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Seismic Response Analysis of Inelastic Non-Stationary Structural Systems

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ABSTRACT

Most structures under strong earthquakes have substantial nonlinear features. Therefore for seismic response analysis and design models, elastic-plastic properties are used. The inelastic behaviors of structures are characterized by development of damages resulting in reduction of rigidity and changes in dynamic parameters. For such inelastic behavior we recommend inelastic non-stationary design models, which is an incremental procedure by taking into consideration the inelastic deformation of the loading history. In other words, this procedure takes to account for the damage accumulation of the structure during the earthquake. For most structural systems these models can be realized from the generalized "force - displacement" relationships (diagrams of elastic-plastic deformation) with variable characteristics. The changes of dynamic parameters of structures are function of three directional components of the earthquake ground motions. The proposed approach is implemented with the help of RUPS software. Examples of structural behavior under strong earthquakes are given.

INTRODUCTION

Earthquake response of building structures depends on both the ground motion characteristics and the dynamic properties of the structures. Design of seismic resistant system depends on what type of structural system is analyzed and what method of analysis is used. The linear systems approach sometimes can not cover the responses due to strong earthquake ground motions, and inelastic approach must be employed to obtain more accurate responses. But in reality most of structures have finite stability and at strong earthquake don't behave as linear system. At the time of design earthquake structural damage is expected.

Past major earthquakes show that many buildings and structures (even of modern design) are damaged so that there were changes in their rigidity, strength and energy dissipating characteristics. These phenomena are observed in reinforced concrete, masonry and large panel buildings. To study seismic resistance of such systems design models with variable parameters were used [1, 2, 9]. Nonlinear non-stationary design model for analysis of reinforced concrete frame system subjected to strong seismic motions was used in [10]. Damage accumulation models were used in [4, 8, 10].

INELASTIC BEHAVIOR OF STRUCTURES

As a general rule, failure of structures and structural elements is the result of damage accumulation in the process of dynamic loading and can be described by certain elastic-plastic relationship. In the cases of brittle failure the property of structural elements could be described by certain limiting force (shear force, main stretch stresses or eccentric compression).

The equations of motion of inelastic system can be described by using force displacement relations (diagrams of deformation). Properties of diagram depend from many factors which are functions of the type of structures.

As was establishing by numerous tests the diagrams for steel, aluminum and more metal alloys have permanent property, independent of cycles of elastic-plastic deformation. For this reason behavior of steel structures, subjected to strong seismic motions, can be described based on of non-elastic stationary models. However, for most other type of structures, such as reinforced concrete structures more complex behaviors exist.

To study the properties of reinforced concrete moment resisting frames at a stage close to failure a cycle of experimental work is carried out at quasi-static and dynamic loads of high intensity [7, 9]. All elastic-plastic dependencies of force-deformation response had distinct non-stationary character and reflected the changing of rigid property at the process of cyclic elastic-plastic deformation. The same results were obtained by many investigators, who conducted experiments of RC structures and structural elements in elastic-plastic stage.

To describe the properties of RC structures at a stage, near ultimate strength, the results of tests were analyzed based on the damage accumulation concept. For analytical description of inelastic properties of structures the rigidity in n-th inelastic semi-cycle rigidity $C_1^{(n)}$ and damage criterion $\gamma^{(n)}$ (sum of relative plastic deformations accumulated by the system during previous cycles) were taken as initial data. The experimental force-deflection relationships were described by multi-linear, modified Ramberg-Osgud and exponential diagrams, the change of the parameters depends on the damage accumulation parameter $\gamma^{(n)}$. The Eq. for multi-linear diagram is given by:

$$C_{k_1}^{(n)} = C_{k_1}^{(1)} e^{-a_k Arcsh\, \gamma_k^{(n)}} \qquad (1),$$

where: $\gamma_k^{(n)} = \left| \sum_{i=1}^{n-1} \alpha_k^{(i)} \right|$ - damage parameter; $\alpha_k^{(i)}$ - relative plastic deformations on i-th plastic semi-cycle. Parameters of $C_{k_1}^{(1)}; a_k; \alpha_k^{(i)}$ depend on properties of structures (strength, rigidity) and experimental data.

