LIQUEFACTION-ASSOCIATED GROUND DAMAGE DURING THE VRANCEA EARTHQUAKE OF MARCH 4, 1977

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ABSTRACT

During the Vrancea earthquake of March 4, 1977, severe damage was incurred to the ground in the old riverbed deposit of the Dimbovitza river in the city area of Bucharest, Romania, due to the development of liquefaction. Following the earthquake extensive in-situ investigations were carried out to clarify subsurface soil conditions by means of the Dutch cone penetration test and the Swedish automatic ram sounding test. On the basis of the results of these tests, analyses were made for the liquefaction at two sites in this area ; one site where apparent signs of liquefaction were observed and the other site where no such sign was visible on the ground surface following the earthquake. The result of the analyses indicated that liquefaction could occur in the sand deposit underlying the surface layer composed of clayey silt. However, because the surface crust was not thick enough at one site, it was broken up and the liquefied sand migrated through the fissures upwards on the ground surface. In contrast to this, the surface layer at the other site was sufficiently thick to prevent such breach and consequent sand ejection from occurring during the earthquake.

One of the most alarming aspects of the collapse in the ground was several huge sinkholes which developed in the flood plain area of the Danube river near the town of Giurgiu in Romania. As a result of boring investigation, cavernous limestone formation was found to exist at a depth of about 20 m below a thick sand deposit of alluvial origin. It is then speculated that the cave-in must have taken place as a result of raveling failure in the weathered upper portion of the limestone formation. In order to provide some quantitative interpretation to the mechanism of the sinkhole development, an instability analysis was made with respect to the seepage conditions in the weathered limestone layer which might have been altered due to the increase in pore water pressure in the upperlying alluvial sand deposit during the earthquake. The result of the analysis indicated that the seepage-associated instability caused by the increase in pore water pressure in the sand deposit is most likely to be an immediate cause of the catastrophic sinkhole development in the ground during the Vrancea earthquake.

Key words : earthquake damage, liquefaction, seepage (IGC : B 4/C 3/E 8)

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INTRODUCTION

Vrancea earthquake of March 4, 1977 caused severe damage to buildings and several kinds of civil engineering structures in a widespread region in Romania and Bulgaria. General feature of the disruption incurred by the earthquake is described in reconnaissance reports by Tokyo Metropolitan Government (1977) and also by Tezcan et al (1978). The feature of the ground failure such as landslides and liquefaction of sand deposits is described somewhat in detail by the papers by Mandrescu (1978, 1981), Botea et al (1980) and Perlea (1981).

Apart from the damage due to landslides, the main feature of the ground damage was the widespread occurrence of liquefaction in sand deposits near rivers and the development of large sinkholes in the flood plain of the Danube river near Giurgiu, Romania. Since the liquefaction occurred in so many places and so sparsely populated areas that only a few cases have been investigated to clarify their causes. The most closely investigated area among others was the area of the old riverbed deposit of the Dimbovitza river located in the city of Bucharest. Over thirty borings ever performed in this area were collected and a series of the penetration tests was newly conducted to clarify the soil profile and to provide some basic soil property data for the further analysis of the failure. The first part of this paper describes the feature of the liquefaction-induced ground damage, the results of subsurface soil investigations and the analysis conducted for the liquefaction of the sand deposit in this area.

In view of the most peculiar nature of the large sinkhole development and also of its engineering significance, extensive investigations have been carried out to clarify its cause of occurrence. The site of the biggest sinkhole has been thoroughly investigated by means of a series of borings, penetration tests and in-situ pumping tests. The results of these in-situ studies and simple speculative consideration for the cause

of the event will be described in the later part of this paper.

VRANCEA EARTHQUAKE OF MARCH 4, 1977

A severe earthquake occurred on March 4, 1977 in the Vrancea region some 150 km north-northeast of Bucharest, Romania. Its magnitude 7.2 placed this event among the greatest earthquakes to have occurred in Europe in recent decades. The epicenter was located in the mountain near Vrancea at the bents of the Carpathian Arc extending eastward across central Romania and breaking sharply northward, as shown in Figs. 1 and 2. The focal depth is estimated to have been 97 km. The earthquake shock the widespread area covering the south to the

Fig. 1. Major cities where ground damage was incurred during the Vrancea earthquake

Fig. 2. Intensity distribution of the Vrancea earthquake of 1977 in terms of the MSK scale

south-east part of the Romanian territory. Intensities of shaking in terms of the Russian MSK scale which is similar to the Modified mercalli scale were greatest at Bucharest, 8, diminishing to between 7 and 8 in Zimnicea on the Danube 100 km southwest of Bucharest. The intensity assigned to the earthquake-inflicted area is indicated in Fig. 2. One of the unusual features of the overall intensity distribution was the elongated shape of strongly shaken area extending from the northeast to southwest direction spanning a distance of about 200 km from the epicenter. It should also be noted that some isolated areas far from the epicenter were locally shaken with an extraordinary high intensity. For example, the area of Craiova about 300 km apart from the epicenter was shaken with the intensity as great as 7 to 8 on the MSK scale.

The Romanian territory had suffered many earthquakes previously in the historical time. One of the most recent which occurred at very nearly the same epicentral location was the earthquake of November 10, 1940 with a magnitude of 7.4. The asymmetric pattern of intensity distribution as shown in Fig. 2 was also noted in the 1940 earthquake, but was more confined to the northeast than the event in 1977. For example, the Focsani region was assigned an intensity of 9 in 1940, but it was shaken only moderately in the 1977 event.

