated strength of the concrete which is lower then the design strength. This lower strength enters the hysteretic models and the numerical analysis and causes heavier damage than in Case I. A measure of the influence of quality of the concrete on seismic risk is the k coefficient defined in [5] :

$$k = \frac{\text{MDR (II) - MDR (I)}}{\text{MDR (I)}} \tag{16}$$

The subscript I or II denotes the corresponding case of concrete strength. The values of k are given in Table 3. The mean value of k is 0.083 which is calculated with an analysis involving four accelerograms normalized to PGA=2.0 m/s*s.

Table : 3

accelerogram	k
Z1	0.022
Z2	0.043
Z3	0.203
Z4	0.080

More reliable conclusions about the influence of the concrete quality on the seismic risk are possible only after a more extensive analysis similar to that described in this paper. For instance, it is necessary to investigate multistorey frames for a great number of accelerograms. Further, other structures than frames, and other building materials than concrete should be involved.

REFERENCES

1. Kisliakov, S.D., Towards a General Theory of Seismic Risk Assessment, Proc., 8-th European Conference on Earthquake Engineering, 1986, Lisbon, Portugal.

2. UNDP, Earthquake Risk Reduction in the Balkan Region, working Group E, 1983, Skopje.

3. Newmark, N., Concept to Complex Stiffness Applied to Problems of Oscillations with Viscous and Hysteretic Damping, Aeronautical Research Council Reports and Memoranda, 1962.

4. Row, D.G., Powell, G.H., A Substructure Technique for Non-linear Static and dynamic Analysis, University of California, Rep. N UCB/EERC 78-15, Aug. 1982, Berkeley.

5. Shah, H.C., Earthquake Engineering and Seismic Risk Analysis, CE 282 B, Standford University, 1982.