

GEOTECHNICAL IN SITU INVESTIGATION USED FOR SEISMIC DESIGN OF BUILDINGS

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SUMMARY

The present process of harmonization of Romanian seismic codes with European standards requires that the effect of local site conditions be included through the soil factor, S , which must be included in the National Annex of Eurocode 8. The modern codes for earthquake resistant design, including Eurocode 8, classify the soils based on qualitative indicators (description of stratigraphy, soil type, layer thickness etc.) and quantitative indicators (average shear wave velocity, Standard Penetration Test results, etc.). Eurocode 8 specifies that the influence of local ground conditions on the seismic action shall generally be accounted for by considering the five ground types A, B, C, D and E, described by the stratigraphic profiles and parameters. The results of the SPT, down-hole prospecting, surface-wave method and CPT tests from the only one available Romanian site (Bucharest Tei) will be presented. The borehole data and the experimental research performed in the last years revealed a new series of elements regarding the stratification and soil characteristics in Bucharest. The evaluation of the soils liquefaction resistance based on in situ tests and the use of the earthquake records will be presented.

INTRODUCTION

The characterization of local soil conditions using in situ prospecting for establishing the superficial geology and to determine the values of dynamic parameters is an essential element for seismic design of constructions and urban planning. The use of those prospecting techniques represents an important stage in the implementation in Romania of modern codes for seismic design (EC8, EC7, ASCE and UBC). The present paper contains data obtained from geotechnical and seismic investigations at Bucharest Tei, situated in the north-eastern part of Bucharest. Technical University of Construction and National Centre for Seismic Risk Reduction specialists together with the colleagues from Building Research Institute, Japan and Tokyo Soil Research, Japan performed the tests.

The Bucharest metropolitan area is located in the Romanian Plain, along the Colentina and Dambovitza rivers. The main source of earthquakes for Bucharest is the Vrancea seismic zone. Some distinct sedimentary complexes, with distinct peculiarities and a large interval of thickness characterize the geology of the city of Bucharest. Those superficial geological complexes were identified as Quaternary alluvial deposits.

The characterisation of ground conditions from the seismic point of view requires the knowledge of local geology, of the dynamic soil properties, especially of the shear wave velocity that is used by many codes for ground type classification.

The superficial geological complexes intercepted in the borehole executed in 2004 at the Bucharest Tei by using FRASTE Drilling Rig Type Multidrill XL equipment (Figure 1) performed up to a depth of 65.00 meters (Figure 2) are classified (after Liteanu, 1951) from the ground surface to bottom, as follows:

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Figure 1: Drilling equipment (UTCB Tei site)

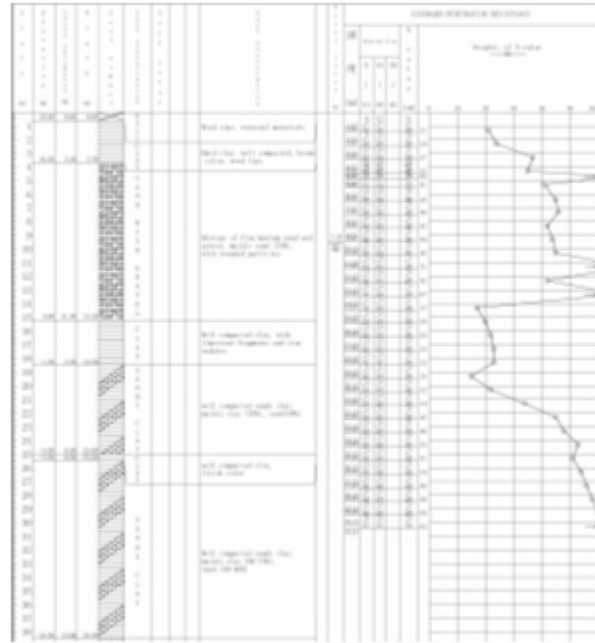


Figure 2: Boring log at Bucharest Tei site

1. Topsoil or backfill complex, between 0.00 to 1.00 meters, containing vegetal soil and heterogeneous materials;
2. Sandy-clay superior deposits, between 1.00 to 3.00 meters, containing Quaternary deposits as clay and sandy clay;
3. „Colentina” gravel complex, between 3.00 to 15.00 meters, containing gravels, medium and fine sand;
4. Intermediate deposits of lacustral origin (predominantly clay and lenses of sand with lacustral origin), between 15.00 to 25.00 meters;
5. “Mostistea” thick bank of sands, between 25.00 to 65.00 meters, containing sand with lenses of sandy clay and silty clay.