During the 1977 earthquake, acceleration records were obtained in the basement of a one-storey reinforced concrete frame building located at the Building Research Institute (INCERC) in the eastern part of the city (Fig. 3). The instrument was a Japanese-built three-component SMAC-B type accelerograph having a 10 Hz natural frequency. The soil profile beneath the building is shown in Fig. 4, where it may be seen that a silty loam layer of about 4.5 m exists over a sand and gravel deposit. The time histories of the recorded accelerations are shown in Fig. 5 and the response spectra are shown in Fig. 6. The most notable feature of the ground accelerograph record

Fig. 3. Location of the recording house and place of liquefaction in the city of Bucharest

Fig. 4. Approximate soil profile at the site of the SMAC recording house

Fig. 5. Acceleration records obtained at the INCERC building, Bucharest

(a)

(b)

Fig. 6. Acceleration response spectra of the motions recorded at the basement of INCERC building in Bucharest during the March 4, 1977 Vrancea earthquake

is the large amplitude pulses amounting to 213 gal and 173 gal in NS and EW directions, respectively, that occurred about 20 seconds after the instrument was triggered. Following each pulse, the accelerations were lower in amplitude and higher in frequency. By contrast, there were no long-period pulses in the vertical component where accelerations were generally in the 8-10 Hz frequency range with the maximum acceleration of 115 gal.

GENERAL FEATURE OF THE GROUND DAMAGE

The region most severely inflicted by the Vrancea earthquake was the outer zone of the Carpathian Arch Bend. This region corresponding to the southern and eastern part of the Romanian territory is topographically a relatively flat plain covered by

windblown deposits. The surface windblown silty deposits are eroded by many tributaries joining the Danube river along the southern border of the country. The channel and flood plain deposits along these tributaries and their confluences consist of alluvial deposits of sand and silt. Most of the ground damage incurred by the earthquake were concentrated on the loose and soft deposits along these rivers. Cracking and sand boiling were the typical feature of the ground damage observed in these areas following the earthquake. The locations where signs of liquefaction such as sand boils and fissures were observed are indicated in the map of Fig. 7.

Among these inflicted localities, the area

Fig. 7. Locations where signs of liquefaction were observed following the Vrancea earthquake

Photo. 1. Cracks in the wall of a barn due to liquefaction-induced uneven settlements of the ground

south of the village of Bragadiru (Fig. 1) was most severely devastated by the liquefaction occurring over the flood plain deposit of the river Vedea. Sand and water was reportedly seen gushing out of a series of long cracks shortly after the earthquake. Photo. 1 shows cracks which developed in the walls of a barn in the neighborhood as a result of uneven settlement due to liquefaction. The village of Bragadiru is located on a terrace composed of windblown loess or fine grained fluvial deposits. The terrace is about 3 m to 5 m above the nearby swampy flood plain where signs of liquefaction were observed. Lateral spreading accompanied by a graven 3 m to 5 m wide occurred at the edge of the terrace, causing some disruptions to several houses.

In the stream deposits along the tributaries and confluences of the Danube meadow, signs of liquefaction such as gushing of sand and silt were observed at numerous places following the earthquake.

SUBSURFACE SOIL CONDITIONS IN BUCHAREST

The city of Bucharest lies on the platform of Paleozoic, Mesozoic and Neozoic rock formations consisting mainly of crystalline schists. Over the platform, windblown loess deposits exist, but this general geomorphological condition was disturbed in the

Fig. 8. Geological and topographical map of the city of Bucharest

city area by erosive and depositional action of two main rivers which took place in the more recent Quaternary era. Fig. 8 shows the geological feature of the city of Bucharest, where it can be seen that the flat platform area outside the city is covered by the loess deposit, whereas diluvial and alluvial clay or sand exists along the rivers and the city area. The Colentina river, flowing from northwest to southeast through the northern part of the city, has the natural meandering pattern. However, the Dimbovi-

Fig. 9. Subsurface soil conditions through the city of Bucharest

Fig. 10. Subsurface soil profile near the Dimbovitza

tza river flowing on the south side presents a simple and rather straight-line shape, because its channel was modified artificially and aligned as it is now at the end of the last century. Typical cross sections of the city area are shown in Figs. 9 and 10. It may be seen that, underneath the loess or the clayey loess, there exists a layer of sand with gravel which is generally medium dense to dense in the central part of the city, but loose near the riverbed of the Dimbovitza. This gravelly sand layer offers a sound base for the spread foundations for most of the buildings in the city of Bucharest. Below this layer, there exists a deposit of predominantly clay containing sand and silt underlaid by a sand deposit. These deposits, being of diluvial origin, are indurated to some extent due to overconsolidation and cementation. Underlying this sand deposit is a layer of uncemented clayey shale or clayey marl which is encountered at a depth of about 50 m. As can be seen in the cross sections, B-B', C-C' and D-D' of Figs. 9 and 10, the old riverbed deposit of the Dimbovitza before its channel was aligned forms a flat low-lying area with a high elevation of the ground water table. Since the near-surface deposit is of predominantly sand, liquefaction developed at one location in the riverbed deposit indicated in Fig. 3.

LIQUEFACTION IN THE CITY OF BUCHAREST

General Description

The occurrence of liquefaction in the city of Bucharest during the 1977 Vrancea earthquake was limited to one small area in the old riverbed of the Dimbovitza where the subsurface soil was predominantly of sand. The location is indicated in Fig. 3. The overall geomorphological profile around this area is shown in the D-D' section of Fig. 10, where it may be seen that the sand containing gravel exists underneath the surface clay silt layer. Following the 1977 earthquake, signs of liquefaction such as ejection of fine sand through the fissures or

Photo. 2. Sand ejection through cracks in the riverbed of the Dimbovitza

cracks were observed here and there in this area. Photo. 2 shows typical sand ejection over the ground surface. It is to be noted that, because of the presence of the cohesive soil layer near the surface, the liquefied sand was unable to gush through hole-shaped channel by breaking up weak spots in the surface layer, as is generally the case with liquefied deposits overlaid by cohesive soils. In the present case where the liquefied deposit is covered by the crust of cohesive soils, fissures or cracks developed first and then the sand came out through them over the ground surface. Therefore, the resulting sand volcanoes were aligned in one direction over the ground surface, as shown in Photo. 2, in parallel to the fissures developed in the surface crust of cohesive soils.

The area in the old riverbed of the Dimbovitza was very sparsely populated and most of the land was used for farming. A few farmer's houses constructed just on the place of sand ejection were seriously affected by the disruption such as sinking of columns and uneven settlement. Otherwise, the damage was to crops in the farms.

