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TAKING INTO ACCOUNT THE STRUCTURE SELF-VARIABLE STIFFNESS FOR ESTIMATION OF EXISTING BUILDINGS' SEISMIC RESISTANCE

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SUMMARY

One of the typical structural system of dwelling-houses is the frame building with masonary walls. Sometimes,the calculation of such existing buildings on the basis of modern seismic codes showes that their seismic resistance does not provide under earthquakes adequate to given region. One of reasons for that is the insufficient consideration of self-variable stiffness of the building under the seismic exsitation. The experimental and theoretical investigations show that under strong earthquakes the RC structural system independently changes his stiffness for adaptation to the given earthquake. In this case, the seismic forces decrease significantly (possibly two times), and the building can be take into consideration as structure with sufficient seismic resistance. We taked into account this fact for getting the common estimation of the existing buildings. The estimation method includes also the soil characteristics of the region, architectural and structural peculiarites, quality of the construction and materials, and common stage of the building. If we get the negative resulte, i.e. the seismic resistance of the building does not provide, it is necessary to use by additional braces with passive or active controlled stiffness. In this paper, the estimation model is given for calculation of the seismic resistance of the existing building in Israel with considerating its self-variable stiffness during earthquake excitation.

INTRODUCTION

One of the problems in developing the earthquake resistance theory is estimation of existing buildings' state in seismic regions. Existing approaches are mainly based on the building's earthquake resistance qualitative estimation. For more accurate estimation, a quantitative evaluation should be done. It can be done in a similar way to that described by Sekhniashvili [1], Danielashvili [2,3] etc.. According to the suggested method, the seismic resistance of an existing building is estimated

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.Taking into account the real non-linear stress-strain stage of the structure during an earthquake, allows a reevaluation of the seismic resistance. Iskhakov [4] has shown, that RC fully braced frame changes its stiffness and adapts its properties in order to provide an optimal seismic response. The frame regulates its behavior, attenuating the seismic response through autonomous disengagement of its concrete braces in tension. The advantage of concrete physical non-linearity in compression is also taken into account. The system has several levels of seismic regulation and a suitable one is selected for optimal response to agiven earthquake. The above factors significantly reduce the seismic forces and dynamic displacements, and create an optimal scheme of the structure Iskhakov[4].
The bracing system adopts the optimal state of the RC structure. As a result energy dissipation is increased and the seismic forces are reduced accordingly. It yields a higher seismic resistance estimation of the structure. If, however, the estimated seismic resistance is still not enough for a given seismic region, then an artificial variable stiffness system Kobory [5] or other energy dissipation systems based on active or semi-active control Ribakov [6] should be used.

EVALUATION METHOD FOR A BUILDING'S EARTHQUAKE RESISTANCE

The estimated earthquake resistance level

$$E_{er} = R_{er} - S_{er}. \tag{1}$$

where R_{er} is the required level, and S_{er} is the estimated earthquake resistance shortage.

Another concept of the proposed method is the relative earthquake resistance, K_{re}. It is assumed to be an expert estimation of the deviation between a design solution for the existing building condition and that, required by the seismic design codes. According to Eq. (1), it is required to calculate the value of S_{er}, since the value R_{er} was previously set, according to the relevant seismic design code. The value of S_{er} is determined according to the value of the Relative Earthquake Resistance Coefficient K_{re}.
The functional dependence between S_{er} and K_{re} is described in further details below. The Relative Earthquake Resistance of a building and the corresponding coefficient K_{re} are expressed as a fraction of a unit, according to the required earthquake resistance level: according to the characteristics and concepts of the seismic design codes and recommendations, that are valid during the expert estimation; as regards certain main factors (about 25), that are identified as important to determine the earthquake resistance of a building; setting the "weight" of each chosen factor, based on the estimated conformity of tested buildings to the valid seismic design codes; setting the degree of wear and damage of a certain weight factor and their influence on the earthquake resistance of the said factor.
The final influence of each factor is determined based on the quantitative estimation of the earthquake resistance of each factor that equals the product of the three stated parameters. A set of factors, which are used for calculating the Relative Earthquake Resistance Coefficient K_{re}, is determined by the main formalized concepts of the seismic design regulations, recommendations and codes. These factors are included in special questionnaires (an example for a completed questionnaire on a concrete building in Jerusalem is presented below). Three parameters are used to describe each "*i*" factor. The first, q_i, estimates the "weight" of this factor to form the overall earthquake resistance of a building. It is expressed in fractions of a unit. The value "1" means that the factor has a vital importance for the earthquake resistance of the building. Conversely, the value "0" means that this factor is not important. The second parameter, s_i, estimates the degree of deviation of the factor "*i*" from the requirements according to the main concepts of the seismic design recommendation as set by the codes and regulations. The value "1" for the second parameter means that all the design requirements and recommendations are completely met. The value "0" means that the condition of the factor "*i*" is totally wrong. The value of the second parameter is also expressed in a unit fraction.
For existing buildings and engineering structures, the wear and damage coefficient d_i is also considered for estimating the factor "*i*". Of course, the coefficient d_i is estimated only for wear- and damage-related coefficients. For other factors, we assume that d_i equals "1" for simplicity purposes and according to the formalized approach. The numerical values for all factors are provided according

