# THE INTENSITY OF GROUND MOTION OF THE SKOPJE 1963 EARTHQUAKE

By Apostol Poceski[I]

## SYNOPSIS

The intensity of ground motion, on the basis of energy absorbed by damaged structures, is analyzed. The analysis indicates maximum pseudorelative velocity response of about 90 cms. per sec. Such strong ground motion, associated with relatively small magnitude and limited destruction, is due mostly to the short duration of the strong motion, energy relief on small area, and amplification of the intensity on the ground surface.

## INTRODUCTION

The behaviour of structures during strong motion earthquakes is the best check of their appropriate design, design assumptions, theoretical investigations, and experimental results. The effects of earthquakes have been a subject of the greatest interest. However, these effects are a result of many factors involved, which make the investigations very difficult.

The strong earthquakes in epicentral area seldom have been instrumentally recorded. Not long ago the strongest earthquake instrumentally recorded was the El Centro 1940 earthquake, with maximum ground acceleration of 0.33g. What could be the maximum intensity of strong motion earthquakes has been a question of great importance. There have been some attempts to deduce the intensity of ground motion by analysis of structures subjected to earthquakes(1). The work carried out by the author of this paper was an attempt to indicate what could be the intensity of ground motion of the Skopje 1963 earthquake, also by analysis of structures.

The Skopje 1963 earthquake was of magnitude 6.0. The epicenter of the earthquake was about 8 km. north-east of the center of the city. Intensive damage of structures was distributed on small area, which indicates shallow focal depth, probably of only few kilometers. Most of the structures remained leaning on one side, indicating shock type earthquake.

The earthquakes of shock type have not been extensively investigated. Because they are of local character, they are not always recorded. Recent destructive earthquakes of the shock type have been, the Agadir(Morocco) 1960, and Skopje 1963 earthquakes. They showed that even relatively small magnitude shocks can be very destructive. The Parkfield 1966 earthquake proved that while shock type earthquakes can be of very strong ground motion, the intensity of destruction may not be high. A similar conclusion can be drawn for the Skopje 1963 earthquake.

[I] Assistant, College of Civil Engineering and Architecture, University of Skopje, Skopje, Yugoslavia.

## DAMAGE DISTRIBUTION

The damage distribution of the Skopje 1963 earthquake was specific. In a small area the intensity of damage is completely different. Figure 1 represents damage distribution in the city area. Heaviest destruction (cross-hatched area on Fig.1) was recorded in the central and west part of the city. Damage zone I represents region of heaviest damage and destruction, zone II - regions of heavy damage and occasional destruction, zone III - moderate damage, and zone IV - little damage.

From Fig.1 is clearly evident the sharp change of the intensity of damage. At a small distance there are damage zones I and IV. The epicenter of the earthquake was about 8 km. north-east from the center of the city, but heaviest damage is west of the center. This means the epicentral distance did not play any important role.

Heaviest destruction is associated with thickness of the alluvial layer of about 15-30 meters, and with a predominant period of microtremors of the layer of about 0.36 secs. A predominant period of seismic waves of about 0.3-0.4 secs., corresponding to the epicentral distance of about 8-10 km., seems possible. Thus, the surface layer probably vibrated with frequency near resonant frequency. K.Kanai's formula(4) for such a case gives an amplification of the intensity of motion on the surface of about 3 times. Similar amplification give analyses of structures in regions of different intensity.

The line of heaviest destruction goes along the belt of the cross-hatched area (Fig.1, west part of the city), and the south - west border of zone I. That is a belt where the thickness of alluvium sharply changes, from about 4-6 meters, to about 15-30 meters and more. It is interesting to point out, there were several destroyed buildings outside of the belt, on the side of shallow alluvium. Most of them were symmetrical, lying on gravel with sand of equal thickness. It seems the reason for destruction of these buildings, and all buildings along the belt, should be searched in occurence of twisting vibrations. The belt represents a border of two regions, one vibrating with large amplitudes (thick layer), and another vibrating with small amplitudes (shallow layer). As a result of this twisting vibrations were caused(5).

## ANALYSIS OF STRUCTURES

Damage of structures is the result of many factors. Therefore, by a study of the remaining effects on structures, it is hard to expect to give any precise result. The opinion,"One can get any answer he wants"(6), is quite true. However, in the case of lacking of any recorded data, this is the only way one can get an idea of what the intensity of ground motion could be.