Individual locations where the sand vents were observed following the earthquake are indicated in Fig. 11, where it may be seen that they developed mostly in the area nearer the cliffs to the highland on the south. A number of existing boring logs over this area were collected after the earthquake to investigate the soil conditions. The borings

Fig. 11. Locations of sand boils and soil investigations in the old riverbed area of the Dimbovitza, Bucharest

consisted of three sets each to different depths. Three borings indicated soil profiles to a depth of about 20 to 25 m. Another set of borings indicated soil profiles to an intermediate depth of about 10 to 12 m and still other set to a shallow depth of 8 to 10 m. Individual locations of these borings are indicated in Fig. 11. Typical soil profiles obtained from the three deep borings at the location, H3, H4 and H5 are shown by the logs in Fig. 12. It may be seen in these figures that the sand layer containing gravel exists between the depths of about 5 m and 15 m and above this layer there exists a surface layer about 5 m thick consisting of clayey silt with variable sand contents. Fig. 12 also shows that the gravel-containing sand layer is underlaid by layers of clayey silts or silty clays with variable thicknesses. Observation of all the boring logs including those shown in Fig. 12 indicates that the entire area is covered by a surficial layer of clayey silts varying in depth from about 3 m to 6 m underlain by fine or coarse sand containing some gravel.

Laboratory Test

The sand ejected on the ground surface was sampled and taken to the laboratory

(a)

(b)

(c)

Fig. 12. Soil profiles in the old riverbed area of the Dimbovitza

Fig. 13. Grain size distribution curve of the sand from the Dimbovitza riverbed deposit in Bucharest

to perform several kinds of test. The result of the sieve analysis showed the grain size distribution curve indicated in Fig. 13, where the mean grain size, D_{50} , is read off as 0.2 mm. The specific gravity of the sand was 2.67. A picture of microscope indicates that individual grain of the sand is sub-angular. The maximum and minimum void ratios are $e_{max}=1.045$ and $e_{min}=0.581$ according to the test procedure recommended by the Japanese Society of Soil Mechanics and Foundation Engineering (JSSMFE, 1979).

The cyclic triaxial tests were conducted on the sand specimens reconstituted to a relative density of approximately 67%. The method of pluviation was employed to prepare the sand specimens. Details of procedures for the sample preparation are described elsewhere (Ishihara and Ansal, 1982). The number of cycles required to cause initial liquefaction, 5% and 10% double-amplitude

Fig. 14. Cyclic stress ratio versus the number of cycles for the sand from the old riverbed of the Dimbovitza

Fig. 15. Relationship between the cyclic strength and relative density for clean sands

axial strain was determined for given cyclic stress ratios, $\sigma_{dl}/(2\sigma'_0)$, where σ'_0 is the initial confining pressure and σ_{dl} is the amplitude of the axial stress employed in each test run. The results of the tests are presented in Fig. 14 in terms of the cyclic stress ratio versus the number of cycles, where it may be seen that the cyclic stress ratio causing 5% double amplitude axial strain in 20 cycles was 0.14. This value is plotted in Fig. 15 in which the similarly determined cyclic stress ratios for other typical sands are plotted versus the relative density of the test specimen. Fig. 15 indicates that the cyclic stress ratio causing 5% double-amplitude strain for the Bucharest sand is somewhat lower than that for other sands such as Japanese and United States standard sands.

In-Situ Penetration Test

For the purpose of gaining insight into the resistance to liquefaction of the sand, two kinds of penetration tests were conducted at two sites in the riverbed deposit of the Dimbovitza. These were the Dutch cone penetration test and the automatic ram penetration test developed in Sweden. The latter test consists of measuring the number of blows per 20 cm penetration produced by a drop hammer of 50 kgf falling through a height of 50 cm. In the test standardized in Sweden, a drop hammer weights 63.5 kgf, but in the present investigation, 50 kgf hammer was used instead. One of the novel features of this machine lies in the way the

skin friction is measured and used for correction to obtain a true value of the point resistance. When operating in the field, the vertical rod is turned two complete revolutions each time the cone penetrates through a stroke of 20 cm. The maximum torque during the rotation of the rod is monitored to correct the measured resistance.

Two sites indicated in Fig. 11 were selected for the in-situ penetration tests. Site 1 is located in the vicinity of the boring F29

Fig. 16. Penetration resistance by means of the Dutch cone penetration test

Fig. 17. Penetration resistance by means of the Dutch cone penetration test

where signs of liquefaction were clearly manifested on the ground surface during the 1977 event. Site 2 is located about 50 m distant from the boring F315 where no sign of liquefaction was identified at least on the ground surface. The results of the Dutch cone penetration test are presented in Figs. 16 and 17, where the point resistance from a pair of tests conducted in parallel is plotted versus depth of the deposit. The soil profiles from the logs of the borings F29 and F315 are also shown in Figs. 16 and 17, respectively. These figures show that the point resistance by the Dutch cone is practically the same both at sites 1 and 2, as far as the surficial layer of clayey silt is concerned, giving a value from approximately 2 000 kN/m² to 10 000 kN/m². Below this surficial layer, a sand deposit exists at site 1 where the point resistance is approximately on the order of 5 000 kN/m². At site 2, the sand deposit contains some percentage of gravel and, therefore, the cone resistance is much greater.

Shown in Figs. 18 and 19 are the depthwise distribution of the cone resistance by the Swedish automatic ram sounding test. Two types of penetration tests were carried out using the same test machine. In one type

Fig. 18. Penetration resistance by means of the Swedish automatic ram penetration test

Fig. 19. Penetration resistance by means of the Swedish automatic ram penetration test

of the test, the vertical rod was revolved each time 20 cm penetration is finished, but in the other type the cone was advanced continuously without imposing any rotation. In Figs. 18 and 19, the results of the two types of tests with and without rod rotation are demonstrated together. It may be seen in these figures that for the depth range shallower than about 15 m there is no consistent trend showing the difference in the cone resistance between the two types of the tests performed with and without rotation of the vertical rod. As was the case with the Dutch cone tests, the cone resistance through the penetration of the surficial layer was almost the same for both of the sites being investigated. In the underlying sand deposit, the number of blows per 20 cm penetration was on the order of 20 for the site 1, but for the site 2 the blow count values increased to the order of 50 below the depth of 10 m where the sand contains presumably more percentage of gravel.