to the expert estimation. The product of coefficients q_i, s_i and d_i leads to the relative coefficient of decreasing the earthquake resistance by this factor "*i*".
The total of the relative earthquake resistance coefficients for the entire building, according Danieli [3], is estimated as follows :

$$K_{re} = \sum q_i s_i d_i / \sum q_i \,. \quad (2)$$

The factors taken into consideration may be conventionally divided into two groups. The first group includes general factors for various structural systems. It outlines the most general concepts of the seismic design codes and provisions. The second group consists of various factors for various types of structural systems (frames, precast concrete large panels, load-carrying concrete and masonry walls, etc.). Correspondingly, two questionnaires are filled out for each type of building.
The first estimates the influence of the general suggestion for the earthquake resistant construction on the building's earthquake resistance and the second estimates the influence of the legal requirements on the building's earthquake resistance of the tested type. For existing and partly damaged buildings, the wear and damage coefficients may be estimated according to special data. Numerical values of the estimated shortage of earthquake resistance Ser are approximated according to the following formulas:

$$S_{er} = 0.008(1-K_{re})+0.921(1-K_{re})^2-1.686(1-K_{re})^3+1.223(1-K_{re})^4 \quad (3)$$

or

$$S_{er} = 0.1443 \ \ln\frac{1}{K_{re}} \ ; \ K_{re} \leq 0.75 \quad (3a)$$

This dependence corresponds to the increase in the value of the shortage of earthquake resistance by 0.1, as the relative earthquake resistance factor decreases twice this rate. Eq. (3) and (3.a), should be used when $K_{re} > 0.125$. These values of S_{er} cover the range of the coefficient's horizontal ground accelerations $0.075 \leq Z \leq 0.3$.
The functional expression is presented in Fig. 1. Based on the diagram of Fig. 1 it can be assumed that the slope of the graph for the value of K_{re} that tends to $K_{re} = 1$, is smaller than the left zone of the graph. It may be used to explain the following point: in case that $K_{re} = 0.9$, small differences in the expert estimations will have smaller influence on the value of S_{er} than in the case of smaller values of K_{re}. During the determination of factors S_{er} and E_{er} there is a certain lack and uncertainty of input data and the earthquake resistance estimation is characterized in a conventional way. Therefore, the estimated value of E_{er} requires a correction while considering the exact condition and properties of a tested building.
The value of E_{er} is to be analyzed in relation to two adjacent values of Z (Z_0, Z_0') according to Israel Standards (IS 413-1998), in order to meet the following condition: $Z_0' \leq E_{er} \leq Z_0$. Then, the value of E_{er} must be approximated as equal to Z_0 or Z_0' (towards the nearest value). If $0.06 \leq E_{er} < 0.075$, it is assumed that $E_{er} = 0.075$. According to the mentioned above, for $Z > 0.1$, it is required that the earthquake resistance level will be for $K_{re} = 0.810$; $S_{er} = 0.025$ and for $Z \leq 0.1$ for $K_{re} = =0.875$; $S_{er} = 0.0125$.

