CRITERIA FOR EARTHQUAKE RESISTANCE CODES BASED ON ENERGY CONCEPT

DRAFT DESIGN CODE

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SYNOPSIS

The knowledge of the energy concept is necessary to understand the effect of intense earthquakes on structures. Our studies confirmed the possibility of transforming the energy concept in a concrete and active engineering instrument for the interpretation of the behaviour of structures during earthquakes, for the design of earthquake resistant structures and in consequence as basic concept of the codes for earthquake resistant design.

The energy response gives different possibilities for judgement of safety by energy criteria, for different kinds of earthquakes and different intensities. The authors prepared a Draft Design Code founded on energy philosophy.

FUNDAMENTALS

The energy concept was firstly formulated by G.W. Housner. He also showed the importance of this new concept for designing.(1, 2, 3, 4). Important contributions have been published by different authors during the past few years. (5, 6, 8, 9, 10, 11, 13, 17).

The physical reality is that during the earthquake the structures are in a seismic energy field. There is a continuous energy interaction process between the ground and the structure wuth energy transfer into the structure and an energy reflection process. The seismic vibration of a structure is considered to be a continuous process of energy feed. The energy response is also considered to be a continuous process. In the energy response the energy balance gives the time history for the whole earthquake duration of the variation of the kinetic energy ($\frac{d\,Ek}{dt}$), the potential energy ($\frac{d\,Ep}{dt}$), the consumption of energy by yielding ($\frac{d\,Ey}{dt}$), and by viscous damping ($\frac{d\,Ev}{dt}$), for each storey mass and for the structure as awlole.

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The energy response also gives the time history of Ek (kinetic energy in a given moment during the earthquake), Ep (potential energy stored in the structure in a given moment), Ev (viscous energy consumption from the beginning of the earthquake to the moment under consideration), Ey (energy consumption by yielding from the beginning of the earthquake to the moment under consideration), Ef (the energy feed in the structure from the beginning of the earthquake to the moment under consideration). So it is possible to examine how propagates the seismic energy waves from the base to the top of the structure, how the energy process develops along the height, and especially the energy losses. We can also examine the influences of the different types of earthquakes on the energy response, and the influence of the energy response when the intensity of the earthquakes increases until it becomes theoretical collapse.

Energy response basic relations, derived for systems acting in dynamic range, especially for the seismic action, are:

1. for the energy balance

$$\frac{d\,Ef}{dt} = \frac{d\,Ek}{dt} + \frac{d\,Ep}{dt} + \frac{d\,Ev}{dt} + \frac{d\,Ey}{dt}$$

2. for the energies

$$Ef = \frac{d\,Ef}{dt}; \quad Ek = \frac{d\,Ek}{dt}; \quad Ep = \frac{d\,Ep}{dt}; \quad Ev = \frac{d\,Ev}{dt}; \quad Ey = \frac{d\,Ey}{dt}$$

3. for the energy feed

$$Ef = Ek + Ep + Ev + Ey$$

For these relations it was necessary to derive the necessary mathematical expressions, for bilinear systems which were programmed at the ELECTRONIC COMPUTER CENTER of the Institute of Seismology, Earthquake Engineering and Town Planning at the University of Skopje.

The energy response was studied for several structures. The main conclusions are:

a. During stronger earthquake the values of potential energy are insignificant.

b. While structure works elastically the energy feed process is very strong.

c. After a duration of time with very strong energy feed process follows a time interval in which develops a very strong energy consumption process by yielding. This permits the developemnt of the dynamic energy adaptation of the structure.

d. The values of the kinetic energy and of the variation of the kin. energy are greater than the other energy forms.

e. The viscous dissipation is very active while the structure remains elastic. This contribution is very small during the period when the structure behaves plastically.

f. The kinetic energy gets maximum value before the structure enters the plastic range. After that this energy decreases strongly because, at the same time, it is very strong the process of the plastic absorbtion of the energy by yielding.

g. By analogy, the dissipation by yielding in the structure reacts as a break.

h. The energy feed curve shows fluctuation caused by the kinetic energy fluctuation.

i. The differences between the maximum values of the energy "Ef" and the maximum values of the energy absorbed (Ev + Ey) show the reflection and refraction process of the seismic waves which transport the seismic energy.

j. The energy feed and reflection process are aspects of the interaction process between the ground and the structure.

k. For multi-storey structures the energy balance of the total structure can be calculated as a sum of the energy balance of all masses. Comparing the energy diagrams of different storeys we get the phenomena of reflection and refraction of the seismic energy waves.

l. At storeys, where the plastification process is strongly developing appears a concentration of energy absorbtion by yielding which causes localised absorbtion. In addition, in these storeys develops a strong process of reflection of the seismic energy waves.