In this case the analysis was easier than it might be expected, because the earthquake was of short duration. Most probably the response of structures which suffered long plastic deformations consisted of one long and big amplitude cycle, followed by small amplitude cycles. Otherwise, there could not have occured the case of all structures being tipped on one side, as they really were. For this reason the author considered an analysis of damaged structures could give reasonable results.

For the purpose of analysis 14 structures have been chosen. Five of them are reinforced concrete buildings, 8 brick masonry buildings, and 1 factory brick masonry chimney. In order to get results as good as possible, the structures chosen for analysis had to satisfy many requirements. First of all they had to be simple. In order to decrease the relative errors they had to have large plastic deformations, the damage to be concentrated in the first story for approximation of the vibration of the building as one degree of freedom system, and so on. Most of the requirements were satisfied by only 14 structures, although there are records for about 300 structures. The position of the structures in the city is shown on Fig.1, by numbers from 1 to 14. The appearance of structure No.1 is shown on Fig.2.

The analysis has been carried out on the basis of energy absorbed by the structures. The energy absorption by reinforsed concrete structures has been calculated upon the moment-rotation curves. The moments of the moment-rotation curves are calculated on the basis of limit analysis theory. The rotations are calculated by integrating the moment-curvature curves. The ultimate curvature is obtained by assuming the ultimate strain of concrete as 0.008. One example of moment-rotation relationship, of structure No.1 (Fig.2) is given on Fig.3

The energy absorption of brick masonry structures is computed, primarily, by taking the friction in the material along the cracks. The coefficient of friction is taken in value of 0.56-0.70.

Now the energy absorption has to be converted to response of equivalent elastic structures. The maximum energy stored in an elastic system, per unit mass, is in relation with the pseudorelative velocity response by the following expression,

$$S_v^2 = 2\,\ni_o \quad \ldots\ldots \tag{1}$$

where, Sv is pseudorelative velocity response, and $\ni_o$ is energy per unit mass stored in the system.

However, for non-linear systems there is not any such simple relationship. In order to solve the problem the relationship between the maximum displacements of linear and non-linear systems should be known. This relationship has been frequently investigated. It has been shown, for short period systems, the principle of conservation of energy approximately holds, and for long period systems, the maximum displacements of linear and non-linear systems can be taken as approximately equal. If the maximum response displacements of linear and non-linear systems are taken as equal, then from the absorbed energy in a non-linear system can be found the equivalent pseudorelative velocity response from the following expression,

$$S_v^2 = \frac{2\mu^2}{2\mu - 1} \, \ni_o \quad \ldots\ldots \qquad (2)$$

where $\mu$ is the ductility coefficient, ratio of total displacement witl the displacement at yield, and $\ni o$ is energy absorbed per unit mass.

For the purpose of this analysis, for the structures with natural periods longer than 0.3-0.4 secs., and ductility coefficient smaller than 10-15, it is assumed that maximum displacement of linear and non-linear systems are equal, i.e. the equivalent velocity response could be computed from equation 2. For the structures with short period it is assumed that the principle of conservation of energy holds,i.e. that the response could be computed from equation 1.

The results of the analysis are presented in the table and in Fig.5. The computed energy absorption $\ni o$ is given in column 6 of the table. In column 7 is given the computed velocity response according equation 1, that is by application of the principle of conservation of energy (Fig.4, OABO = OCDEO). Because the principle of conservation of energy completely does not holds, even for very short period systems, these results should represent lower boundary. The results given in column 10, the author thinks, should represent the probable velocity response with certain errors. Results No.1, 2, 6, 8 and 10 are computed according to equation 2. From the analysis of structures No.3, 4 and 5 the probable value could not be obtained, because the ductility coefficients are large. Because the periods of the structures No.7, 9, 11,and 13 are short, or the ductility coefficient is large, the probable value of the calculated response from analysis of these structures is the same as the lower boundary.