IN SITU PROSPECTING METHODS USED AT VARIOUS SITES IN BUCHAREST AREA

Standard Penetration Test (SPT)

The geotechnical method used for soil prospecting in different sites in Bucharest area consists in Standard Penetration Test (SPT). Starting to 2003, NCSR received, as a donation from JICA, drilling equipment FRASTE Drilling Rig Type Multidril XL, which has as attachments an automatic device used for Standard Penetration Test. The method is easy and possible to apply to different soil types. It is most often used in granular materials but also in other materials when simple in-place bearing strengths are required. SPT is also the only soil investigation technique that allows collecting disturbed samples. The purpose of the Standard Penetration Test is to identify the soil stratification, the layer thickness in order to estimate the geological and hydrogeological conditions and other engineering properties of soil layers. The relative firmness or consistency of cohesive soils or density of cohesionless soils can be estimated from the blow count data.

The SPT is a combined sampling and in-situ test technique applied within a borehole, as shows in Fig.1. The test method provides a soil sample for identification purposes and for laboratory tests that allow the use of disturbed samples. In addition, there are many geotechnical correlations, which relate SPT blow count, or N-value, and geotechnical behavior. The N-value becomes a guideline of hardness and softness of soil (the foundation material). Many local correlations as well as widely published correlations, which relate SPT blow count and the engineering behavior of earthworks and foundations are available.

The resistance to penetration is obtained by counting the number of blows required to drive a steel tube of specified dimensions into the subsoil to a specified distance using a hammer of a specified weight (mass). The results of the SPT measurements are quantified in the number of blows required to affect that segment of penetration, N_{SPT} . The SPT equipment, SPT split-barrel sampler and the drill rods used for soil penetration test are shown in Figure 3, and the soil sampling is presented in Figure 4.



Figure 3: Standard Penetration equipment



Figure 4: Soil sampling using SPT

In many countries, Standard Penetration Test remains the subsurface investigation technique of choice for geotechnical engineers. The test is well established in practice, provides a soil sample, and a vast amount of local experience and correlation data have been collected by researchers. The testing procedure varies in different parts of the world. Therefore, standardization of SPT was essential in order to facilitate the comparison of results from different investigations. Figure 5 present distinct items specifications included in the SPT standards of several countries. SPT standards used in different countries are as follows: Japan – Japanese Industrial Standard (JIS A 1219-2001), United States of America (USA) – American Society for Testing and Materials (ASTM, D 1586-2000), United Kingdom (UK) – British Standard (BS 5930:1981 – Code of Practice for Site Investigations), European Standard –ISO 22476-3:2005 „Standard Penetration Test”, EUROCODE 7 (ENV), Part 3 „Design assisted by field testing”, 1997, revised in 1999, Romania – SR EN ISO 22476-3:2006 “Geotechnical investigation and testing – Field testing – Part 3: Standard Penetration Test, International Reference and Technical Papers on Penetration Testing, J. De Rutier – „Penetration Testing”, 1988.