m. In the first time interval, after the structure enters the plastic range, the value $\frac{d\,Ey}{dt}$ is maximum and after that rapidly decreases.

n. When $\frac{d\,Ep}{dt}$ 0 it means that the system cannot continue with an uniform elastic deformation. For $\frac{d\,Ep}{dt} = 0$ we have maximum potential energy Ep.

o. When $\frac{d\,Ek}{dt}$ 0 the energy feed stops. For $\frac{d\,Ek}{dt} = 0$, Ek is maximum. The negative value of $\frac{d\,Ek}{dt}$ also shows that the energy is reflected and refracted or consummed.

p. When $\frac{d\,Ey}{dt}$ 0 the structure is at the end of the plastic phase, then the absorbtion by viscous plastification is very small. When $\frac{d\,Ey}{dt}$ 0 the accumulation of the plastic deformation stops.

q. When $\frac{d\,Ef}{dt}$ 0 means that the danger concerning the stability of the structure has passed, because the mechanism for the energy absorbtion is advantageous.

r. The variation of the energy balance is advantageous for structures with bilinear positive diagram.

The energy concept offers a basic criterion for the structural safety analysis. G.W. Housner recommends a "ultimate energy design" (1,2,3,4). J.A. Blume (6) brings in "the reserve energy technique". Muto's concept is "sufficient strength and high ductility" (9,10). A.M. Frendenthal expounded recently (14,15) that the probability theories and statistical methods are the most adequate in a view of the structural safety analysis and that it is possible to examine by energy method the failure and the unstability in the case of static load. The authors considered that all these concepts for earthquake design can constitute an entity.

The authors in the study "Energy Interpretation of the Structures Behaviour during the Earthquake of 26 July, 1963 in Skopje" showed the valuability of the energy concept at the real behaviour of the structure. (13). It is a very good agreement between the real behaviour and the theoretical analysis foundation energy concept.

This concept has been recently applied to the design of several buildings having 6 - 16 storeys (Photo 1,2). The authors took part in their designing. These buildings having R.C. structures of the shear type of frames have been separately designed in Yugoslavia for the reconstruction of the town Skopje and in Romania for towns in seismic areas. Their structural characteristics and mass distribution show interesting features : slabs with large cantilevers, flexible ground floor, tall structures with ununiform distribution of masses as well as structure with special columns.

The"natural laboratories" (Alaska, Caracas) showed that the existing Codes give unrealistic safety. The structures elaborated according to the existing Codes can have concentration of energy dissipation by yielding, a small capacity of energy dissipation and one erroneous construction.

All these ideas form one new philosophy for designing which can be taken as a base for preparation of a new Code. To improve actual designing in our countries in order to avoid errors in the structures designed according to the existing Codes, we have come to the conclusion that it is necessary and possible a transition period while the official code can be used at the same time with the new ideas. A Draft Code was prepared by the authors and applied to the earthquake resistant designing of different types of structures.

DRAFT CODE
FOR EARTHQUAKE RESISTANT DESIGN

1. BASIC CONCEPTS

1.1. This Code is founded on the energy concept. To resist against strong earthquakes the structures should be able to absorb large quantities of energy. The new philosophy of design requires simultaneous sufficient strenght and high dustility. The main rational aims are:

- to avoid structural damage for earthquake with intensity that corresponds to the standard seismic intensity of the site;
- to preveht structural significant damage by large accumulation of plastic strain, for stronger earthquake;
- to resist without collapse the strongest probable earthquake of the site.

1,2. A carreful attantion should be provided for the construction (or conformation) of structural elements and for the detailes of joints and conections.

1.3. The proper qualities of the materials are the ductility and the strength in postelastic stage under numerous cycles of alternative loading. For the decrease of the seismic response of the structure it is recommended to use lightwieght materials and structures. The principal criterion for proper selection of the type of the structure is the energy dissipation ability. Brittle materials and brittle structures should be avoided.

1.4. It is recommended to consider the results of the existing experiments for the postelastic deformability, ductility and energy dissipation properties of materials and structures.

1.5. The following methods are used:

a. methods according to the elastic theory for the stress analysis due to vertical loads. For the stresses analysis due to seismic horizontal load, elastic and plastic methods;

b. the probability and statistical methods in order to examine the structural safety problems, taking into account the variations of the parameters of the structure properties and of the earthquake characteristics;

c. for the strength design of elements the methods of official codes (alowable stress method or ultimate strength design);

d. for the seismic analysis, in addition to the equivalent static load of the official load, the methods of direct dynamic and energy analysis (with electronic computer);

e. the plastic theories in order to establish the postelastic properties for plastic hinges, for structural members and for each storey, namely the strength properties (yield and ultimate moment), plastic deformability, ductility ability, energy dissipation ability by yielding.