Most of the results of analysis of brick masonry buildings are obtained by application of equation 1. The resulting plastic deformation of these structures given in column 5 of the table contain some deformations from the oposite side, because the cracks in one direction did not close when the structure was deformed in the other direction of vibration. By assuming that the part of deformation in the first direction is about 0.4 of that in the direction of resulting damage, the amplification coefficient in column 9 is obtained, which is lower than corresponds to the ductility coeficient. The result of analysis of structure No.14, the factory brick masonry chimney, was obtained on the basis of appearance of cracks in the base of the chimney due to the first mode of vibration. Because there has been energy absorption along the cracks, this result should be the lower boundary. This is taken from preliminary analysis carried out in 1964.

The results 1, 2, 3 ..., presented in Fig.5, are taken from column 7 of the table, and should represent the lower boundary. The other results 1', 2', 6' .... are taken from column 10, and should represent probable value of the response. Result 2' is obtained from analysis of building No.2, which was on the belt of highest intensity of destruction. Therefore, this result is the highest point in the figure. The buildings No.6, 8, 9 and 10 were in the region of smaller damage intensity, damage zone II (Fig.1), and therefore they gave lowest results.

The curves A and B of Fig.5 would be the approximate response spectra for the most damaged region. Curve A is drawn so as to fit approximately the results 1', 11', 12' and 13'. It would represent the probable response spectrum. Curve B is drawn to fit the results of the lower boundary, and therefore it should represent the lower boundary of the response spectrum.

The accuracy of the results presented herein could not be expected to be good enough. A ductility coefficient of up to 10-15, for which equation 2 has been applied, seems pretty high. However, some analysis of response of elastic and elastoplastic systems subjected to the Port Hueneme 1957, Parkfield 1966, and Matsushiro 1966 earthquakes, indicate that maximum displacements of elastoplastic systems could be smaller than those of elastic systems, even in cases of much higher ductility than 10-15(9). In the analysis several assumptions have been made. However, because the pseudorelative velocity response is proportional to the square root of the energy absorbed, the errors of the computed velocity responses are much smaller than the errors of the computed energy absorption. Thus, although for the particular factors influencing the energy absorption rather large possible errors have been assumed, the probable error of the computed responses is found to be about $\pm$ 20%. Also, there are not enough points to draw complete and precise spectra curves. However, the curves as drawn do indicate that the maximum response in a strong motion earthquake, such as the one analyzed, can be very high in epicentral area. This was one of the main intentions of the study.

To the maximum velocity of Fig.5, of about 90 cms. per sec. corresponds a maximum response acceleration of about 1.5g. If an amplification coefficient of about 2.5-3 is assumed, maximum ground acceleration of about 0.5-0.6g is obtained.

These results indicate very high maximum intensity of ground motion. However, if there is taken in account the amplification of the intensity on the surface of about three times, the maximum acceleration of the bedrock arround the center of the city would be about 0.2g, which is not high value. A large part of the city, where the bedrock(neogen) appears on the surface and the alluvium is shallow, probably vibrated with maximum acceleration of that order. Therefore, it seems understandable why the intensity of damage on large city area was pretty small.

The author obtained the results presented herein in 1966, before he knew anything about the Parkfield earthquake of 1966. In that time he had in mind G. Housner's opinion about the maximum possible intensity of ground motion of an earthquake(7). Nevertheless, he obtained almost higher intensity than G. Housners sugestion about upper limit.

The Parkfield 1966 earthquake, of magnitude of about 5.5, maximum ground acceleration of 0.5g, and maximum response acceleration of more than 1.5g(8), proved that the results presented herein, for the Skopje 1963 earthquake, could be quite possible.

The author thinks the character and intensity of the Skopje 1963 earthquake was similar to that of the Parkfield 1966 earthquake. But the destruction intensity of the Parkfield earthquake was small, and that of Skopje earthquake pretty high. The difference in destruction probably mostly is due to the difference in the types of construction. In Skopje most of destroyed and badly damaged buildings were of brick masonry type. The reinforced concrete buildings, in general, survived the earthquake pretty well, although most of them had not been calculated on any horizontal forces. Thus, on the basis of the effect of the earthquake on ductile structures, as reinforced concrete structures usually are, the earthquake could be classified as one of small intensity.

The discrepancy between the high maximum intensity of ground motion found in the analysis, and relatively small intensity of damage of reinforced concrete buildings of the Skopje 1963 earthquake, and the similar discrepancy of the Parkfield earthquake, seems mostly due to the short duration of the earthquake. Thus, the duration of the strong ground motion for the intensity of destruction of an earthquake appears to be one of most important factors. It may be even more important than the maximum intensity of ground motion.