Main items of specifications	JAPAN	U.S.A.	U.K.	EC7	ROMANIA	AUSTRALIA	CANADA	GREECE	MEXICO	PORTUGAL	POLAND	INDIA
Scope - hammer weight (63,5 kg) - falling distance (760 mm)	✓ ✓	✓ ✓	✓ ✓	✓ ✓	✓ ✓	✓ ✓	✓ ✓	✓ ✓	64 750	64 750	65 750	65 750
Boring restrictions - clean hole capability - hole diameter restriction (mm)	✓ 65-150	✓ 57-152	✓ <150	✓ <150	✓ min. 75	✓ not specified	✓ 57-152	✓ min. 75	✓ 57-152	✓ not specified	✓ not specified	✓ 55-150
Sampler assembly (length:mm) - extern diameter shoe 51,1±1 mm - intern diameter shoe 35±1 mm - sampler diameter 457 mm - solid cone	✓ ✓ 535 no	✓ ✓ 533 no	50 ✓ 533 yes optional (for GBWR)	✓ min. 525 no	✓ 533 no	✓ min. 533 yes	✓ min. 533 no	✓ min. 483 -	✓ -	no details	no details	50,8 ✓ 675 no
Drive rods - rod weight (4,33-10,03 kg) - rod diameter (40,5-60 mm)	✓ 40,5	✓ ✓	✓ ✓	✓ ✓	✓ ✓	✓ ✓	✓ ✓	✓ ✓	✓ ✓	✓ 42-51	✓ ✓	✓ 55-150
Executing test - maximum blows	50 during the test	100 (50/50 mm)	50 during the test	50 during the test	50 during the test	60 during the test	100 (50/50 mm)	50 during the test	50/100 mm	100 (50/50 mm)	-	20/250 mm
Usage and practice - materials range S=sandy and silt C=clay G=gravel WR=weak rock	S, C & G	S, C & G	S, C, G & WR	S, C & G	S, C & G	S, C, G & WR	glacial III	S, C	-	S, C & G drilling mud in loose sand	-	S, C

Figure 5: Standard Penetration Test standards in different countries

Cone Penetration Test (CPT)

The Cone Penetrometer Technology (CPT) provides cost-effective, real-time data for use in the characterization of the subsurface. The cone penetrometer consists of a steel cone that is hydraulically pushed into the ground while in situ measurements are continuously collected and transported to the surface for data interpretation and visualization. The cone penetration test is used for cohesive and cohesionless soils, especially for sandy soils and the maximum depth of the static penetration can be established between 25.00 to 30.00 meters. The purposes of the cone penetration test is to evaluate the soil type, geological and hydrogeological conditions,

soil stratification, layers limits, thickness and inclination of the soil layers in the lithological profile, soil density and in situ stress condition, shear strength parameters. During the static penetration, the cone resistance (q_c) and the friction sleeve (f_s) are measured.

In July 2004, the CPT equipment from *Geomil, Holland*, was received as donation from JICA. The Cone Penetration Test equipment from Geomil and the electrical piezocone are presented in Figure 6 and Figure 7.



Figure 6: CPT equipment



Figure 7: Electric piezocone

As you can see from Figure 8, many uncertainties are related with the set up of the test done according with Romanian standards. For research and also for academic community around the world those missing information make impossible correlations with the test results done by others.

Main items of specifications	JAPAN	U.S.A.	U.K.	EC7	ROMANIA	GERMANY	FRANCE	SWEDEN	NORWAY
Cone diameter (35,7±0,3 mm)	✓ no tolerance	≥35,7 ≤36,0	≥35,7 ≤36,0	✓	36,5	≥35,7 ≤36,0	≥34,8 ≤36,0	≥34,8 ≤36,0	≥34,8 ≤36,0
Thrust machine 100 – 200 kN 50 – 100 kN for soft soils	no details	✓	✓	✓	no details	no details	no details	no details	no details
Apex angle of the cone (60°)	✓	✓	✓	✓	✓	✓	✓	✓	✓
Area of the cone (1000 mm ²)	✓	✓	✓	✓	✓	✓	✓	✓	✓
Sleeve area (15.000 mm ²)	no details	✓	✓	no details	no details	no details	14.995	✓	✓
Rate of penetration (20±5 mm)	10-20	✓	✓	✓	✓	✓	piezocone 20±2 mm	piezocone 20±2 mm	✓
Interval of readings (10 – 50 mm)	≤10	>50	≥200	no details	no details	20-100	≤200	piezocone 10-20	20

Figure 8: Cone Penetration Test standards in different countries

In situ P and S-wave measurements

Probably the only direct method to know the elastic properties in depth is borehole logging using elastic waves. A relatively new technique, the suspension PS logging method, is used. Field results of P and S wave velocity measurements using this down-hole method are discussed for the present site investigation. The equipment used for the measurement of compression (P) and shear wave (S) velocities versus depth is a donation from Japan International Cooperation Agency (JICA).