1.6. The designer should lead the energy dissipation process by definition of zones where plastic hinges may occur, by choice of special energy dissipation, structural elements and by achievements of successive lines of resistance.

1.7. The safety should be the result of seceral analysis and not a coefficient determined according to the code. To vouch for the safety of the structure it should be shown that it possesses necessary energy dissipation ability. For the structural strength the base shear coefficient computer conformity the method of official code it is considered as minimum value. The energy criteria gives the best information about the structural safety. For this purpose the theoretical probable mechanism of the collapse should be investigated. Measures should be undertaken to prevent the localisation of the energy dissipation by yielding. The use of probability methods basel on statistic data is recommended for the establishment of the characteristics of earthquakes (real or simulated).

1.8. To ensure the quality of the structure execution an adequate supervision is necessary.

2. DESIGN OF LATERAL STRENGTH, RIGIDITY, DEFORMABILITY, DUCTILITY AND ENERGY DISSIPATION ABILITY

2.1. The designer ought to make proper analysis of the lateral resistance, rigidity, deformability and ductility, energy dissipation ability with a view to obtain an optimum energy response of the structure. The above properties of the structure as a whole are consequences of the properties of the plastic hinges and structural members.

2.2. The basic properties of the plastic hinges (P.H.) are : yield moment, ultimate moment, plastic deformability, ductility and energy absorbtion capacity. All these properties can be estimated as follows:

- by direct tests under ulternative loading;
- by analogy with the results of other tests;
- by theoretical computation on the base of the rational procedures of the plastic methods under simplifying assumptions.

In the last method the amount of work that is theoretically necessary to break the plastic hinge in a static way represents a criterion value to the estimation of the energy dissipation capacity.

2.3. Design of the lateral strength of the structure.

2.3.1. The necessary lateral strength of the structure is not a precise value. For preliminary design tha base shear coefficient can be evaluated according to the official code requirements and the storey seismic shear force coefficients propor - tional to the elastic dynamic response. The yield values of the storey seismic shear force coefficients can be computed by multiplying the above values with proper coefficients. The distribution of the storey shear force can be made in proportion to the relative storey rigidity, taking into consideration the work of the interacted structural elements.

2.3.2. To obtain several lines of resistance, the storey shear force should be firstly devided into several parts; one part for each resistance line.

2.3.3. The strength design must be made directly for the yield stage by means of plastic rational methods, for the values of the stresses obtained by superposition of the stresses due to vertical loads with those due to yield seismic loads.

2.3.4. The final values of the base shear coefficient and of the storey shear force coefficients can be obtained on the criterion of obtimal energy by balance between the strength rigidity and ductility.

2.3.5. The strength design of structural members should be made for the ultimate stage when plastic hinges occur at both ends of the member. Between the plastic hinges all sections should work elastically. So, for R.C. members adequate arrangements of reinforcement are necessary to provide resistance against diagonal tension crack and flexural plasticity.

2.4. Design of rigidity - A rigidity analysis is necessary to obtain an optimal energy response. A strong increasing in rigidity ordinary entails for the respective storey a big localisation of energy dissipation by yielding. For this storey it is necessary to provide bigger strength and ductility in order to prevent an inadequate energy behaviour.

2.5. Storey plastic deformability and ductility.

2.5.1. Very high storey ductility factors can obtain by plastic hinging of beams which possess proper reinforcement of the plastic hinges (as steel percentage and compressive reinforcement). The storey ductility factor will decrease by plastic hinging of the columns because of the axial load influence.

2.5.2. For the estimation of the storey plastic displacement and of ductility factor it is recommended to take into account the results of different tests, or in absence, the theoretical values obtained by plastic rational methods.

2.5.3. Designers should pay attantion to the fact that different types of constructions have different ductility factors.

2.6. Storey load-displacement relationship.

2.6.1. For the preparation of the storey load-displacement relationship the strength and plastic deformability of all structural members should be taken into consideration.

2.6.2. The assumed storey load-displacement relationship should be ckecked up by engineering judgement with results of experiments under alternating load.

2.6.3. It is allowed to use simplified relationships: linear, elasto-plastic or bilinear hysteretic type.

2.6.4. In order to obtain an optimal energy response various types of rigidity and restoring force characteristics in elastic and post-elastic range will be assumed.

2.6.5. For elements in structures of prestressed concrete it is necessary to consider nonlinear elastic relationship.

2.6.6. The following simplified assumptions are recommended for the load-displacement relationship of masonry diaphragms in frame structures. For masonry diaphragms framed by R.C. members it is allowed to use linear elasticity with variable rigidity. Masonry diaphragms not framed behave like elastic material with brittle faillure.