Simple Analysis of Liquefaction

The analysis of liquefaction during earthquakes requires a knowledge on the intensity of shaking most preferably in terms of the time history of acceleration on the ground

surface at a given site in question. The acceleration records obtained during the 1977 earthquake at a place nearest the Dimbovitza riverbed were those obtained at the basement of INCERC building (Fig. 4), located approximately 8 km northeast of the Dimbovitza site. As shown in Fig. 5, the maximum horizontal acceleration recorded was 213 gals in N-S direction and 173 gals in E-W direction. Although the subsurface soil conditions are somewhat different between the INCERC building site and the Dimbovitza site, it may well be assumed with a reasonable degree of accuracy that the maximum acceleration at the Dimbovitza site was approximately on the same order of magnitude as those recorded at the INCERC building site. Therefore, the value of 213 gals will be used in the following liquefaction analysis as having been the maximum horizontal acceleration at the sites being analyzed.

According to the simple analysis procedure developed by Ishihara (1977), the maximum stress ratio, $\tau_{\max}/\sigma_v'$, induced by an earthquake loading is computed by

$$\frac{\tau_{\max}}{\sigma_v'} = \frac{\alpha_{\max}}{g} \cdot r_d \cdot \frac{\sigma_v}{\sigma_v'} \quad (1)$$

where σ_v and σ_v' are, respectively, the total and effective vertical overburden pressure at a depth in question, and the stress reduction coefficient, r_d , is given by an empirical formula,

$$r_d = 1 - 0.015z \quad (2)$$

in which z denotes the depth in terms of meter. In Eq. (1), $\alpha_{\max}$ is the maximum horizontal ground acceleration and g is the gravity acceleration.

The cyclic strength, $\tau_{\max, l}/\sigma_v'$, to be compared against the induced maximum stress ratio, is determined by the following formula, on the basis of the cyclic triaxial shear test data which are expressed in terms of the cyclic stress ratio, $\sigma_{dl}/(2\sigma_v')$, required to cause 5% double amplitude axial strain in 20 cycles,

$$\frac{\tau_{\max, l}}{\sigma_v'} = \frac{0.9}{0.55} \frac{1+2K_0}{3} \left(\frac{\sigma_{dl}}{2\sigma_v'} \right)_{20} \quad (3)$$

where $\tau_{\max, l}/\sigma_v'$ denotes the maximum stress

ratio required to produce failure in the soil as represented by the development of 5% double amplitude strain under irregular loading condition. σ_{dl} is the amplitude of axial stress and σ'_0 is the initial allround confining stress in the triaxial loading conditions. K_0 is the coefficient of earth pressure at rest. The value, 0.55, in Eq. (3) represents a factor to take into account the effect of irregular nature of time history changes of shear stress during an earthquake. A factor, 0.9, was incorporated in Eq. (3) to allow for the effect of changes in the direction of shear stress application in the horizontal plane during an earthquake.

In the present study, the value of the cyclic stress ratio, $(\sigma_{dl}/2\sigma'_0)_{20}$ for the undisturbed state of in-situ sand deposit is not known. Therefore, the cyclic strength was estimated on the basis of the penetration resistance obtained by the Swedish automatic ram penetration test.

A recent investigation in Japan (Sato and Iwasaki, 1980) has shown that, in case a 63.5 kgf drop hammer is used, the Swedish automatic ram penetration test yields a value of point resistance which is nearly equal to the N -value of the standard penetration test. In the present in-situ test scheme, however, a drop hammer of 50 kgf was used instead of 63.5 kgf. Therefore, the blow count value of the automatic ram test using 63.5 kgf is assumed to be roughly equal to 50/63.5 times the blow count value obtained using a drop hammer of 50 kgf. The blow count value of the 63.5 kgf automatic ram penetrometer thus modified will be taken in the following analysis as being the N -value of the standard penetration test.

In order to estimate the cyclic strength of in-situ sand deposits on the basis of the N -value of the standard penetration test, attempts have been made to establish empirical correlations between these quantities. On the basis of the test data on undisturbed sand samples mainly from the alluvial deposits in Japan, Tatsuoka et al (1978) proposed the use of the following formula,

$$\left(\frac{\sigma_{dl}}{2\sigma'_0}\right)_{20} = 0.0882 \sqrt{\frac{N}{\sigma'_v + 0.7}} + 0.0035 C \quad (4)$$

For alluvial sandy soils, Ishihara (1979) suggested an empirical formula as follows,

$$\left(\frac{\sigma_{dl}}{2\sigma'_0}\right)_{20} = 0.009 \left[0.77 N \log \left(\frac{20}{\sigma'_v} \right) + 6.5 \log C + 13 \right] \quad (5)$$

where C is the content of fines in percent passing #200 mesh and σ'_v is the effective overburden pressure in kgf/cm². The above two empirical formulae will be used to estimate the cyclic strength value on the basis of the blow count value and fine contents shown in Figs. 18 and 19. The fine contents are defined as percentage of clay plus silt contents shown in these figures. The N -value is estimated as the average of the two test data in these figures times the correction factor, 50/63.5, as explained above. The value of the cyclic strength computed at each depth of the deposits at two site of the Dimovitza were introduced in Eq. (3), and assuming the K_0 -value to be equal to 0.5, the maximum stress ratio, $\tau_{max,l}/\sigma'_v$, required to cause initial liquefaction in the sand was determined. Then, the values of $\tau_{max,l}/\sigma'_v$ were compared against the corresponding maximum stress ratio, τ_{max}/σ'_v , which would have probably been induced in the soil deposit during the 1977 earthquake. The induced maximum stress ratio was estimated through Eq. (1) using the value of the maximum horizontal acceleration of 213 gals. The depth of the ground water table was taken to be 1.0 m from the ground surface.