An explanation of such characteristics of the earthquakes of short duration (or shock type), could be given by an elastoplastic analysis. Analyses of response of elastoplastic systems on the Port Hueneme 1957, Parkfield 1966, and Matsushiro 1966 earthquakes, all of short duration, indicate lower response of elastoplastic systems than the response of elastic systems, much lower than could be expected, even of systems of pretty low yield level.

## DESIGN RECOMMENDATIONS

The effect of an earthquake on rigid, short period structures, mostly depends on the maximum intensity of ground motion, while the response of longe period structures very much depends on the duration of the strong motion. If the opinion of some investigators , that the earthquakes in the future probably would be of character similar to those occured before, could be accepted, then in Skopje should be recomended building of flexible, long period , ductile structures.

The most important thing, especially for this type of earthquakes, is that the structures have to be ductile. The design seismic forces could be pretty small, but the structures should be properly designed. The curve of Fig.6 could be recomended as a design spectrum for ductile material structures in the most damaged region.

The seismic forces which Fig.6 gives are many times smaller than seismic forces produced by the earthquake. A structure designed according to the present codes, on the seismic forces Fig.6 provides, subjected to an earthquake like Skopje 1963, would have ductility coefficient of up to 6-8. This ductility coefficient, because earthquakes in Skopje seldom occur, does not seem high. For long periods the seismic coefficients of Fig.6 are smaller than the same of SEAOC codes, for instance, because the earthquakes of short duration give lower response of structures with long periods than that of the earthquakes of "long duration".

Brittle material structures, such as masonry structures, should not be built, especially in the most damaged region. Along the belt of sharp change of the thickness of alluvium, new buildings should not be built.

## CONCLUSIONS

The results of the intensity of ground motion have been obtained in a way which does not guaranty accuracy. However, they could be an indication of possible intensity of strong ground motion.

The maximum pseudorelarive velocity response could be about 90 cms. per sec., maximum acceleration response about 1.5g, and maximum ground acceleration about 0.5-0.6g, with predominant period of about 0.3-0.4 secs.

The maximum intensity of ground motion was very high. However, because the strong motion was short, the damage intensity of the earthquake was small, especially in the intensity of damage of ductile material structures. Good earthquake resistant structures, in the case of earthquakes of such character, can be built without much increase of cost.

## REFERENCES


1. J. A. Blume,"A Structural-Dynamic Analysis of Steel Plant Structures Subjected to the May 1960 Chilean Earthquakes", Bulletin of the Seismological Society of America, Vol.53, Feb. 1963.
2. R. Clough, R.Binder, T.Hggins, and W.Kirkland,"The Agadir, Morocco Earthquake", American Iron and Steel Istitute, New York 1962.
3. G. Berg,"The 1963 Skopje Earthquake", Proceedings of the Third World Conference on Earthquake Eng., Vol.III, New Zealand 1965.
4. K. Kanai,"An Empirical Formula for the Spectrum of Strong Earthquake motion", Proc. Second world Conference on Earthquake Eng., Vol.III, Tokyo 1960.
5. A. Poceski,"The Ground Effect of the Skopje 1963 Earthquake", Bulletin of the Seismological Soc. of America (In print).
6. G. Berg, Reply on "The 1963 Skopje Earthquake", By G.Berg, Discussion by A. Poceski, Proc. Third world Conference on Earthquake Eng., Vol.III, New Zealand 1965.
7. G. Housner,"Intensity of Earthquake Ground Shaking Near the Causative Fault", Proc. Third World Conf. Earthquake Eng., Vol.I, New Zealand 1965.
8. G. Housner and M. Trifunac,"Analysis of Accelerograms - Parkfield Earthquake", Bulletin of the Seis. Soc. America,Vol.57, Dec.1967.
9. A. Poceski,"Response Spectra for Elastic and Elastoplastic Systems Subjected to Earthquakes of Short Duration", Proc. Fourth World Conf. Earthquake Eng., Chile 1969.