DETERMINING THE EARTHQUAKE RESISTANCE LEVEL FOR AN EXISTING RESIDENTIAL BUILDING IN JERUSALEM

The proposed method is applied to a real residential building in Jerusalem (Z = 0.1) and correspondingly $R_{er} = 0.1$. Figures 2, 3 and 4 present typical drawings of a floor, elevation and overall view, respectively, of the tested building. The Relative Earthquake Resistance Coefficient Kre was estimated based on the design data and the results of the inspection at the building site. Then, two special questionnaires were filled out (see Table for details). The Relative Earthquake Resistance Coefficient Kre was finally estimated, according to the data shown in the Table 1.. The value of the related seismic safety factor of the building was calculated according to Eq. (2) :

$$K_{re} = \frac{\sum_{(1)} q_i s_i d_i + \sum_{(2)} q_i s_i d_i}{\sum_{(1)} q_i + \sum_{(2)} q_i} = \frac{11.86 + 4.42}{12.9 + 4.90} = \frac{16.28}{17.80} = 0.914$$

The index in the round brackets (near the sum sign) corresponds to the table number as shown above. Correspondingly, the value of the estimated shortage of earthquake resistance Ser, as approximated according to Eq. (3) and diagram (Fig. 1), is $S_{er} = 0.0064$. The estimated earthquake resistance level of

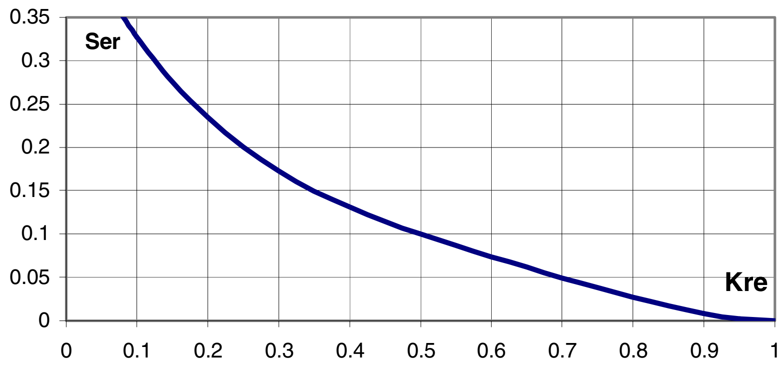


Fig. 1. The K_{re} - S_{er} relation.

a building E_{er}, according to its definition and Eq. (1) equals the difference between the values of R_{er} and S_{er}:

$$E_{er} = R_{er} - S_{er} = 0.10 - 0.0064 = 0.0935.$$

The calculated value of the estimated earthquake resistance level Eer is to be compared with two adjacent values of the ground's design horizontal acceleration Z. The values of Z (for $E_{er} = 0.0935$) are $Z_0 = 0.10$ and $Z'_0 = 0.075$. Therefore, according to the value of the estimated earthquake resistance level $E_{er} = 0.0935 > \frac{0.075 + 0.10}{2} = 0.0875$. The final value of the estimated earthquake resistance level for the tested residential building will be $E_{er} = 0.1$. This value corresponds to the ground's design horizontal acceleration for the region Z = 0.1. The obtained values of the shortage of earthquake resistance Ser and the estimated earthquake resistance level E_{er} are the expert quantitative characteristics of the earthquake resistance for the tested building. According to the described method, the tested building meets the required earthquake resistance level $E_{er} = Z$. Sometimes, there is a significant estimated shortage of earthquake resistance or that the estimated earthquake resistance level is less than required. In such cases, the multi-factor estimation approach (see Table 1) as described above, may be used for taking a decision on the improvement of the design solutions. In addition, using this method of quantitative expert estimation for the earthquake resistance of a building may be helpful in order to take a correct design decision and to choose optimal structural schemes of a building for a new design. Moreover, the method may be used for testing existing partly damaged buildings and for designing their strengthening.

If the estimation shows that the building does not correspond to the seismic resistance required for a certain seismic zone, non-linear stress-strain behavior of structural elements and contribution of variable stiffness should be taken into account. For example a multistory braced frame is studied.