2.7. Design of storey energy dissipation ability.

2.7.1. To determine the energy consumption by viscous damping (h) can be taken into consideration:

For steel structures	h = 0,01 ÷ 0,03 hcr
For R.C. structures	h = 0,04 ÷ 0,05 "
Supplement for masonry Diaphragms framed by steel or R.C. members	max h 0,05 "

2.7.2. The necessity to dissipate energy by yielding increases very strongly when the lateral strength of the structure is small and when the earthquake intensity increases. To increase the energy dissipation ability the most effective way is to increase the ductility of all plastic hinges. When there are difficulties to achieve very large amounts of energy dissipation by increasing the ductility, the second way is to increase the lateral resistance of the storey. The balance between the storey lateral strength and ductility ability should be a result of special analysis.

3. THE DESIGN OF PLASTIC HINGES AND JOINTS

3.1. The plastic hinges and the joints represent small material volumes but the structural safety depends on their proper design. Adequate details are necessary in order to ensure for plastic hinges, strength, high ductility, strain reversibility under alternating loads, plastic stability (avoidance of plastic buckling) energy dissipation ability by yielding. For the joints, measures are necessary to prevent the failure of the joint before the failure of plastic hinges occurs. It is necessary to improve the strength of the plastic hinges after the ultimate stage (damage stage) in order to obtain the carrying ability for shear force and axial load.

3.2. For the plastic hinges of R.C. members it is recommended low steel percentages, compressive reinforcement, high quality concrete, reinforcement of the confining concrete compressed zones by welded grills or helical reinforcement, measures against the plastic buckling of reinforcing bars. The most effective ways in order to obtain high ductility and energy dissipation ability by yielding are additional compressive reinforcement in case of beams, and in case of columns by decreasing the ration $n = \frac{P}{f'_c \cdot A_c}$

where : P - axial load

f'_c - concrete strength

A_c - total area of the concrete section

3.3. In case of steel members, particular care should be given to the avoidance of local plastic buckling and joining of members in the plastic hinge zone.

4. THE STRUCTURAL LAYOUT

4.1. The leading of the energy process development by proper structural layout should be a permanent concern of the designer. The concept of several lines of resistance offers an important way with this object. By fixing the rigidities of members, the strength capacities of various sections of members and the ductility factors such schemes of the succession of the resistance lines should be the choice that improves the energy response.

4.2. The frame structure

4.2.1. .o limit is set for the height of multi-storey buildings with frame structures.

4.2.2. On the basis of the concept of several lines of resistance, a frame building can have, besides the frames with elasto plastic behaviour, other frames that work elastically for bigger relative displacements and additional elements, such as masonry framed diaphragms and special elements (for example slit walls after Muto's concept).

4.2.3. Always in the analysis, the strength, rigidity, and energy dissipation properties of the masonry framed diaphragms should be taken into consideration. Sometimes, these can have unfavourable influence on the behaviour of the building to stronger earthquake. For the rigidity analysis it is allowed to consider linear hysteretic curves but with a variable rigidity in reverse proportion to the inter floor displacement (Fig .).

4.2.4. To improve the storey ductility ability of the frame it is recommended to obtain the occurance of plastic hinges in beams by proper choice of the strength ability of ends sections of the columns and of the beams.

4.3. The shear wall structures with windows or doors openings. In this case the tie-beams that can be effective members in dissipating the energy by yielding should represent the first resistance line. For this reason, for the tie-beams it is necessary to provide very high ductility factors and proper shear reinforcement. The vertical bands represent the second resistance line.

4.4. The load bearing masonry wall. Because the realised structures only of reinforced masonry are fragile, it is recommended to obtain the necessary strength and ductility by their association with steel in the following systems:

a. reinforced grout core;
b. reinforcement in horizontal joints or in vertical joints or in both directions;
c. load bearing masonry with an enclosed R.C. skeleton consisting of columns and collar beams.

The first system is the best. In the last system the R.C. skeleton works plastically even under vertical loads. In their analysis it is admitted to adopt the simplified scheme as a multiple truss in which the masonry panels work as compression struth. In this system the lateral strength is offered only by masonry panels. The R.C. skeleton can also dissipate energy. Proper design and reinforcement of joints are necessary in order to prevent a premature failure under the effect of compressive struths.