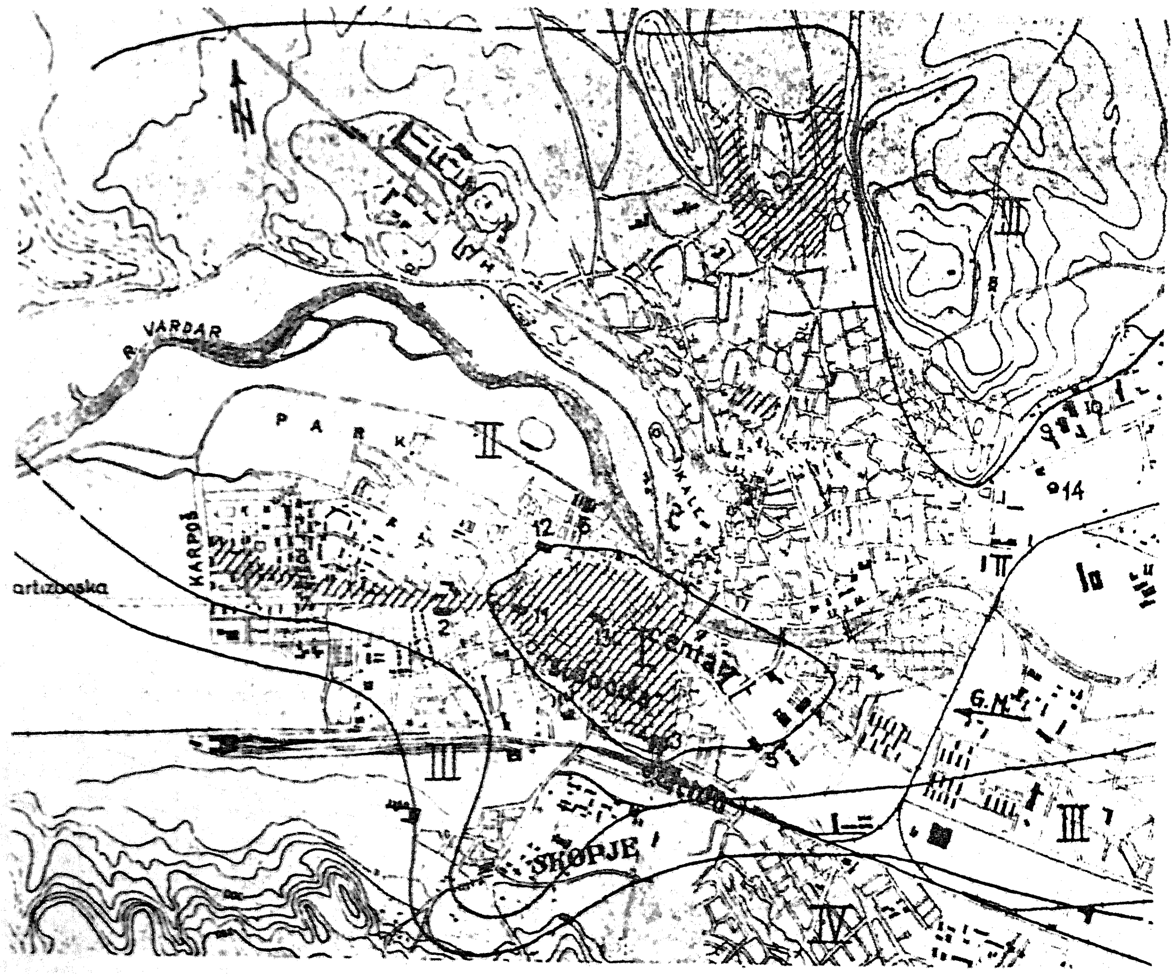


FIG.1 DAMAGE DISTRIBUTION, SKOPJE 1963 EARTHQUAKE.