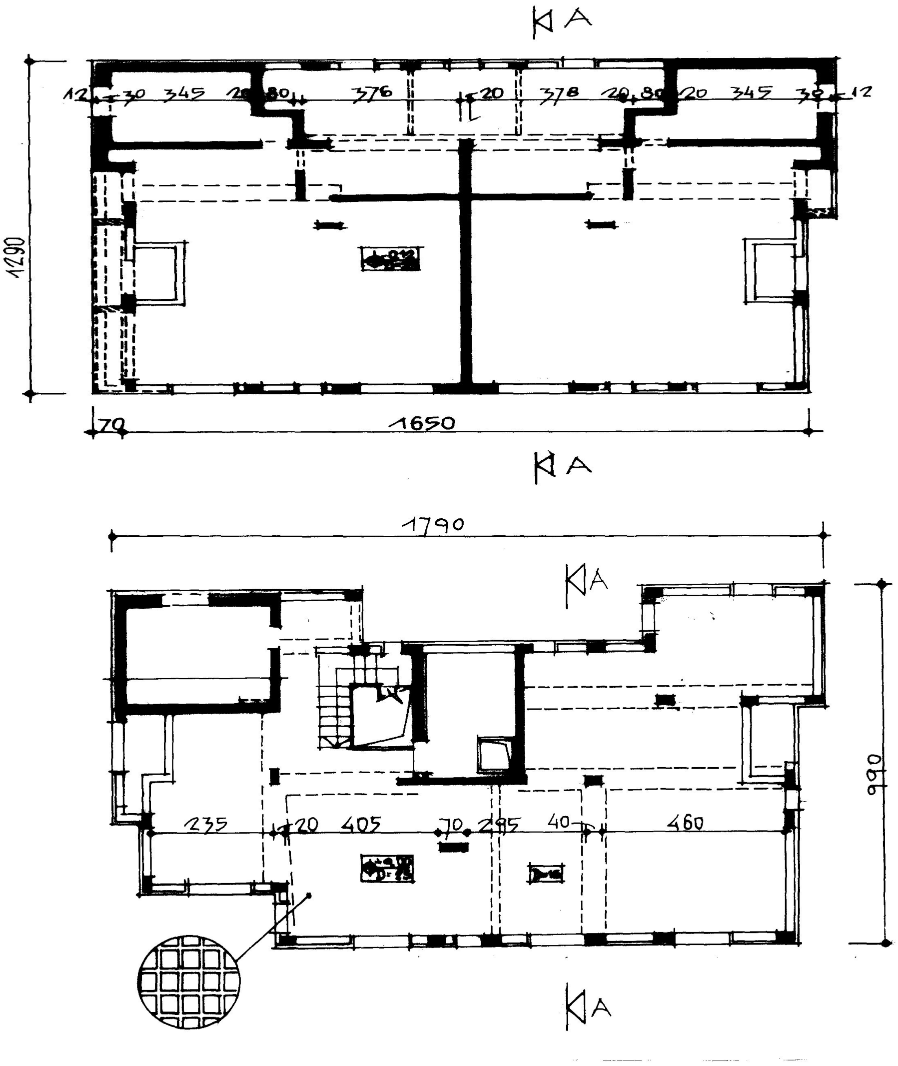


Fig. 2. Schematic structural typical designs of a floor (unit: cm).
1. level "-0.12" ; 2. level "+9.00" (unit: m).