The dynamic response of a site depends strongly on the dynamic properties of the soil. Researches into the dynamic characteristics of soil have been carried out using laboratory soil testing and in situ tests, including geophysical methods. The down-hole technique has become indispensable for determining values of dynamic parameters and Poisson's ratio of soils at relatively small strain levels. Low-strain tests operate below the strain specified above and are based on the theory of wave propagation in the materials.

One of the low-strain field tests is PS Logging, a seismic down-hole technique. In the down-hole method the sensors are placed at various depths in the boring and the source of energy is above the sensors - usually at the surface. This technique does not require as many borings as the cross-hole method, but the waves travel through several layers from the source to the sensors. Thus, the measured travel time reflects the cumulative travel through layers with different wave velocities, and interpreting the data requires sorting out the contribution of the layers. Since S and P wave velocities are calculated from the slope of a depth/travel time curve, the velocities are obtained not for each incremental interval but for a velocity layer that has a certain thickness including many measuring points as an average values. The P-wave is generated by hitting a wooden pile with a large wooden hammer (as shown in Figure 9), and S-wave is generated by hitting the end of a plank horizontally with the same hammer (as shown in Figure 10).



Figure 9: Generation of P-waves



Figure 10: Generation of S-waves

Direct measurement for soil or rock stiffness in the field has the advantage of minimal material disturbance. The modulus is measured where the soil exists. Furthermore, the measurements are not constrained by the size of a sample. Moduli measured in the field correspond to very small strains. Although some procedures for measuring moduli at large strain have been proposed, none has been found fully satisfactory by the geotechnical engineering community. Since the dissipation of energy during strain, which is called material damping, requires significant strains to occur, field techniques have also failed to prove effective in measuring material damping. In situ techniques are based on measurement of the velocity of propagation of stress waves through the soil.

An earthquake source produces two types of waves: body and surface waves. The body waves, which travel through the body of the soil or rock, can be divided into P-waves and S-waves. In P-waves the particle motion is in the direction of propagation of the wave. In S-waves the particle motion is perpendicular to the direction of propagation. Surface waves occur at the surface of a material or at the interface between materials of different properties. A typical earthquake accelerogram contains three main groups of waves: P, or primary; S, or secondary; L, or surface waves. The fastest moving of the waves are the P-waves (P for primary). In these waves the particles move back and forth in the direction of propagation, so the wave consists of alternate compression and extension. In a medium of mass density ρ with shear modulus G and Poisson's ratio ν , the velocities of propagation is $V_p = \frac{2G(1-\nu)}{\rho(1-2\nu)}$ and $V_s = \sqrt{\frac{G}{\rho}}$. The ratio of V_p to V_s is $\frac{V_p}{V_s} = \sqrt{\frac{2(1-\nu)}{1-2\nu}}$ which is always greater than 1.0 for realistic values of Poisson's ratio.

Because the P-waves or compressional waves are dominated by the response of the pore fluid in saturated soils, most techniques measure the S-waves or shear waves. If the velocity of the shear wave through a soil deposit is determined to be V_s , the shear modulus G is $G_0 = \rho \cdot V_s^2 = \frac{\gamma_t \cdot V_s^2}{98}$, where V_s - shear wave velocity (m/s²), G_0 - kg/cm², γ_t - unit weight of the soil (t/m³), ρ - mass density of the soil.

Poisson's ratio ν is given by $\nu = -\frac{\epsilon_3}{\epsilon_1}$, where ϵ_3 is transverse strain and ϵ_1 is axial strain

In most of borehole loggings, only the initial phase's data, especially the travel-time data, are used to obtain the velocity profiles. The analysis of travel-time data coordinated with the site stratigraphy revealed the seismic velocity profiles and other related parameters as Young's modulus (E_{din}), shear modulus (G_{din}) and Poisson's ratio (ν_{din}).

The graphic of P and S-wave velocities are presented in Figure 11, and the corresponding measured values of Young's modulus (E_{din}), shear modulus (G_{din}) and Poisson's ratio (ν_{din}) are presented in Figure 12, respectively in Figure 13.