4.5. In the case of combinations of frame and shear walls acting together, with or without a core wall, special considerations for the interaction between the systems are to be taken into account concerning the strength, the plastic deformability and the energy dissipation ability

5. THE SEISMIC DYNAMIC RESPONSE ANALYSIS

5.1. It is recommended to carry out always a complete dynamic analysis. It is necessary to have the elastic response, the nonlinear response and the energy response, for different types of earthquakes and different intensities.

5.2. The energy response enabled to determine the variation in time for each storey and for the structure as a whole, of the energy dissipation process by viscous damping and yielding and of energy supplied to the structure since the beginning of the earthquake.

5.3. It is recommended to use simulated earthquake motions in relation with the ground characteristics.

5.4. In order to obtain an optimum structural solution the de-designer should apply a thinking of a cybernetic type for the leading of the energy response, using different "adjustment mechanism" (by proper changing of the lateral strength and rigidity distribution.

5.5. In the case of very complexe structures the designer should use the profited knowledge plus an engineering judgement in order to concieve a proper theoretical model for the dynamic analysis.

6. THE STRUCTURAL SAFETY ANALYSIS

6.1. Safety criteria are in reality energy criteria.
6.2. The complete seismic structural safety analysis of a structure should take into account variable values for the strength, rigidity, ductility factors and energy dissipation ability properties. Maximum, medium and minimum values should be determined under consideration of the statistical dispersion of the properties parameters.

6.3. Probability theory and statistical method should also be used in view to determine in probabilistic terms the type of earthquake motion and the different intensity until the extreme maximum intensity that can be expected in the region to occur.

6.4. To the usual responses (displacements, velocities, accelerations, shear force and overturning moments) it is also recommended to add the energy response.

6.5. The response for the structural safety of a structure should be expressed by the probability of survival to the strongest earthquakes that can occur in region, by the risk of structural damage, for stronger earthquakes, and by the probability to resist without structural damage to the standard intensity.

6.6. These give the possibility to obtain the probable behaviour of the structure and permit to exprime the structural safety in probabilistic terms on the basis of energy criterion. The response energy offers an active theoretical possibility in order to examine the mechanisms of the failure and collapse or of the energy unstability during the earthquake motion.

CONCLUSIONS

- The energy response is considered as an advanced development of the method of structural safety analysis that offers a better knowledge on the theoretical behaviour of the structure.

- By energy response analysis the designer can understand how much, who and how reserves should be provided for the energy dissipation ability for a probable behaviour of the structure.

- It is possible to lead the energy response by adequate mechanisms of regulation. This implies methods and thinking of the cybernetic type. So we can speak on "SEISMIC CYBERNETICS" (18).

- The energy concept represents a very good way to improve the existing codes.

- The earthquake resistant design should be an "energy design".

ACKNOWLEDGMENTS

The digital computer work was done on a IBM 1130 at the Computer center of the Institute of Seismology, Earthquake Engineering and Town Planning at the University of Skopje.

The authors are extremely thankful to Prof. Eng. Tiberije Kirijas, director of the Institute, for his encouragements. Also they wish to express their appreciation to Eng. Dimitar Petrovski, M. Sc., of the Computer center, for his cooperation in the preparation of the programme for the Energy Response and for his working out the numerical examples.

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TABLE 1a

VARIATION OF THE ENERGY DISSIPATION BY YIELDING IN A MULTI-STOREY STRUCTURE FOR DIFFERENT EARFHQUAKES MOTIONS AND DIFFERENT INTENSITIES

STOREY	PT.HU Sv=20	PT.HU Sv=40	PT.HU Sv=60	EC.Cs 100%
15	0.25	0.74	0.92	4.17
14	-	0.07	0.09	0.15
13	0.02	0.54	0.82	6.72
12	0.70	0.72	1.55	7.02
11	0.03	1.01	2.10	13.00
10	-	0.86	1.46	11.80
9	-	0.47	0.87	10.30
8	-	0.30	0.86	12.70
7	-	0.11	0.06	12.50
6	-	0.03	0.68	12.90
5	-	0.06	0.87	12.85
4	-	0.01	0.76	14.15
3	-	0.09	1.18	10.90
2	-	0.33	2.33	15.40
1	-	0.39	2.56	15.10
Σ	1.04 tm	5.73	17.51	159.66

PT-HV = MODIFIED PORT HUENEME 1957 MARCH 15 ACCELEROGRAM. S_v=20;40;60 =MAXIMUM VALUES OF THE VELOCITY SPECTRUM

TABLE 1b

DETERMINATION BY PLASTIC R.C. THEORY OD THE PLASTIC DEFORMABILITY FOR THE FIRST STOREY OF KOCO RACIN BUILDING (SEE PHOTO 4)