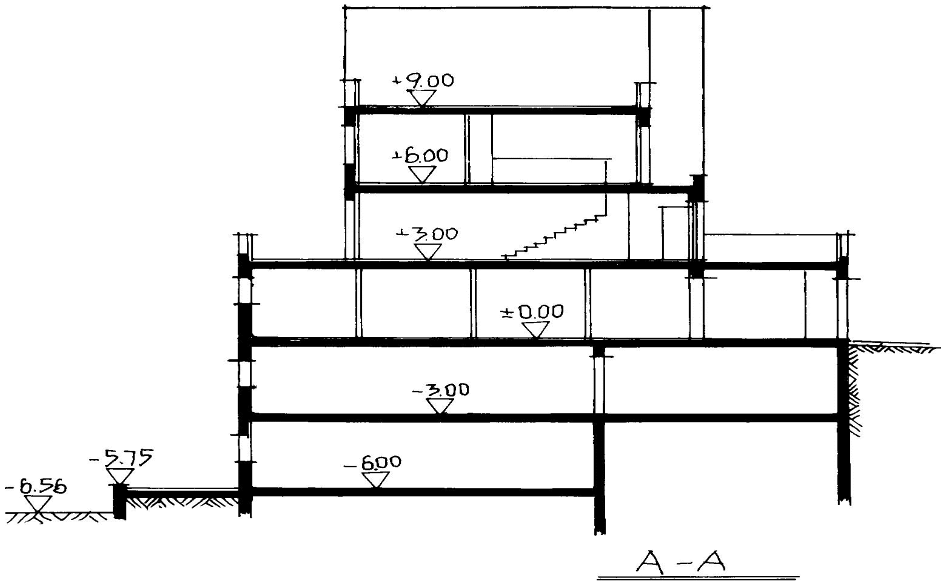


Fig. 3. A schematic elevation view "A – A" of a building in the transverse direction (unit: m)

Fig. 4. The building's overall view

Table 1. A multi-factor estimation of the relative earthquake resistance for Calculating K_{re} for a test of a residential building in Jerusalem.

##	The name of the factor that affects the earthquake resistance	The conformity of a factor to the regulations and recommendations of the codes and general principles of seismic construction	The factor's Importance q_i	The value of factor s_i	Wear and damage coefficient d_i	$q_i x s_i$ x x d_i
1	2	3	4	5	6	7
		1. General factors for various structural systems				
1	Soil conditions	Unfavorable conditions for a seismic building that is located on a sloped area. The slope angle is more than 20 degrees. Soil type: rock.	1.0	0.8	1.0	0.8
2	The importance factor of a building	Residential building (number of flats – 8).	1.0	1.0	1.0	1.0
3	Structural characteristics (regularity; symmetry; uniform distribution of shear walls and masses; general dimensions)	Non-regular building, non-symmetrical design (partial symmetrical for the first 2 floors only – in the traverse direction). Number of floors – differs for different zones in the design (from 2 to 6), non-uniform distribution of masses and rigidities. Dimensions in a design – 12,9x18,0 m, height of a building – 9,0-18,0 m.	1.0	0.7	1.0	0.7
4	Structural scheme of a building	For vertical loads – a system of multi-span flat beams and columns, shear walls. For horizontal loads – shear walls.	1.0	0.8	1.0	0.8
5	Integrity and homogenous properties of structures	All load-carrying structures (beams, columns, shear walls) are designed as monolith structures, the same concrete class (design strength 30 MPa) is assumed for all structures.	1.0	1.0	1.0	1.0
6	Structural expansion joints due to seismic conditions	Expansion joints are absent	0.8	1.0	1.0	0.8

1	2	3	4	5	6	7
7	The backgrounds for a structural design	A series of structural analysis was done (modal analysis) according to (IS 413-1998): the coefficient of the ground's predicted horizontal acceleration Z=0.1; the torsion modes of vibrations were also taken into consideration. The earthquake resistance of a building by analysis was assumed as provided, after adding some new shear walls and increasing the column reinforcement (final stage of analysis).	1.0	1.0	1.0.	10
8	Zones of staircases	Staircases are located non-symmetrically in building's drawing. The staircases are separated from other structures.	0.9	0.8	1.0	0.72
9	Floor slabs	Concrete slabs (solid and bi-directional ribbed floor slabs).	0.8	1.0	1.0	0.8
10	Partitions	The material of partitions: light concrete hollow blocks united by sand-cement mortar. The partitions are not connected with slabs and columns by steel links or bars.	0.6	0.9	1.0	0.54
11	Protruding elements (enclosure elements, balconies)	Reinforced concrete framework and ribbed slabs filled with lightweight concrete blocks. Balcony slabs – solid reinforced concrete.	0.8	1.0	1.0	0.8
12	Foundations	Pile foundations. Continuous reinforced concrete beams in both directions connecting the piles.	1.0	0.9	1.0	0.9
13	Additional elements – retaining walls	Cantilever type reinforced concrete retaining walls are used to decrease the influence of a slope at a building site on the earthquake resistance of the building.	1.0	1.0	1.0	1.0
14	The quality of construction and materials	The construction quality is good. The real strength of concrete was estimated by a dedicated standard hammer – about 40 MPa, to compare with the design strength 30 MPa.	1.0	1.0	1.0	1.0
		Total	$\sum_{(1)} 12.$[illegible]			$\sum_{(1)} 11.$[illegible]