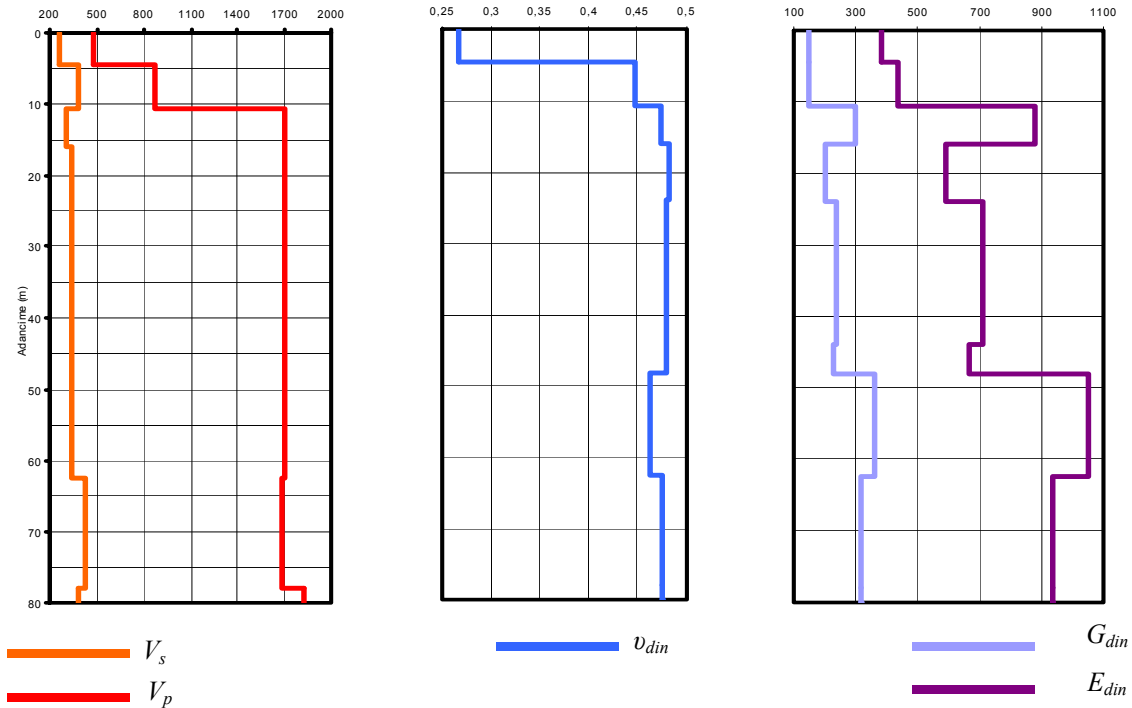


Figure 11: P and S-wave velocities (m/s)

Figure 12: Poisson's ratio values (ν_{din})

Figure 13: Young's modulus (E_{din}), shear modulus (G_{din}) (MN/m^2 , MPa)