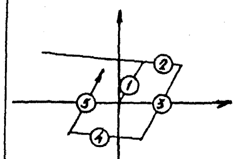
COLUMNS	NUMBER OF COLUMN	d YIELD	d PLASTIC	d ULTIM	RIGIDITY	Q YIELD
1	2	1,05	1,60	2.65	9.70	8.90
2	2	1,05	3,20	4.25	8.00	10.30
3	2	1,23	3,20	4.43	6.76	30.90
4	6	1,31	1.60	2.91	23.70	15.20
5	4	1,38	2.60	3.98	11.10	20.60
6	9	1,46	2.60	4.06	23.40	15.60
7	4	1,91	1.60	3.51	10.80	34.34
8	4	2,00	3.20	5.20	8.32	8.34
9	1	2,22	0.80	3.02	3.10	10.70
10	2	2,30	2.50	5.00	4.30	10.70
11	2	2,50	2.50	5.00	4.30	8.34
12	2	3,05	0.90	3.95	5.00	16.68

DISPLACEMENTS IN CENTIMETERS

RIGIDITY : t/sm.

Q YIELD – tons.

TABLE 2. ENERGY RESPONSE FOR KOCO RACIN BUILDING OF SKOPJE FOR A SHOCK TYPE EARTQUAKE (MODIFIED PORT HUENEME 1957–MARCH 15)



TIME HISTORY FOR ENERGY BALANCE $\frac{dE_K}{dt} + \frac{dE_P}{dt} + \frac{dE_V}{dt} + \frac{dE_Y}{dt} = \frac{dE_f}{dt}$

TIME HISTORY FOR FED ENERGY $E_f = E_K + E_P + E_V + E_Y$

stage	time	dE_K/dt	dE_P/dt	dE_V/dt	dE_Y/dt	dE_f/dt	E_K	E_P	E_V	E_Y	E_f
	34	331,80	-21,030	13,110	0,000	324.00	1742,00	9,026	106,50	206,10	2065
	35	281,60	-13,200	15,760	"	284.10	2024,00	-3,278	122,20	"	2349
	36	211.70	-2,671	17,770	"	226.80	2236.00	-5,950	140,00	"	2576
	37	137.70	9,620	19,270	"	166.60	2374,00	3,670	159,30	"	2743
3	38	61,37	22,820	20,060	"	104.20	2435,00	26,490	179,40	"	2847
	39	-16.70	36.200	20.260	"	39.76	2418.00	62.700	199,60	"	2887
	40	-100.20	48.890	19.800	"	-31.54	2318.00	111,600	219,40	"	2855
	41	-177.20	59.900	18.650	"	-98.73	2141.00	171,300	238.10	"	2756
	42	-234,50	68,280	16,880	"	-149,30	1906,00	239,700	255,00	"	2607
	43	-277,10	-7.835	14.760	71,520	-198,70	1629.00	231,900	269.70	277.60	2408
	44	-299.50	-8.771	12,330	65,320	-230.60	1329.00	223,100	282,10	342,90	2178
	45	-297.80	-9.119	9.816	58,230	-238,80	1032,00	214,000	291,90	401.20	1939
	46	-269.50	-8.938	7.444	50,700	-220,30	762.60	205,100	299.30	451.90	1719
4	47	-228,20	-8,343	5.382	43,170	-187.90	534.30	196,700	304.70	495,10	1531
	48	-175,90	-7,396	3,683	35,640	-144.00	358.40	189,300	308,40	530.70	1386
	49	-122,90	-6,357	2.443	29.000	-97.81	233.40	183,000	310,80	559.70	1289
	50	-79.81	-5,366	1.604	23.470	-60.10	155.60	177.600	312.40	583.20	1229
	51	-48,25	-4.553	1.081	19.260	-32.45	107.40	173.100	313,50	602,50	1196

TIME INTERVAL 0,01 sec.

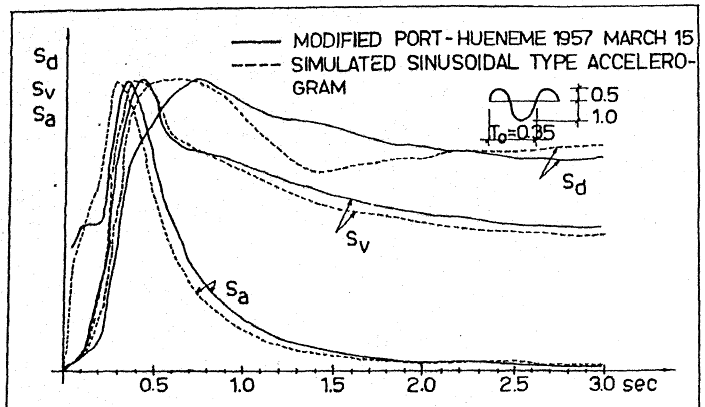
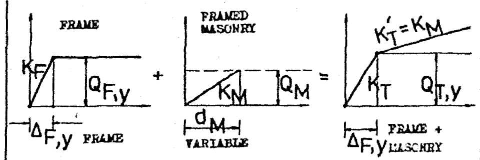