2. **Typical factors for reinforced concrete buildings**						
1	2	3	4	5	6	7
1	The connections between the bearing elements of a system (between columns and beams, between beams and shear walls, columns and walls)	Special strengthening of joints by additional meshes, spiral type links and sloped reinforcement bars was not implemented.	1.0	0.8	1.0	0.8
2	Strengthening of column and beam zones in the joint regions	Decreased stirrups spacing in columns and beams was used in the joint regions.	0.8	1.0	1.0	0.8
3	The presence of rigid walls (shear walls, diaphragms, coupled shear walls)	Separate and coupled shear walls are present. Shear walls are located non-symmetrically and non-uniformly by the drawing and by the building's height.	1.0	0.8	1.0	0.8
4	Filling of external walls; connecting the walls to structural elements	Filling of light hollow concrete blocks between reinforced concrete elements was used. Reinforced concrete border beams and joists, according to (IS 413- 1998) were used. The connections between the fillings and load-carrying elements were provided by longitudinal concrete inserts in the filling. The rigidity of fillings was not taken into consideration in the structural analysis.	0.8	0.9	1.0	0.72
5	The connections of stone cladding to a masonry wall	Stone cladding was connected to the masonry walls with steel connection bars and cement mortar.	0.6	1.0	1.0	0.6
6	The resistance of structural elements to plastic deformations	The necessary conditions for the development of plastic deformations are present. Cross-sections of the reinforced concrete elements are properly designed, the amount of steel for a cross-sectional reinforcement is not too large.	0.7	1.0	1.0	0.7
		Total	$\sum_{(2)} 4.9$			$\sum_{(2)} 4.4$

DESIGN SCHEME

The structure is a monolithic RC six-story two-bays frame with flat-slab floors Iskhakov [4]. Its dimensions are 12×12 m, the spacing of the column is 6 m in either directions, story height is 3 m. Each story has diagonal braces in both bays. The cross section dimensions of the elements are as follows: columns - 0.4×0.4 m; braces - 0.2×0.4 m; floors - 6.0×0.16 m. The braces are reinforced in their middle part against the bending moment due to the dead load and include the constructive reinforcement only in their main part up to 0.5 m from the joints. The constructive reinforcement is able to get the tensile force in the crack.
The acting forces consist of the ead load and static live load, plus the horizontal seismic forces concentrated at the floor levels. It was assumed that the dead and the live loads per unit floor are 0.52 and 0.26 t/m^2 respectively. A total frame load is 3.12 and 1.56 t/m. It has been shown Iskhakov [4] that its vibration period is 0.638s. The horizontal seismic forces in the frame at the floor levels are as follows: 2.19t (first story); 4.38 t; 6.57 t; 8.75 t; 10.94 t; 13.13 t (sixth story).
The braces are structural elements of the frame, designed against axial tension and compression forces and arranged symmetrically in the two spans under control direct. Their bearing capacity is 96 ton in compression and 7.2 ton in tension. Under tensile forces a brace cracks, and in the absence of reinforcement would yield unilateral disengagement. However, in practice failure does not occur, because the stress rapidly alliterates in sign and the brace is constructively reinforced. Upon reversal of the vibration sign, the cracks close and the brace is re-engaged in compression.
When a new vibration cycle begins, the brace does not more withstand tension and works unilaterally in compression only (at the modulus value of the preceding cycle). The latter decreases from cycle to cycle, but so long as the compressive force exceed 96t, the brace adjusts to the given earthquake and retains the final modules value. Up to this stage the energy dissipation occurs. If, however, the above force level is exceeded, the brace disengages irreversibly and the vibration period of the structure increases. The braces thus have two disengagement levels - in tension and in compression, representing distinct (jump-type) levels of dissipation of the system energy, as well as numerous supplementary levels associated with the changing values of the stress-strain modulus.