Evaluation of liquefaction potential was made in two procedures. One method is in terms of the factor of safety, F_l , defined by

$$F_l = \frac{\tau_{max,l}/\sigma'_v}{\tau_{max}/\sigma'_v} \quad (6)$$

The other method is in terms of the residual pore water pressure ratio, u_r/σ'_v , where u_r is the residual pore water pressure which has built up in the sand after an earth-

quake-induced irregular load is applied to the sand. It is well understood that the residual pore water pressure ratio increases as the amplitude of the irregular shear stress, $\tau_{\max}$, is increased and eventually reaches a value of unity when the amplitude, $\tau_{\max}$, becomes sufficiently large to induce liquefaction in the sand. The relationship between the maximum shear stress ratio, $\tau_{\max}/\sigma_v'$, and the residual pore water pressure ratio, u_r/σ_v' , was investigated by Ishihara (1977) using the laboratory test equipments. In this type of the relationship, if the maximum stress ratio, $\tau_{\max}/\sigma_v'$, is normalized to its limiting value at liquefaction, $\tau_{\max,l}/\sigma_v'$, it is possible to obtain a curve which expresses the residual pore water pressure ratio, u_r/σ_v' , as a function of the maximum shear stress normalized to the maximum shear stress at liquefaction, $\tau_{\max}/\tau_{\max,l}$. Since the ratio, $\tau_{\max}/\tau_{\max,l}$, is equal to the reciprocal of the factor of safety, F_l , defined by Eq. (6), it becomes possible to obtain a relationship between the factor of safety and the residual pore water pressure ratio. A number of laboratory tests ever performed have shown that this relationship does not change appreciably depending upon the wave form of irregular loading, type of sand and density of sand. Then, it was possible to establish a reason-

Fig. 20. An average relationship between the factor of safety and the residual pore water pressure ratio

able average curve through the entire set of data that may be used without incurring appreciable loss of accuracy. The curve demonstrated in Fig. 20 shows the relationship thus established.

Having known the factor of safety computed by Eq. (6) it is now possible to estimate the residual pore water pressure ratio on the basis of the relationship presented in Fig. 20.

The factor of safety computed through the above procedures is shown in Figs. 21 and 22 for the deposits of the two Dimbovitza

Fig. 21. Factor of safety against liquefaction for the deposit of the Dimbovitza

Fig. 22. Factor of safety against liquefaction for the deposit of the Dimbovitza

sites. Since the cyclic strength values were estimated based on the two empirical formulae, two curves are presented in these figures. These curves indicate that Eq. (4) gives higher cyclic strength than Eq. (5) does when some fines exist in the sand, but if there is no fines both formulae yield nearly the same value for the cyclic strength. The value of the residual pore water pressure estimated from the relationship of Fig. 20 is

Fig. 23. Estimated residual pore water pressure for the deposit of the Dimbovitza

Fig. 24. Estimated residual pore water pressure for the deposit of the Dimbovitza

presented in Figs. 23 and 24 for the deposits of the two sites being considered. The results of the liquefaction analyses as shown in Figs. 21 through 24 appear to indicate that the liquefaction at site 1 might have taken place at depths approximately between 3.5m and 10m in the sand deposit underlying the surficial clayey silt layer, and the occurrence of liquefaction at site 2 might have been limited to a deeper deposit of gravel-containing sand at depths approximately between 6.5m and 9.5m. For the deposit at site 1, the surface layer composed of clayey silt was considered to be thin enough to have fissures developed due to the excess pore water pressure generated in the underlying sand layer and consequently to have sand ejected and spread over the ground surface as was observed following the 1977 event. In contrast to this, it may well be conceived that the thickness of the unliquefiable surface layer was sufficiently large at site 2 to prevent breakage of the surface layer due to the excess pore water pressure generated in the underlying gravel-containing sand layer. On top of it, the range of depth in the sand deposit through which liquefaction is conceived to have developed is only 3 meter at site 2 as against the corresponding depth range of about 7 m at site 1. Therefore, it may well be speculated that total power of excess pore water pressure was not sufficiently great to break up the surface crust at site 2.

SHINKHOLES IN THE FLOOD PLAIN OF THE DANUBE RIVER

One of the most peculiar feature of the ground damage was large-scale sinkholes which developed in the area of flood plain along the Danube river in the vicinity of the city of Giurgiu. The locations of these sinkholes are shown in Fig. 25. Approximate diameters of the sinkholes are indicated in Table 1.

The sinkholes at sites B, C, and D were discovered a few days after the earthquake and the exact feature of how they developed

Fig. 25. Locations of the sinkholes in the vicinity of the city of Giurgiu

is not known. Site A is located in the low-lying flood plain area covered by sand

Table 1. Size of the sinkholes

Location	Diameter (m)	Site conditions
A	50	Reclaimed land
B	5	Sand bank
C	10 and 15	Forest in the sand bank
D	12	Forest in the flood plain

fills. Nobody was at the site when the earthquake hit the area at 9:22 p.m. on March 4, 1977, but around 1:00 a.m. next morning some persons inspected the site and found a huge dropout in the middle of the vast low-lying land. At that time there was little water inside the sinkhole. At dawn in the next morning the water level was still lower as shown in Photo.3, but by

Photo. 3. A sinkhole at site A several hours after the earthquake

Photo. 4. A water-filled sinkhole at site A about 3 days after the earthquake

Fig. 26. Plan view illustrating the sinkhole at site A

Fig. 27. Side view illustrating the sinkhole at site A (Cross section A-A' in Fig. 26)

noon water had come in up to the level of the ground water table which was equal to the water level of the Danube river about 1.0 km distant from the site A. On the upperleft side of photo.3, water is seen flowing out of the sand fill which is sealed underneath by an impervious soil layer. A picture of the cave-in taken 3 days later from approximately the same location is shown in Photo.4. On the left side of the photo, the trace of a collapse due to the water flow is visible. The situations surrounding the cave-in at site A is roughly illustrated in Figs.26 and 27. Water level of this area was depleted most of time except in rainy season, but because of the implementation of industrial complex planned, placement of saturated sand had been in progress to raise the ground level by a height of 4 m. As illustrated in Fig.27, the land was first enclosed by dikes consisting of friable lumps of loam. The sand was then transported by the hydraulic method. At the time of the earthquake, filling operation had been

almost completed at the place of the open sinkhole. The dropout occurred near the junction of two dikes, swallowing the dike material and the sand behind the dike. Several sand volcanos were observed after the quake here and there in the original unfilled area in front of the dike as shown in Fig.26.