Comparison of experimental and design diagrams show that the proposed relationships suitably describe the processes of elastic-plastic deformation of reinforced-concrete structures, subjected to strong seismic motions.

The results of analysis of many earthquake show that the behavior of structures is significantly effected by the vertical component of the seismic ground motions near the epicenter areas. This is supported by analysis of earthquakes in Tashkent (1966), Gazli (1976 and 1984), San Fernando (1971), Spitak (1988). At the same time the vertical motion do not cause major damages of load-bearing structures. In general horizontal motion affect building responses more significantly.

The effect of axial forces on the natural frequency of structures became noticeable when they are of the value approaching the critical buckling load level. Since this is not typical in real design, the effect of axial force on dynamic parameters may be neglected .

However, this effect should be taken into account in deriving the inelastic force-deformation relationships (e.g. the moment-curvature diagrams).

In most cases the vertical load does not effect the frequency and modes of building vibrations and hence rigidity of the system at all motion stages can be taken as constant. On the other hand, experiments with eccentrically compressed reinforced concrete members showed that the change in axial load substantially effects the shape of the inelastic behavior after cracking took place that reduces plasticity of the structures.

The elastic-plastic relationships (loading – deformation) may be described by exponential diagrams as follows. Designing the system for vertical forces action, the value and direction of vertical load $P(t)$ is determined and then depending on it the ultimate elastic reaction of the system $R^{(n)}(t)$ is determined at a given time moment t by the Eq. (2):

$$R_e(t)= R_e^1[1 \pm \frac{P(t)}{P}] \quad (2),$$

where $R_e^{(1)}$ - ultimate elastic reaction of the system ignoring vertical component, $P(t)$ - vertical load at t moment, P - constant vertical load.

Having the value of $R_e(t)$, it is possible to determine the parameters of the exponential relationship.

Thus, deformation diagram of elastic-plastic systems with varying rigidity parameters and vertical component of seismic process may be written as [9]:

$$R^{(n)}(t)= R_p\{1-e^{-[a\frac{y^{(n)}(t)}{y_e^{(n)}}+b(t)(\frac{y^{(n)}(t)}{y_e^{(n)}})^2]}\} \quad (3).$$

Where the rigidity of the n semicycle is determined by Eq. (1).

ANALYSIS OF INELASTIC SYSTEMS AT MULTICOMPONENT GROUND MOTIONS

In a number of cases it is necessary to use complex design models to account for three-dimensional characteristics of structures and multidirectional seismic forces. The main purpose of the approach is the determination of peculiarities in the elastic-plastic deformation responses which affect limit state parameters of inelastic structures. Therefore, the obtained results were compared with those obtained by using a common stressed state. For this purpose the physical parameters of the structure are assumed.

The buildings which had the periods of free vibrations in the direction of main axes T_x = T_y = 0.5; 1.0; 1.5; 2.0 and 2.5 sec have been considered. The masses were adopted equal to 10 ts2/m while rigidity was selected so that vibration periods are consistent with the above. The coefficients of inelastic resistance were taken equal to 5 % of the critical one. Analysis was performed by using the RUPS-3K program.

The input ground motions used are those having records of three components: 8-3 (El Centro); 8-1 (Vernon); 8-6 (Taft); 8-8 (Eureca); San Fernando; Gazly and 7-25 (Ferndal). Design sample of the earthquakes and the results of processing of the main accelerogram characteristics are given in Table 1. In the same Table normalization factors reducing accelerogram intensity to the maximum horizontal component (8-3, G-52) and vertical one (8-3, Z) of a known earthquake El Centro, 1940 are also given.

Thus for all the earthquakes the relationship between vertical and horizontal components is the same as at El Centro earthquake About 0.8. At this for the sample on the whole the intensity of vertical vibrations is somewhat overestimated (especially for 7-25 and 8-8 earthquakes) and therefore when analyzing the effect of vertical component on system motion parameters, the variants for which the intensity of vertical component changed as compared with the main sample were considered.

Table 1. Design sample of accelerograms at the multicomponent earthquake action.