THE SELF VARIABLE STIFFNESS (SVS) MECHANISM

Unilateral (in tension only) or complete (in compression as well) disengagement of the braces yields substantial reduction of the system stiffness. On the one hand it weakens the seismic forces, and on the other - lengthens the vibration period, thereby maintain the structure out of resonance. A particular role is played in this process by the vertical static loading. When a brace is disengaged in tension and asymmetry is created, the structure acquired a horizontal components in its deflections, opposite to its displacements under the seismic forces. Since, however, the latter are themselves function of the structure mass and the live load, increase of the seismic forces makes for a corresponding increase in the counter-effect of the static loading. All the above factors unilateral or complete disengagement, reduction of the stress-strain modulus, the counter-effect just mentioned - substantial reduction (over 50%) of the seismic forces and dynamic displacements, and create an optimal scheme for the structure with respect to the earthquake in question. In view of the individual character of the scheme, however, it cannot be prescribed in advance.
A total of seven schemes were analyzed, numbered from 1 (full bracing) through 7 (unbraced frame) (Fig. 5), while scheme 4 represents a frame with unilateral disengagement. For each scheme, the following data were sought: the periods of the first three modes of vibration; the brace forces; the horizontal displacements of the system (total and separate for each loading); the story drifts; the normal forces in the columns and the bending moments in the floors. Also analyzed was the effect of a "Loma Prieta" type earthquake over a 10s interval, with maximum acceleration amplitude 1.874m/s^2 and 2.384m/s^2 in the x and y directions respectively - in terms of the base shears and brace normal

forces. The analysis was carried out for two cases of modulus (constant and variable) and the two load combinations ($F_g + 0.2\ F_q + F_D$ and F_D; F_g and F_q being the dead ("static 1") and live ("static 2") loads and F_D the seismic load).
Each of the seven schemes being a particular response, the adaptation process of the structure. In scheme 5 through 7 disengagement in compression took place, indicating that the forces in these braces reached the 96 ton level.

DYNAMIC ANALYSIS OF THE SVS SYSTEM

The structural response to real earthquakes was obtained using the ETABS [7] software. The vibration periods for all modes are seen to increase regularly with the serial number of the scheme, and so do the maximal horizontal displacement drift ratios, whereas the stress-strain modulus of the braces in compression decrease. When the brace force in the given scheme is zero, this means that in the preceding scheme it has exceeded the 7.2 ton limit, and brace is now unilaterally disengaged in tension but still engaged in compression. Fig. 6 shows the time histories of the base shears for the schemes 1,4 and 7 ($E_c \neq$ const).
The analysis shows Iskhakov[4], that scheme 4 (Fig. 5) is the threshold case, after which the shear deformations are largely stabilized and the bending ones (those of the lower stories) increase. Note that the optimal effect of the static loading is manifested for stories 2, 4 and 6 in the schemes with the same numbers, i.e. optimization is a process in itself. The mutual displacement of consecutive stories (i.e. the shear in the columns) increases steeply after scheme 4. The same applies to the bending deformations within each story, jading by the variation pattern of the drift ratios themselves. The horizontal displacements of the frame also increase steeply after scheme 4. In scheme 4 the counteractive effect of the static loading reaches maximum. The scheme also represents the threshold for the first vibration mode even though already before it the vibrations are almost relative to scheme 1. All the makes for substructure weakening of the seismic forces. As a consequence, the next brace may remain engaged, indicating that the structure has adapted to the given earthquake and its state is the corresponding scheme is optimal. This is the essence of the SVS system.

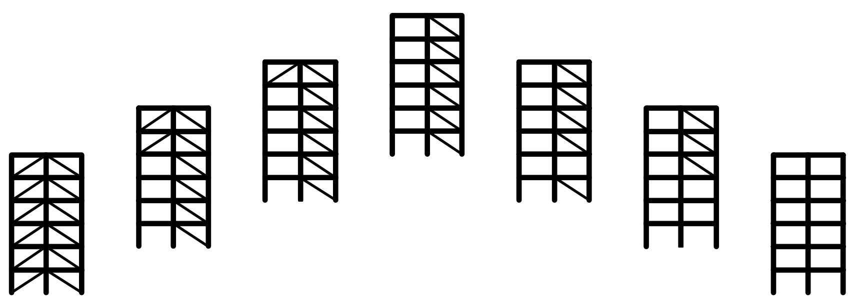
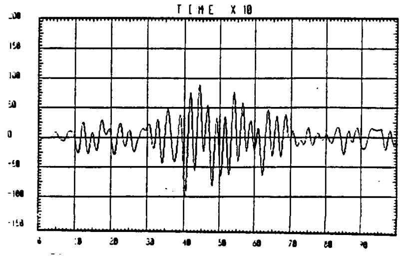
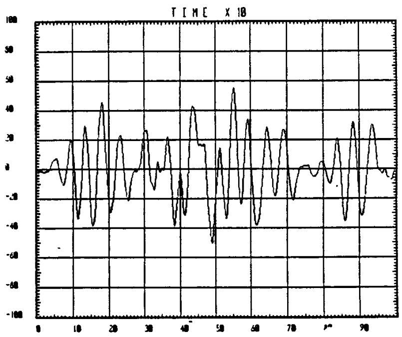