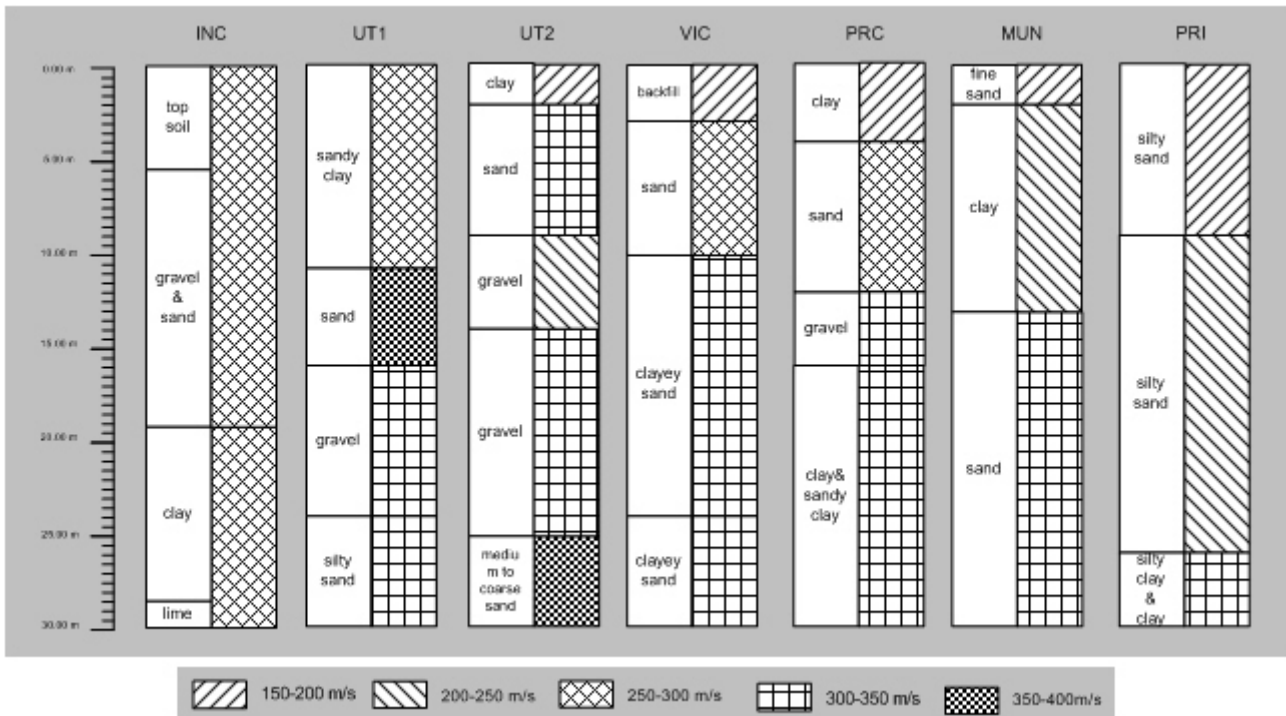


Figure 14: Results of in situ S-wave measurements at various sites in Bucharest area