Eartquake	Accelero-gram	Maximum Spectra of Reactions, T_R, sec	Duration, $t_{0.3}$, sec	Standard, σ_0, cm/s2	Normalized coefficient K_n
El-Centro, 8-3	8-3, G-52	0.25 - 0.4	11.0	84.8	1.0
	8-3, G-38	0.25 – 0.55	11.0	65.0	1.3
	8-3, Z	0.1	11.0	66.3	1.0
Vernon, 8-1	8-1, G-33	0.3 – 0.9	11.0	32.0	2.65
	8-1, G-57	0.3	11.0	27.0	3.15
	8-1, Z	0.2	11.0	15.4	4.3
Ferndal, 7-25	7-25, G-40	0.4 - 1.55	11.0	39.1	2.2
	7-25, G-50	0.2 -1.4	11.0	30.8	2.75
	7-25, Z	0.45 - 0.8	11.0	6.3	10.5
San-Fernando	S-F, G16	0.40	9.8	197.0	0.43
	S-F, G-74	0.1 – 0.5	9.0	195.0	0.43
	S-F, Z	0.1 - 0.95	11.0	200.0	0.33
Gazli	G, N-S	0.1	10.4	131.0	0.65
	G, E-W	0.1 - 0.55	10.0	141.7	0.60
	G, Z	0.1	11.0	264.3	0.25
Eureka, 8-8	8-8, G-10	0.2 - 0.45	11.0	58.9	1.45
	8-8, G-80	0.3	9.8	43.9	1.95
	8-8, Z	0.3	11.0	4.6	14.5
Taft, 8-6	8.6, G-69	0.25	11.0	28.6	3.0
	8-6, G-21	0.3	11.0	28.1	3.0
	8-6, Z	0.3	10.0	16.6	4.0

The following was determined at each step of integration (each step was taken at 0.001 sec) for each mass in direction of the main axes OX and OY: reaction; acceleration; absolute, relative and permanent displacements; accumulation of relative plastic deformations; absolute and relative energy within an elastic-plastic semicycle throughout the entire deformational range. Also the change in rigidity characteristics and periods of vibrations after each elastic-plastic semicycle are determined. In addition vertical acceleration and the values of vertical component of seismic vibrations, angular mass acceleration and change in periods of free vibration of torsional vibrations during an earthquake were determined. These parameters were printed out at the time intervals corresponding to specific states of the system. They provide information trace the changes in the parameters throughout the total loading history. At the end of the analytical process the maximum values of limit state parameters in the direction of the main axes of translational vibrations, horizontal and vertical mass acceleration, the periods of fundamental vibration along the main axes T_x^k , T_y^k , T_z as well as torsional vibrations T_Θ^k were determined.

Under the action of multi-component seismic forces it is rather difficult to interpret the results of the interdependency of the three directional forces. There are available very few experimental results by using three directional shaking tables.

At the first stage of the analysis of three-dimensional elastic-plastic systems it is reasonable to evaluate the peculiarities of their behavior as compared with two-dimensional models. Towards this end it is sufficient to carry out comparative analysis of both design models having equal characteristics by individual parameters when analyzing for equal forces. Then, system characteristics can be changed to examine the difference of the results.

The effect of eccentricity between the centers of rigidity and mass as well as vertical component of seismic vibrations on the parameters of system motion was examined in this study. In addition dependence of limit state parameters of inelastic systems on periods of free vibrations of vertical and torsional vibrations was also performed.

Taking into account the complexity of the problem and variety of factors effecting the behavior of elastic-plastic three-dimensional systems under multi-component ground motions, nonlinear one-mass systems were analyzed. Rather wide range of free vibration periods was selected: translational - $T_x = T_y = 0.5$ - 2.5 sec with step of 0.5 sec; vertical - T_z = 0.10 - 2.5 sec, $C_z = C_x(1 - 100)$; torsional - $T_\Theta = 0.03$ -3.0 sec, $C_z = C_\Theta(10 - 80)$. Each three-dimensional system ($T_{tran} = 0.5$; 1.0; 1.5; 2.0 and 2.5 sec) was analyzed by 18 variants for each design earthquake, this a total of 630 variants were carried out. The results of analysis of the system $T_x = T_y = 0.5$ sec at El Centro earthquake are given in Tables 2 and 3.