INVESTIGATION OF THE SINKHOLE

Soil Profile

In order to avoid aggravation of the cave-in, the cave at site A was filled up several weeks after the earthquake with the sand used for the land filling around this site. Subsequent to the filling, drilling was carried out at six places to establish soil profiles around the site of the dropout. Using the results of the borings, an approximate picture of the cross section of the soil deposit was drawn as shown in Fig.28. The borings performed in the vicinity of the dropout disclosed the original soil profile consisting of a surface layer of silty clay underlain by an alluvial deposits composed predominantly of sand reaching a depth of about 20 m. It was discovered that a limestone formation with cavities exists under the alluvium. At the place where the land caved in and was filled up afterwards, the sand blended with the reddish clay existed down to the depth of about 20 m where the limestone is encountered. Upon going down

Fig. 28. Inferred configuration of a sinkhole

Fig. 29. Standard penetration test results of the flood plain deposit of the Danube river near Giurgiu, Romania

deep into the limestone, the same sand was found to exist in the cavities, as indicated in Fig.28. On the basis of the soil investigation as above, an approximate configuration of the sinkhole was established as illustrated by dashed lines in Fig.28, where it may be seen that at the periphery of the cave-in, the steep slope sloughed in until it stabilized with the horizontal on the roof of the limestone formation.

Along with the drilling operation, the Standard penetration tests were conducted. The blow count values obtained in the tests are shown in Fig.29. It may be seen that in the hydraulically placed sand fill, the blow count values range mostly between 15 and 20 and somewhat wider variation exists in the blow count value at the deeper layer of alluvial origin. The surprisingly small values of the blow count encountered in one of the bore holes between the depth of 15 and 20 m may be construed as resulting from local flow-out of the sand into the underlying limestone cavities.

Pumping Tests

In order to elucidate the mechanism of the dropout, it was considered necessary to know the phreatic level of the ground water existing in each layer of the deposit. Therefore, in the more comprehensive investiga-

Fig. 30. Locations of the pumping tests near the sinkhole south of Giurgiu

Fig. 31. Piezometric heads in different strata prior to and during the pumping tests

tion scheme implemented in the 1980-1981 period, ten boreholes in all were drilled from the surface of the sand fill at three locations within 500 m from the place of the sinkhole and a series of pumping tests was conducted. The three locations referred to as Group 2, Group 3 and Group 6 are indicated in Fig.30. Each group consisted of three to four boreholes drilled to different depths to reach the elevations of key deposits being investigated. The depths of these boreholes and the soil profiles obtained therefrom are shown in Fig.31. As indicated in Fig.31, casings were used for the boreholes reaching the elevation lower than the clayey silt layer (original surface layer before the land filling) existing at the depth of about 5 m. Two-inch diameter pipes were penetrated about 50 cm ahead of the bottom of the boreholes to reach the key portion of the deposit and to enable its piezometric level to be measured

in isolation from the piezometric head in the upper or lower deposits.

After installing sets of pipes as described above, a series of pumping tests was carried out in which water was pumped up at a constant rate of discharge from one of the boreholes and changes in water level in the remaining pipes were observed. Pumping was continued for about 15 hours until it was made sure that the water levels in all the pipes became stationary. The elevation of the water level before pumping and after it became stationary is shown in Fig. 31. In the Test 1 conducted using the Group 2 boreholes, water was pumped up from the bottom of the alluvial deposit through the borehole 2B. As indicated in Fig. 31, the piezometric head in the boreholes, 2A and 2L remained essentially unaltered, even when the piezometric level in the borehole 2B was lowered by 1.04 m by pumping. This implies the fact that the water in the surface sand fill and in the limestone formation is isolated from each other and also from the water in the alluvial deposit. It was possible to determine the permeability coefficient, k , of the gravelly sand near the bottom of the alluvium on the basis of the results of the pumping test. The permeability coefficient in the Test 1 was $k=0.027$ cm/s. The soil profile at site Group 3 was a little different in that it contained another silty clay layer around the depth of 10 m. Water was pumped through the borehole 3B from the bottom of the alluvial deposit. While a drop of water level as much as 2.5 m was imposed on the pipe 3B, practically no change was observed in the water level of other two boreholes, 3A and 3L. It was therefore concluded that the water in the surface fill, in the alluvial deposit and in the limestone was isolated and there was no connection in water among these three deposits. The permeability coefficient determined in the Test II for the sand above the limestone layer was $k=0.01$ cm/s. In the case of Test V, water was pumped out of the borehole 6C as indicated in Fig. 31. The pumping test showed no change in water level in

the pipes penetrating into the different strata and, hence, it was found as well that the water table was isolated among the three deposits. The permeability coefficient determined in the Test V for the sand deposit above the limestone layer was $k=0.0046$ cm/s.

Since it was confirmed in the above tests that the ground water is isolated from each other, it is no wonder to see the difference in the piezometric head in the normal condition among the three pipes embedded at the different strata. The piezometric heads in the normal condition can be roughly assessed from the elevation of water in each pipe prior to the pumping test. Fig. 31 shows that the ground water table in the surface fill was 2 to 5 m higher than the water table in the underlying alluvial deposit. This is due to the supply of water from the hydraulic filling. Fig. 31 also indicates that the water table in the alluvial deposit was about 1 to 2 m higher than the piezometric level of the underlying limestone formation. It is known that the depth of the limestone formation becomes shallower across the Danube towards the south and it eventually outcrops on the ground surface in the Bulgarian territory. It is therefore conceived that the piezometric level in the limestone is governed by the supply of water from its outcrop on the Bulgarian side.