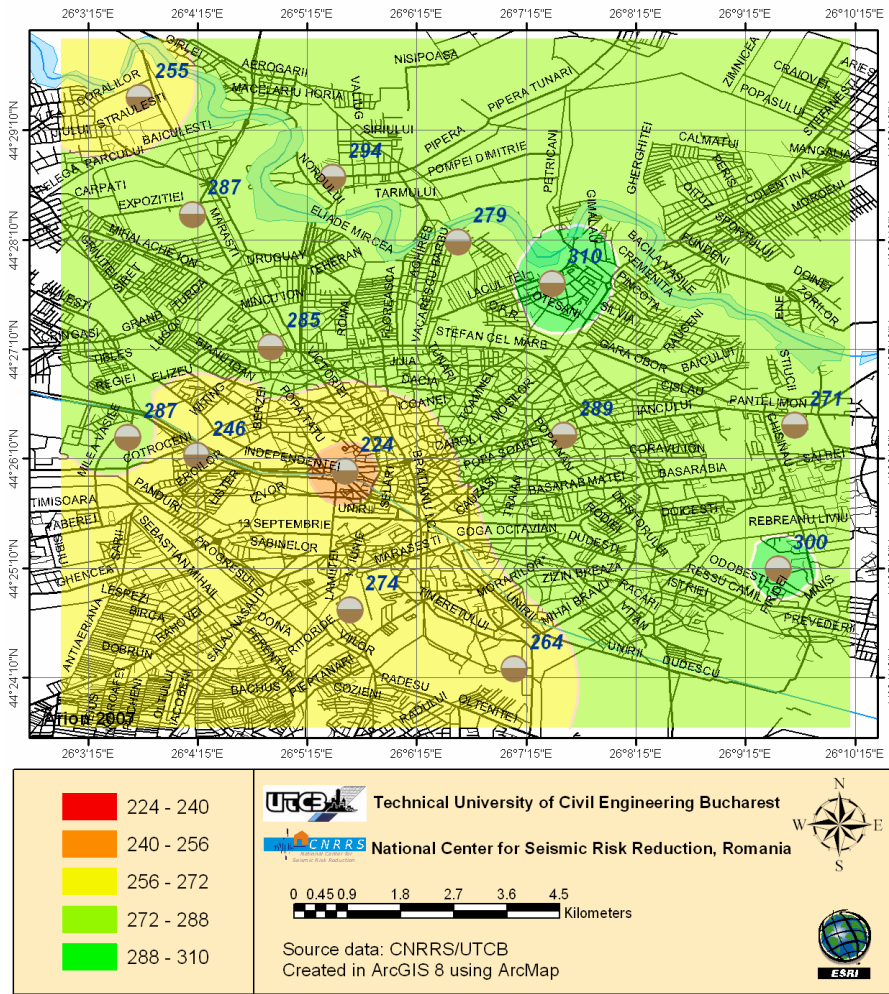


Figure 15: Bucharest. Microzonation map for shear wave velocity (m/s) averaged on 30m

Surface wave method

The surface-wave method can be carried out from ground surface nondestructively. The shear-wave velocity can be obtained by the surface-wave method. Surface-wave (Rayleigh wave) is elastic waves propagating along the ground surface and its energy concentrates near the ground surface. The surface-wave velocity of propagation strongly depends on S-wave velocity. If subsurface S-velocity vary with the depth, propagating velocity vary with its frequency or its wavelength. This character is called dispersion. Sub-surface S-wave velocity structure can be estimated by analysis of dispersion of the surface-waves. The surface wave method is the seismic exploration method in which the dispersion character of the surface-waves is analyzed.

Figure 16 shows the schematic view of a surface-wave method. Data acquisition and analysis for the surfaces is based on the MASW (Multichannel Analysis of Surface Waves) proposed by Park et al.(1999a, 1999b) and

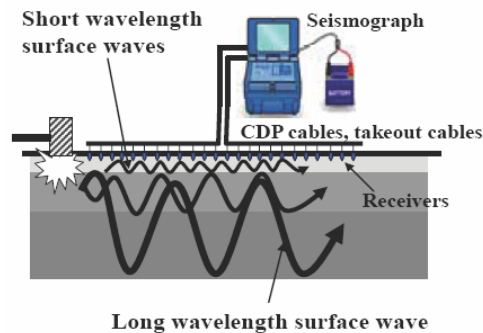


Figure 16: Schematic diagram of a surface-wave method

CMPPC (Common Mid Point Cross-correlation) analysis proposed by Hayashi and Suzuki (2004). A 10kg sledgehammer or 50kg weight drops are used as a source. The sources are placed with 1 to 4m intervals. 12 to 48 geophones (4.5Hz) are deployed with 0.5 to 2m intervals.