The variants which allow the evaluation of the effect of vertical component and eccentricities between mass and rigidity centers on elastic-plastic system behavior at two-, three-component seismic motions are given in Table 2 and those of vertical and torsional rigidity (free vibration periods) is given in Table 3.

Table 2. The numerical results of elastic-plastic systems under multicomponent seismic actions. $T_x^b = T_y^b = 0.5$ sec; $T_z = 0.2$ sec; $T_\Theta^b = 0.1$ sec. El-Centro earthquake.

No. of variant	T_x^e sec	T_Θ^e sec	e_x m	e_y m	N_{max} t	N_{min} t	$\bar{f}_x$	$\bar{f}_y$	$f_{x,nn}$	$f_{y,nn}$	$f_{x,e}$	$f_{y,e}$	K_W^x	K_W^y
1	1.13	-	-	-	-	-	7.9	7.6	3.02	2.8	1.0	2.5	20.	18.7
2	1.13	0.23	0	0	17.3	14.1	1.75	7.8	2.80	3.1	0.8	2.6	20.	17.9
3	1.12	0.23	0	0	34.3	28.0	7.62	7.9	2.72	3.1	1.1	2.6	20.	17.2
4	1.10	0.23	0	0	68.6	56.0	7.2	7.4	8.6	2.9	0.4	2.5	20.	16.3
5	1.15	0.23	0.5	0.2	0	0	8.96	7.2	3.72	2.3	1.8	1.9	19.	16.3
6	1.17	0.23	1.0	0.5	0	0	2.89	7.1	4.26	2.0	2.9	0.9	17.	12.1
7	1.17	0.23	2.0	1.0	0	0	8.89	11.	4.8	3.5	2.1	0.1	40.	49.2
8	-	-	4.0	1.0	0	0	-	-	-	-	-	-	-	15.7
9	1.15	0.23	0.5	0.2	34.3	28.0	8.6	7.3	3.4	2.6	1.5	2.1	17.	11.9
10	1.18	0.23	1.0	0.5	34.3	28.0	10.2	7.3	4.8	2.1	3.9	0.9	34.	38.3
11	1.22	0.24	2.0	1.0	34.3	28.0	8.44	11.	3.19	3.5	1.2	0.4	92.	121.
12	1.18	0.24	4.0	-	34.3	28.0	16.4	61.	7.5	26.	4.3	-	17.	92.7

Table 3. The analytical results of elastic-plastic systems under multicomponent seismic actions. $T_x^b = T_y^b = 0.5$ sec; $T_z = 0.2$ sec; $T_\Theta^b = 0.1$ sec. El-Centro earthquake.

No. of variant	T_y^e sec	T_Θ^e sec	T_Θ^{en}	N_{max} t	N_{min} t	$\bar{f}_x$	$\bar{f}_y$	$f_{x,nn}$	$f_{y,m}$	$f_{x,e}$	$f_{y,e}$	K_W^x	K_W^y
11	1.21	0.10	0.24	34.3	28.0	8.44	11.0	3.19	3.29	1.22	0.44	34.1	38.3
13	1.01	0.08	0.17	34.3	28.0	10.5	6.24	4.56	2.49	4.14	0.26	22.2	12.5
14	1.10	0.06	0.13	34.3	28.0	10.0	7.76	4.60	2.50	3.60	1.50	16.7	12.5
15	1.12	0.04	0.09	34.3	28.0	9.80	7.30	4.50	2.00	3.20	1.10	18.1	11.4
16	1.11	0.03	0.06	34.3	28.0	8.64	7.30	3.35	2.58	1.43	2.11	18.5	15.5
17	1.16	0.10	0.24	44.3	4.1	8.70	12.7	4.00	3.70	0.52	1.41	41.8	34.6
18	1.13	0.10	0.24	16.0	14.5	8.67	12.4	3.65	5.74	1.97	1.73	39.5	45.2

The variants of Table 2 and 3 are explained in the following.