Fig. 1 THE COMPARAISON OF THE SHAPES OF RESPONSE SPECTRA FOR 2 SHOCK TYPE EARTHQUAKES

FIG.2 APROXIMATE RELATIONS TO DETERMINE THE RIGIDITY OF ELASTO PLASTIC FRAME STRUCTURES THAT WORK TOGETHER WITH FRAMET MASONRY DIAPHRAGMS.

NOTATIONS



ASSUMPTION: THE FRAMED MASONRY WORKS LINEAR-ELASTICALLY BUT WITH VARIABLE RIGIDITY

$$K_M = \frac{Q_M}{d_M(\text{VARIABLE})}$$

μ = FACTOR OF DUCTILITY $\quad \mu = \frac{Q_E}{Q_{T,y}}$

Q_E = THE STOREY SHEAR FORCE VALUE WHEN STRUCTURE WORKS PERFECTLY ELASTICALLY

$$\rho = \frac{Q_F}{Q_M} = \frac{\text{FRAME STRENGTH}}{\text{MASONRY STRENGTH}}$$

RELATIONS: RIGIDITY OF FRAMED MASONRY DIAPHRAGMS

$K_M = K_F \Phi_M$ WHERE $\Phi_M = \frac{1}{\mu(1+\rho)-1}$

RIGIDITY OF THE TOTAL STRUCTURE

$K_T = K_F \Phi_T$ WHERE $\Phi_T = \frac{\mu(1+\rho)}{\mu(1+\rho)-1}$

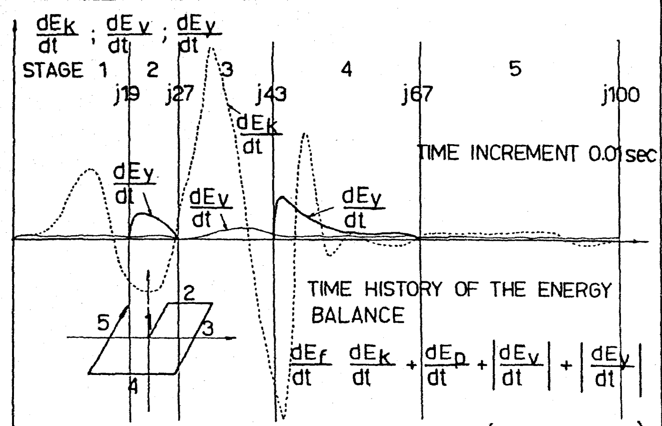


Fig. 3 ENERGY RESPONSE OF K. R. BUILDING (SEE PHOTO 4)

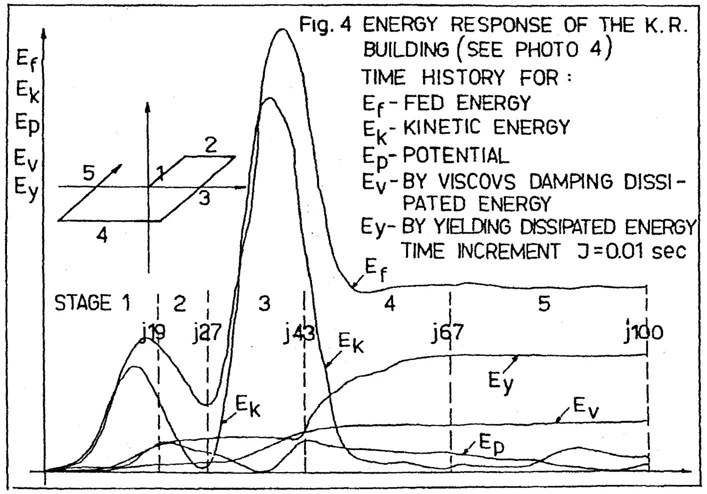


Fig. 4 ENERGY RESPONSE OF THE K. R. BUILDING (SEE PHOTO 4)
TIME HISTORY FOR :
E_f - FED ENERGY
E_k - KINETIC ENERGY
E_p - POTENTIAL
E_v - BY VISCOVS DAMPING DISSIPATED ENERGY
E_y - BY YIELDING DISSIPATED ENERGY
TIME INCREMENT J=0.01 sec