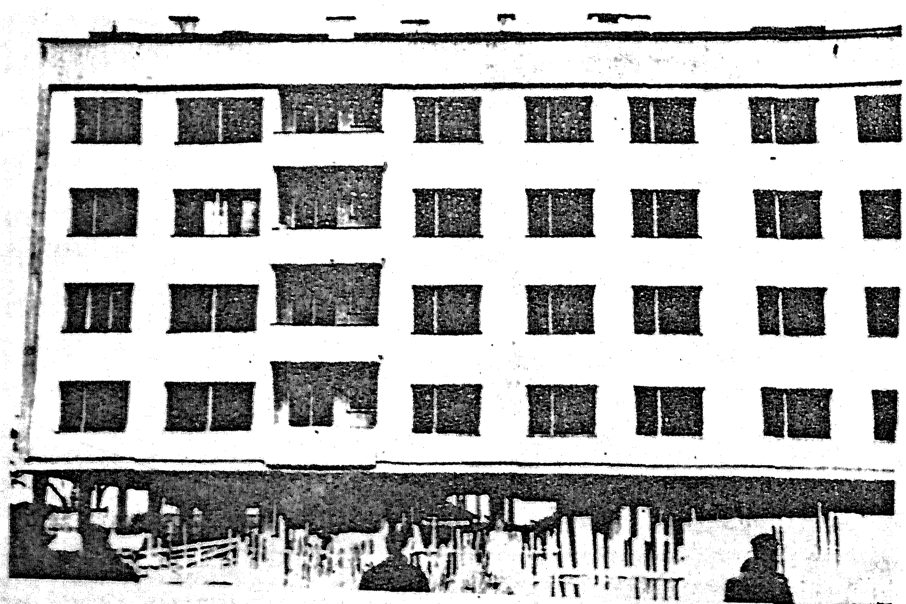

FIG.2 FRONT VIEW OF THE CONSTRUCTION No.1

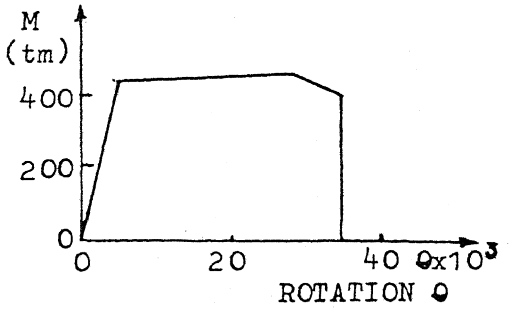


FIG.3 MOMENT - ROTATION CURVE OF STRUCTURE No. 1

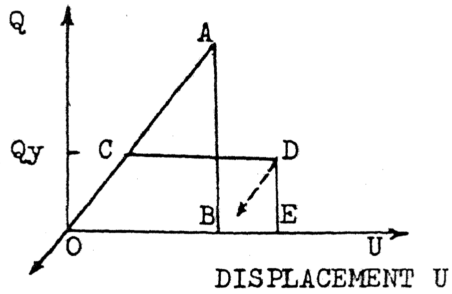


FIG.4 FORCE - DISPLACEMENT CURVE OF ELASTIC AND ELASTOPLASTIC SYSTEM

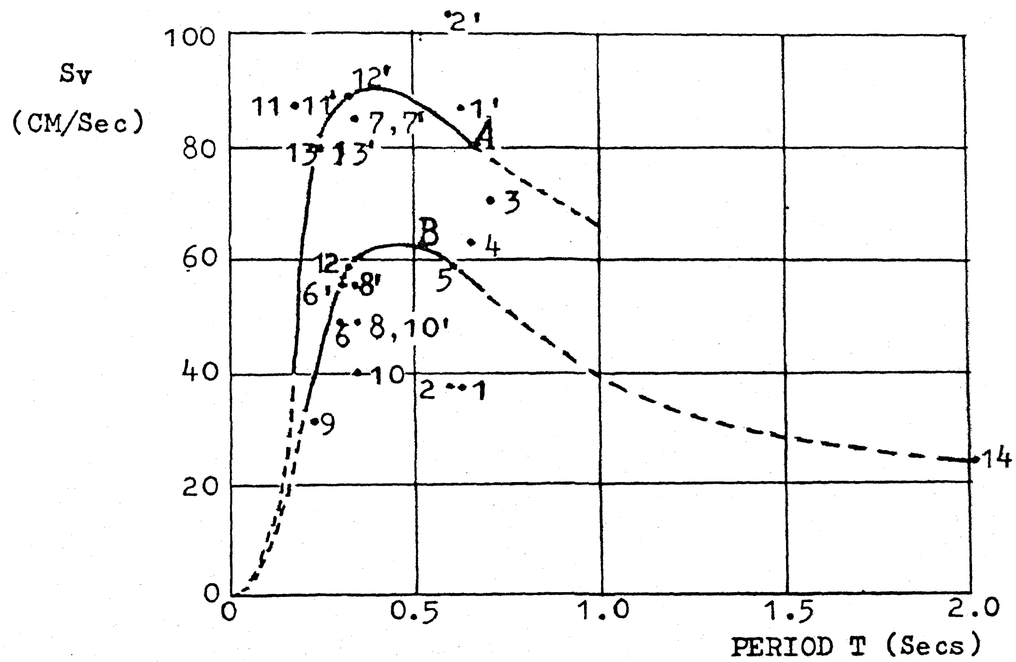


FIG.5 PSEUDORELATIVE VELOCITY RESPONSE SPECTRUM OF THE SKOPJE 1963 EARTHQUAKE; A - PROBABLE VALUE, B - LOWER BOUNDARY(BASED ON STUDY OF DAMAGED STRUCTURES)

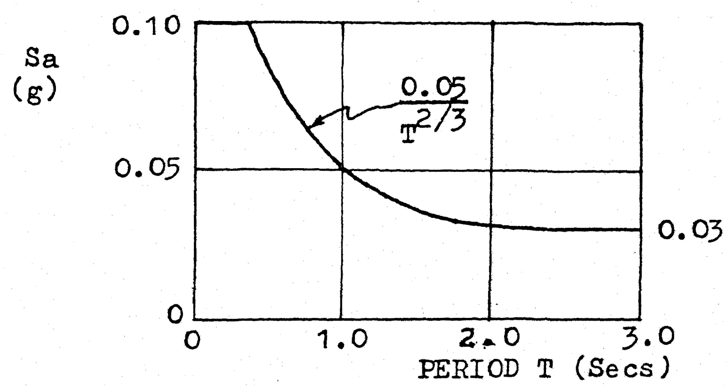


FIG.6 PROPOSED DESIGN SEISMIC COEFFICIENTS

TABLE - RESULTS OF COMPUTATION OF PSEUDORELATIVE VELOCITY RESPONSE

| No. | TYPE of the Structure | Numb. Story | Period Secs. | Def. cm. | Abs.Ener $\partial_o$ $cm^2/sec^2$ | $Sv=\sqrt{2\partial_o}$ cm/sec. | Duct. $\mu$ | $\eta=\sqrt{\frac{2\mu^2}{2\mu-1}}$ | $Sv=Sv\cdot\eta$ cm/sec |
|---|---|---|---|---|---|---|---|---|---|
| 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 |
| 1 | Reinforced Concrete | 6 | 0.63 | 6.0 | 687 | 37.1 | 10.4 | 2.34 | 87.0 |