-variant 1 - the results of analysis by two-dimensional design scheme (direction X - first horizontal component of the earthquake; direction Y - the second component);

- variants 2 - 4 together with 1 serve for analysis of vertical component effect of inelastic system motions without eccentricity $e_x = e_y = 0$. Vertical component varies from 0 (variant 1) to intensity which is twice as much as the design one (variant 4);

- variants 5 - 8 together with 1 are used for studying the effect of eccentricity value between mass and rigidity centers on three-dimensional system behavior without vertical component (two-component forces). Eccentricities change from 0 (variant 1) to $e_x = 4.0$ and $e_y = 2.0$ m (variant 8);

- variants 9 - 12 together with 3 are used for examining the effect of eccentricity of inelastic vibrations of three-dimensional systems subjected to three-component action (vertical component is equal to a design one). Eccentricities vary from 0 (variant 3) to $e_x =$ 4.0 and $e_y = 2.0$ m (variant 12);

- variants 13 - 16 together with 11 were used to analyze the effect of natural frequencies of torsional vibrations on inelastic vibration parameters of three-dimensional systems with vertical component of seismic vibrations and eccentricities between the centers of mass and rigidity;

- variants 17 - 18 together with 11 are used for studying the effect of vertical frequencies on the motion of elastic-plastic three-dimensional systems.

DISCUSSION

Fig. 1 presents the diagrams of inelastic deformation of the system with $T_x = 1.0$ sec of the Ferndale earthquake (7-25). For comparison the diagrams of deformation at one-component ground motion are also presented. The following observations can be made:

The change in intensity of vertical component of seismic vibration within a wide range (for variant 4 it is 1.6 times as much that of horizontal components) does not have noticeable effect on elastic-plastic system behavior under the action of the design earthquake: in most cases deformation and energy parameters of limit state are of the same order that exists when vertical vibration is ignored. But in some cases, for example for the system $T_y^b = 1.5$ sec when analyzing by using the El Centro (8-3) earthquake, energy potential is increased by 1.2 -

1.6 times, absolute deformation - by 1.3 times and remain deformation - by 1.5 - 2.0 times. However in general this difference does not exceed 5 - 10 %. The eccentricities between mass and rigidity centers especially for rigid systems (T^b = 0.5) has a substantial effect and can lead to a sharp growth of deformation and reduce of load-carrying capacity. For instance, for the system T^b = 0.5 sec, e_x = 2.0 and e_y = 1.0 using El Centro earthquake the values of deformation parameters increased 1.5 times while energy parameters - more than twice. Approximately the same situation is observed for the same variant was analyzed using Vernon (8-1), Taft (8-6) and San Fernando earthquakes. With further increase in eccentricities this difference sharply grows and the systems looses its ability to resist seismic loads. The same is also true when both eccentricity and vertical component of seismic vibrations (three-component action) are considered.