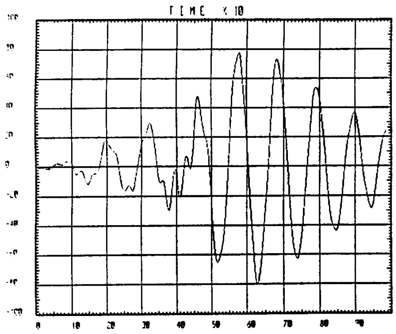
Fig. 5. Frame schemes (4 - optimal scheme).



(a)



(b)



(c)

Fig. 6. Base shears' time histories: (a) scheme 1; (b) scheme 4; (c) scheme 7

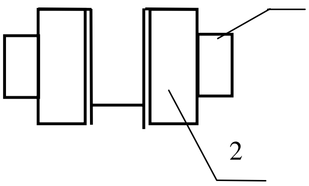


Fig. 7. The principal scheme of an active friction damper.
1-internal element; 2-external element; 3-pressure device

ACTIVE CONTROLLED STRUCTURE WITH VARIABLE STIFFNESS

Active variable stiffness systems (AVSS), described by Kobori[5], Ribakov[6], etc. are aimed to reduce the response of structures to earthquakes by active control of the structure's stiffness. Hence, these systems can be used in the cases, when the structures' own potential including nonlinear strength-strain relationship does not provide proper seismic resistance. The AVSS have the advantage that the control forces at every structural level can be changed within a wide range due to an active or semi-active devices implemented at each story. The control forces in the devices are actively controlled according to an optimal control algorithm. Ribakov[6] have analysed a friction damped seven story structure. The principal scheme of the damper is shown in Figure 7. It consists of an internal element (1) connected to the rigid floor diaphragm, two external elements (2) connected to an inverted V-shaped brace, and to a pressure device (3). The friction force produced in the contact surface between the internal and external elements depends on the pressure. By changing the pressure at every time step, the friction forces in the devices at each level can be regulated according to the requirements of the optimal solution. Ribakov[6] have demonstrated a significant improvement in structural response compared to those of a passive controlled and an uncontrolled structures discussed above (Iskhakov[4]). Under the earthquake histories that were examined the passive controlled structure had a peak displacement reduction of up to 50% compared to the uncontrolled one. For the active controlled structure a peak displacement reduction of up to 75% (compared to the uncontrolled structure) was achieved.

CONCLUSIONS

This paper describes the method of Multifactor Quantitative Estimation of the earthquake resistance of existing buildings. About 25 main factors, assumed to be significant, are taken into consideration. The "weight" parameter that stands for the importance degree is taken into account in the total seismic safety of each factor. The multi-factor estimation is used for a correct decision during the retrofitting in order to choose more optimal structural schemes of a building for a new design. An example for a quantitative estimation of a real residential building earthquake resistance is presented. For buildings, that according to the above method do not satisfy the seismic resistance requirements, a self variable stiffness system is proposed to be used. The basic properties of concrete regulating the structural seismic response and adopting its optimal state with maximum energy dissipation. This fenomena leads a reduction of the seismic forces about twice. The system has several modes of seismic adaptation (in terms of material, structure and loading) which it applies for adapting itself to the given earthquake. Active control significantly improves the behavior of buildings during earthquakes. A reduction of the seismic forces in buildings with such systems is about 75% compared to uncontrolled ones. The above described methods for protection of structures from earthquakes enable to reduce the seismic forces more than twice, adapting the structure to a region with higher seismic activity.

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