MECHANISM OF SINKHOLE DEVELOPMENT

It is well known that rocks composed of calcium and magnesium carbonates termed limestone or dolostone contain a number of pores with different sizes. When slightly acid water circulates in the pores, carbonates are dissolved, and carbon dioxide is formed. A hypothetical carbonic acid thus formed attacks calcium carbonate and enlarges the pores. The individual pores thus enlarged form networks of solution channels, cavities, slots or pits. During these processes, the insoluble constituents are left as residual soils and blanket the rock surface. When some

layers of soils overlies the residual soils, they bridge or arch over the intact cavernous limestone (Sower, 1975). In such a geomorphological setting, it is highly possible that the limestone roofing can collapse due to earthquakes and soil ravelling takes place, forming large-scale dropouts on the ground surface. In the case of the sinkholes developed in the area of Giurgiu, it appears most likely that the earthquake triggered the collapse of the limestone roofing over cavities in the rock below and induced the ravelling failure.

The immediate cause of the failure may be somewhat difficult to establish definitively, but because the cavities in the rock were filled with water, it appears not possible for the roof to collapse and fall down as easily as they can do in air-filled cavities. One of the other conceivable causes of the failure is some instability associated with seepage that can be induced in the weathered limestone composing the roof of the caves. It may well be speculated that, as a result of pore water pressure build-up due to the shaking of the earthquake, the piezometric head in the alluvial sand deposit rose well over the piezometric head in the underlying limestone and a state of critical hydraulic gradient was reached to trigger a type of piping failure. The piping developed through the weathered portion of the limestone roof and transferred the sand of the alluvial deposit down into the cavities. The mechanism of the dropout as envisaged above may be analyzed somewhat in detail as follows.

Pore Water Pressure Build-Up in the Sand Deposit

Following the Vrancea earthquake, signs of liquefaction such as sand boils were observed in the vicinity of the site A. Therefore, it appears likely that liquefaction did develop in the surface sand fill. It is also highly probable that pore water pressures had built up in the underlying sand deposit. As shown in Fig. 29, the blow count value of the standard penetration test in this deposit is estimated to have been approximately

on the order of 15 to 20. Using these values of the blow count, together with the fact that no fines are contained in the sand, the cyclic stress ratio causing initial liquefaction or 5% double amplitude strain in 20 cycles is roughly estimated on the basis of the empirical formulae in Eqs. (4) and (5). The estimated cyclic strength of the sand with N -values of 15 to 20 located at the depth of 20 m immediately above the weathered limestone is approximately,

$$\left(\frac{\sigma_{dl}}{2\sigma'_0} \right)_{20} = 0.205 \sim 0.235 \quad (7)$$

Introducing these values into Eq. (3), the cyclic strength in terms of the maximum stress ratio is obtained as follows,

$$\frac{\tau_{\max, l}}{\sigma'_v} = 0.226 \sim 0.259 \quad (8)$$

The magnitude of the acceleration encountered on the ground surface during the 1977 earthquake is somewhat difficult to assess accurately. However, according to the studies by Stojkovic et al (1977) and Balan et al (1982), the town of Giurgiu is located within the area assumed to have been shaken with an intensity of VII in MSK scale. It may well be assumed, therefore, that the maximum horizontal acceleration developed near the town of Giurgiu was probably in the range of about 150 g to 180 g. Incorporating these values into Eq. (1), the maximum stress ratio induced by the earthquake in the alluvial sand deposit at the depth of 20 m immediately above the weathered limestone is estimated approximately as,

$$\frac{\tau_{\max}}{\sigma'_v} = 0.18 \sim 0.22 \quad (9)$$

For all combinations of the two sets of values given by Eqs. (8) and (9), the factor of safety is obtained from Eq. (6) to range as follows,

$$F_s \approx 1.0 \sim 1.45 \quad (10)$$

Incorporating the above factor of safety into the curve in Fig. 20, the developed pore water pressure is estimated to range between $u_r/\sigma'_v = 0.1$ to 1.0. Therefore, the excess pore water pressure developed during the

1977 event in the sand deposit at the depth of 20 m will be assumed to range as,

$$u_r/\sigma_v' = 0.10 \sim 1.0 \quad (11)$$

Changes in Hydraulic Gradient

Owing to the build-up of excess pore water pressure as examined above, the seepage conditions are conceived to have changed in the deposit of the weathered limestone. The feature of such changes is illustrated in Fig. 32 for a representative soil profile established in accordance to the boring logs shown in Fig. 31. In this soil profile, the weathered limestone is assumed to exist between the depths of 20 and 23 m, underlaid by the original limestone formation. It is also assumed that the piezometric levels in the alluvial sand and the limestone were 5 m and 7 m, respectively, from the ground surface in the normal condition prior to the earthquake. On the basis of these hypotheses, the hydraulic gradient, i , in the pre-earthquake condition is estimated to have been $i=0.67$, as indicated in Fig. 32. The values of the hydraulic gradient considered to have existed immediately after the earthquake range between $i=1.1$ and 7.7 corresponding to the pore water pressure rise of $u_r/\sigma_v' = 0.10$ and 1.0 computed in Eq. (11).

It is to be noted that the hydraulic gradient

Fig. 32. Inferred changes in hydraulic gradient in the layer of weathered limestone

Fig. 33. Relationships between the critical hydraulic gradient and permeability coefficient

Fig. 34. Hydraulic gradient conceived to have developed in the weathered limestone layer due to excess pore water pressures during the earthquake

is inversely proportional to the thickness of the layer through which water percolates. In the present problem the thickness of the weathered portion of the limestone is not definitely known. However it may well be assumed that the thickness is probably between 1.0 m and 3.0 m. For the case of the thickness of the weathered limestone layer other than 3.0 m shown in Fig. 32, values of the hydraulic gradient were similarly computed. The computed hydraulic gradient is plotted in Fig. 34 with the residual pore water pressure ratio taken as a parameter. It may be seen in this figure that the value of the hydraulic gradient right after the earthquake in the layer of the weathered limestone was probably in the range of 1.1 and 20.