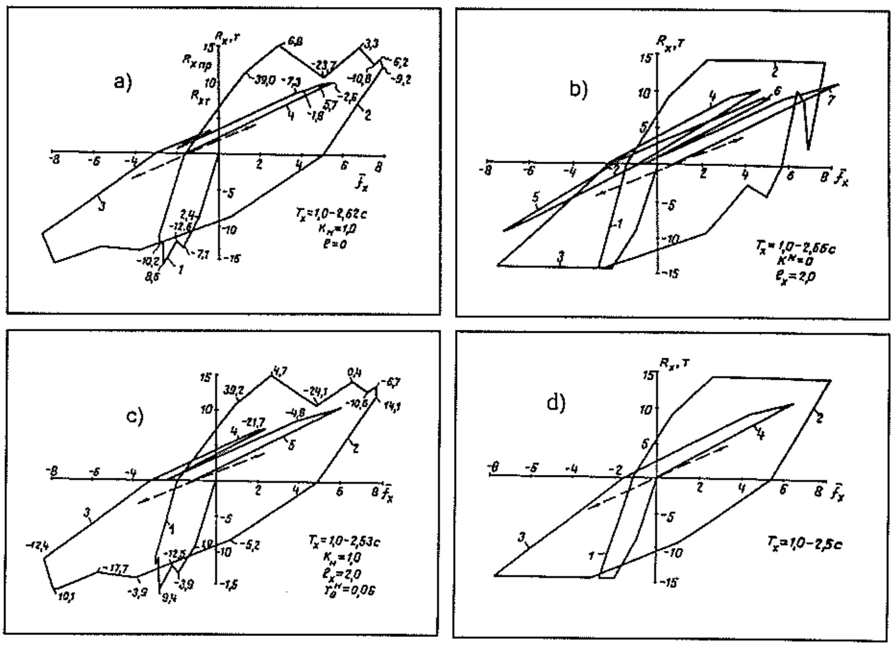

Fig.1. Inelastic diagram of deformation on one-, two- and three-component ground vibrations. Ferndal earthquake.

It should be noted that mismatch of mass and rigidity centers generates more cross effects for rigid systems. For instance, the results are characteristics of the systems T^b = 0.5, 1.0 and 1.5 sec, however for T^b = 2.0 and especially T^b = 2.5 sec the limit state parameters under three-dimensional design model at two and three component action do not differ from the results of variant 1. Thus, analysis for the inelastic behavior of real buildings under the action of strong earthquakes should be carried out with allowance for actual distribution of structure rigidity and unmatched mass and geometric centers.

It is established that in many cases torsional rigidity effects inelastic system behavior under intensive seismic motions. At this limit state parameters and first of all energy characteristics depend on periods of torsional vibrations, this dependence being more distinct for more rigid systems (T^b = 0.5 and 1.0 sec).

For instance, for the system T^b = 0.5 sec at El Centro earthquake the change in T^b from 0.1 to 0.04 leads to 2.5 fold decrease in K_W while at Vernon earthquake - 1.5 times. This indicated the necessity of using as mush as possible real properties of a building including torsional rigidity when analyzing them at elastic-plastic stage of motion.

To evaluate the effect of vertical vibration frequency on behavior of nonlinear systems variants 11, 17 and 18 for which rigidity in vertical direction changed by the factor of 100. Thus motion parameters change with vertical vibrations. However unambiguous relationship between the period T_z and limit state characteristics could not be established. In some cases rigidity reduction in vertical direction leads to increase in ultimate deformation, but energy parameters on the other ones to decreases. On the whole, especially for the systems with the periods of free translational vibrations ($T_x; T_y \geq 1.0$ sec) the frequency of vertical vibrations does not markedly effect the process of deformation of elastic-plastic structural systems.

The effect of the factors (vertical component, eccentricities between mass and rigidity centers, characteristics of torsional and vertical vibrations) on vibrations of three-dimensional inelastic systems under multi-component seismic forces can be traced by the diagrams of elastic-plastic deformation (Fig. 1). The form of diagrams changes greatly at three-component seismic forces what can be explained by the effect of vertical component on strength characteristics R_e and R_p. The presence of eccentricities also changes the diagrams greatly.

The results also suggest that the developed procedure and the RUPS -3K program have great potentials and allow complex analysis of three-dimensional elastic-plastic systems subjected to real earthquakes induces by several components of seismic vibrations (accelerograms). The proposed method allows the study of many factors which effect the behavior of real buildings subjected to strong earthquakes.

CONCLUSION

Performance of most buildings under of strong seismic ground motions is governed by damage progression of structural members and connection details. Thus, to obtain the actual behavior of structures during strong earthquakes should be carried out by considering the damage accumulation. In this paper, me describe a non-linear non-stationary damage accumulation model for the ultimate strength analysis of structures.

For frame buildings these models can be established by using the generalized diagrams of elastic-plastic deformation with variable characteristics. Non-linear properties of structures could be approximated by force-displacement relations (diagrams of deformation) with variable characteristics. The degree variation degree of diagrams depends on the loading history and structure damage accumulation.

The damage accumulation model provides:

- a description of the peculiarities of structural behavior under action of real earthquake;
- the analysis of damage accumulation processes under non-stationary loading. This is important for analysis of structures under random dynamic, including seismic, motions;
- an estimation of the damage of structures at any time of seismic vibration and after earthquake;

- reflect the physical processes of structural behavior in the conditions of strong earthquake, such as the cross effects.

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