| 2 | Reinforced Concrete | 6 | 0.60 | 7.6 | 715 | 37.8 | 14.9 | 2.75 | 104.0 |
| 3 | Reinforced Concrete | 7 | 0.70 | 13.5 | 2920 | 70.3 | 47.5 | - | - |
| 4 | Reinforced Concrete | 7 | 0.65 | 9.5 | 2010 | 63.5 | 20.7 | - | - |
| 5 | Reinforced Concrete | 6 | 0,60 | 16.7 | 1710 | 58.5 | 34.0 | - | - |
| 6 | Brick Masonry | 4 | 0.30 | 1.0 | 1180 | 48,6 | 3.0 | 1.13 | 55.0 |
| 7 | Brick Masonry | 5 | 0.35 | 5.0 | 3670 | 85.0 | 12.6 | 1 | 85.0 |
| 8 | Brick Masonry | 5 | 0.35 | 1.0 | 1185 | 48.7 | 3.0 | 1.13 | 55.0 |
| 9 | Brick Masonry | 3 | 0.23 | 0.5 | 477 | 30.9 | 1.9 | 1 | 30.9 |
| 10 | Brick Masonry | 5 | 0.35 | 1.2 | 800 | 40.0 | 3.6 | 1.22 | 48.8 |
| 11 | Brick Masonry | 2 | 0.17 | 10.0 | 3800 | 87.2 | 34.0 | 1 | 87.2 |
| 12 | Brick Masonry | 5 | 0.32 | 2.2 | 1601 | 58.2 | 6.0 | 1.53 | 88.7 |
| 13 | Brick Masonry | 3 | 0.23 | 5.0 | 3170 | 79.7 | 12.6 | 1 | 79.7 |
| 14 | Brick Masonry Chimny | | 2.04 | Based on appearance of cracks | | 23.4 | - | - | - |