Seepage Failure

Several modes of failure such as piping, boiling and hydraulic fracturing have been suggested as possible mechanism of failure associated with the instability of seepage of water through soil media. Apart from complete clarification of mechanism, it would be of interest to offer an interpretation to the occurrence of the sinkholes on the basis of an instability condition incorporating the concept of critical hydraulic gradient. According to this instability criterion, failure of water-percolating soils is considered to take place when the developed hydraulic gradient increases in excess of a certain critical value. Several investigations have been carried out to date to define the critical hydraulic gradient for different soils with various grain compositions. Sichart (1957) proposed an empirical formula for the critical hydraulic gradient, i_c , as a function of the permeability, k , as follows,

$$i_c = \frac{1}{1.5\sqrt{k}} \quad (12)$$

where k is expressed in cm/s. Similar studies were conducted by the Chubu Electric Power Co. of Japan to define the instability condition for the piping failure in earthdams. The results of the extensive experimental works are summarized in a formula,

$$i_c = \left(\frac{1.67 \times 10^{-2}}{k} \right)^{0.623} \quad (13)$$

The values of the critical hydraulic gradient as determined by the above two empirical formulae are numerically presented in Fig. 33. It can be seen in this figure that the critical hydraulic gradient increases as the permeability coefficient of soils decreases and that the critical hydraulic gradient estimated from the Sichart's formula is much higher than that determined from the formula of the Chubu Electric Power Co. The large difference between the two formulae appears to reflect the widely variable nature of the critical hydraulic gradient depending upon soil type, grain composition, mode of deposition and so forth.

In order to assess the critical value of hydraulic gradient from the above formulae, it is necessary to define the permeability coefficient of the weathered limestone layer being considered. There is no measured data on this, but it may well be assumed that the weathered limestone containing some percentage of fines probably has a permeability coefficient ranging approximately between 5×10^{-4} and 5×10^{-3} cm/s. The values of the critical hydraulic gradient corresponding to these permeability coefficients, estimated from Sichart formula are 30 and 9, respectively, and indicated in Fig. 34 by hatched zone. Similarly, the formula by the Chubu Electric Power Co. yields the critical values of hydraulic gradient of 9 and 2, corresponding to the permeability coefficients of $k = 5 \times 10^{-4}$ and 5×10^{-3} cm/sec, respectively. These values are also indicated in Fig. 34 by dotted zone.

In Fig. 34, the approximate ranges of the critical value of hydraulic gradient as estimated from the above two empirical formulae are demonstrated, together with the value of hydraulic gradient that is conceived to have developed in the weathered limestone layer 1.0 to 3.0 m thick due to the increase in pore water pressure in the upperlying sand deposit. Although the ranges of variability are quite wide both in the critical hydraulic gradient and in the induced hydraulic gradient, it may be seen that both values lie in the range of the same order of magnitude. Therefore, it may be mentioned that chances must have existed during the 1977 earthquake in which the induced pore water pressure in the upperlying sand deposit increased to a point where the hydraulic gradient in the weathered limestone layer became larger than its critical value, bringing about the seepage instability and consequent dropout of the ground. After the onset of the instability, ravelling of the weathered limestone layer must have probably occurred accompanied by the migration of a large quantity of the sand into the caves in the underlying limestone formation. Since the ground water table in the surface

Fig. 35. Inferred sequence of events leading to the seepage failure in the weathered portion of the limestone formation

fill was located about 3 m higher than the piezometric level of the underlying alluvial sand deposit, the water in the surface fill

flowed into the sinkhole, bringing the water level in the pond eventually up to the elevation of the ground water in the surface fill. The feature of the water flow into the sinkhole is clearly visible on the left side of Photo.3 which was taken several hours after the earthquake. The sequence of events as envisaged above is schematically illustrated in Fig.35.

CONCLUSIONS

For the liquefaction-devastated riverbed deposit of the Dimovitza river in Bucharest, a series of in-situ soil investigations was carried out including borings and penetration tests. On the basis of the results of these tests, simple analyses were made for the pore water pressure build-up in the sand deposits at two sites in this investigated area. For one site where apparent signs of liquefaction were observed during the earthquake, liquefaction was shown to have probably developed in the layer of the alluvial sand at depths between about 3.5m and 10 m. Due to the strong upward thrust of the excess pore water pressures, the surface crust about 3 to 4m thick consisting of silty clay was breached and the sand was ejected on the ground surface through the fissures. For the other site where no surface evidence of liquefaction was visible following the earthquake, the simple analysis showed that liquefaction might have developed in the alluvial sand layer at depths between about 6.5 and 9.5 m. Since the liquefied deposit was located at a greater depth underlain by the thick silty clay surficial layer, it appears likely that no fissure could be developed in the surface crust to allow the sand to come up on the ground surface.

A series of in-situ soil investigation was also conducted in the flood plain deposit of the Danube river near the town of Giurgiu, Romania, where a huge sinkhole developed following the earthquake. The boring logs showed that a limestone formation exists below the depth of about 20 m underneath the alluvial sand deposit. Some convincing

evidences were obtained from the boring logs to indicate that the large sinkhole took place as a result of raveling failure in the karst existing in the limestone formation. The cause of the collapse in the roof of the underground caves was speculated to have been due to the increase in hydraulic gradient which accrued as a result of the pore water pressure buildup in the upperlying alluvial sand deposit during the earthquake. A simple analysis for the pore water pressure buildup indicated that it increased the hydraulic gradient in the weathered roof portion of the limestone formation from a pre-earthquake value of about 0.67 to a value ranging between 1.1 and 7.7. These increased values of the hydraulic gradient were compared against the critical value at which seepage instability can take place. The critical hydraulic gradient estimated from existing empirical formulae was shown to be in a range of the same order of magnitude as the post-earthquake value of the hydraulic gradient as above. Consequently, it is speculated that chances must have existed for the weathered limestone layer to suffer an increase in the hydraulic gradient over the critical value, leading to an seepage instability and hence the catastrophic collapse of the ground during the earthquake.

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