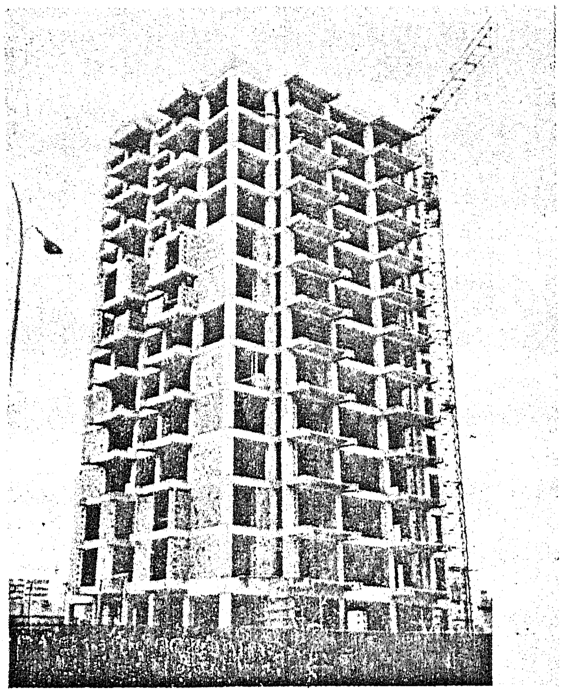


photo 1-New multi storey buildings in Skopje

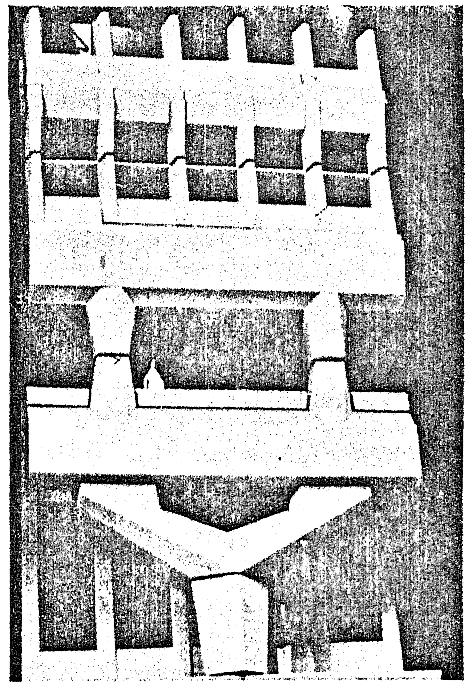


photo 2-New 9 story office building in Romania today under construction

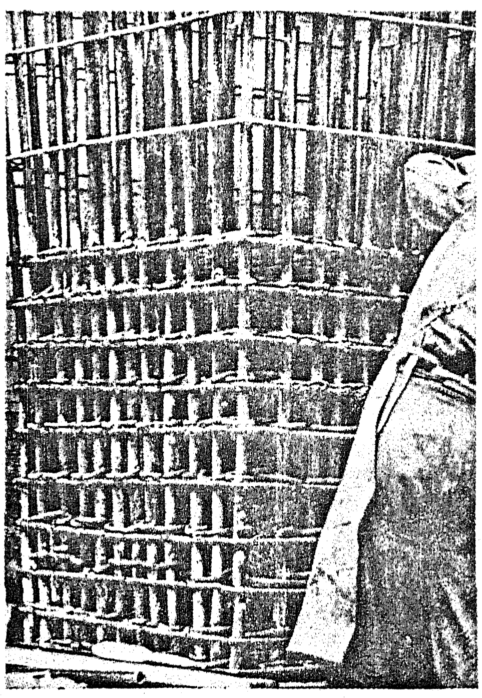


photo 3- Welded grills to increase the ductility of column (see photo 2)

Stage ultimate with concrete crushing in compressive zone occured in the plastic hinges of all columns in the first storey

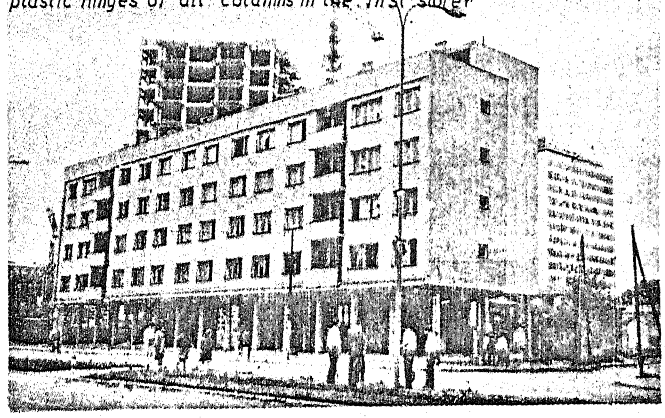


photo 4 -Koco Racin bld. of Skopje survived the 26 july 1963 earthquake