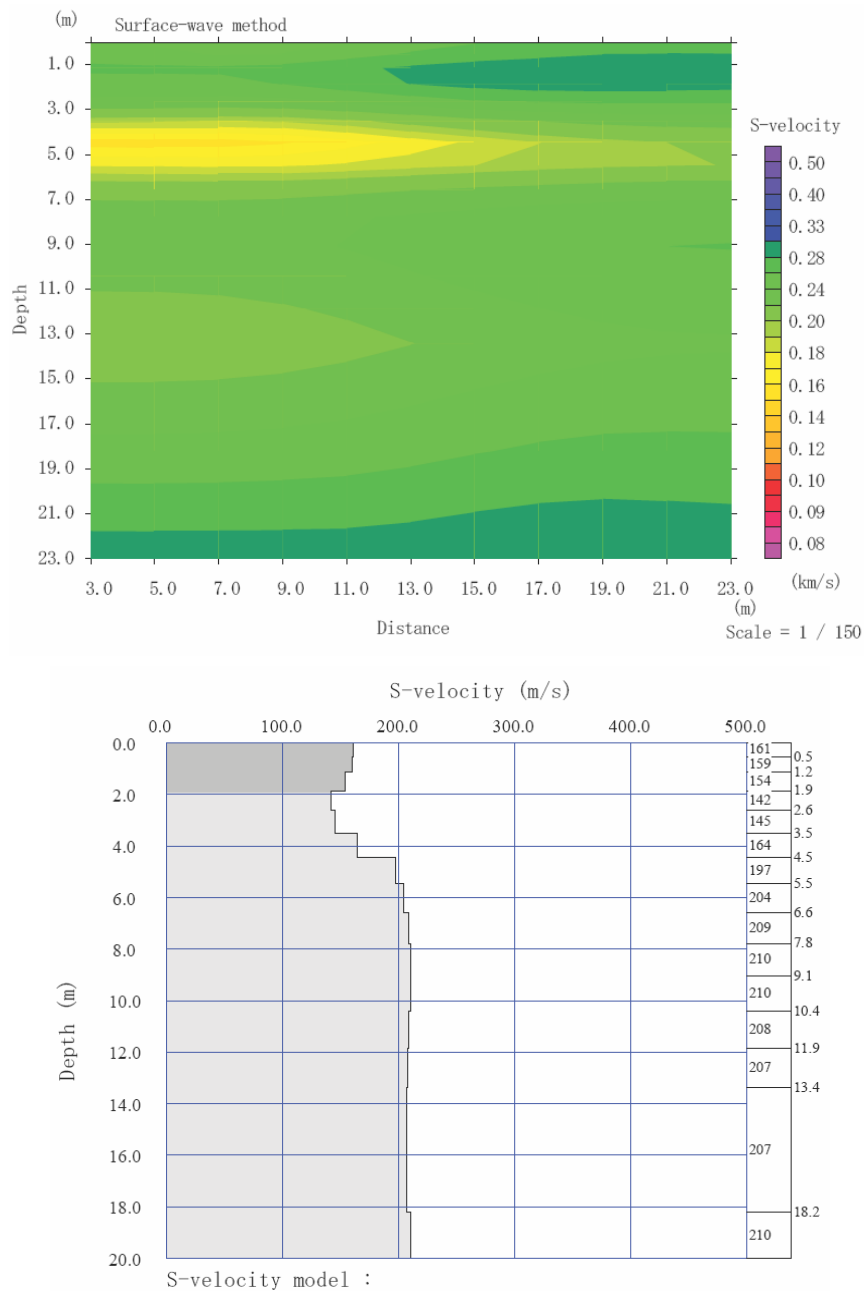


Figure 17: Shear-wave velocity models for the UTCB site obtained through the surface-wave method

EVALUATION OF SOIL LIQUEFACTION RESISTANCE USING FIELD DATA

The recent developments in the numerical analyses for the nonlinear dynamic responses of grounds due to strong earthquake motions have increased the demand for the dynamic soil properties corresponding to large strain level.

The most common cause of ground failure during earthquake is the liquefaction phenomenon that has produced severe damage all over the world. The liquefaction of sandy soils and sands with non-plastic fines as a result of earthquake ground shaking poses a major threat to the safety of civil engineering structures. The problem of liquefaction is addressed in EC8, Part 5, § 4.1.3 "Potentially liquefiable soils" as well in the Annex B "Empirical Charts for simplified liquefaction analysis". The current state-of-practice is described by Youd et al. (2001).

Liquefaction resistance: empirical methods based on in situ penetration resistance

At higher range of shear strains, the behavior of soils is elastic-plastic and produce irrecoverable permanent to measure high-strain. Standard Penetration Test and Cone Penetration Test are of particular importance to measure high-strain characteristics of soil.

Many factors govern the liquefaction process for in situ soils and the most important are intensity of earthquake and its duration, location of ground water table, soil type, soil relative density, particle size gradation, particle shape, depositional environment of soil, soil drainage conditions, historical environment of the soil deposit and building additional loads of these deposits. Liquefaction susceptibility is usually expressed in terms of a factor of safety against its occurrence. This factor is defined as the ratio between available soil resistance to liquefaction, expressed in terms of the cyclic stresses required to induce liquefaction, and the cyclic stresses generated by the design earthquake.

Evaluation of soil liquefaction resistance using SPT results

The liquefaction resistance of soil deposits and prediction of the liquefied thickness is based on Standard Penetration Test blow counts in a single boring log. It is widely accepted that only the recent sediments or fills of saturated, cohesionless soils at shallow depth (< 20 meters) will liquefy in a large magnitude earthquake ($M_w > 7$). Starting in the 1970's, H.B. Seed and his colleagues worked to develop a reliable method for assessing the liquefaction potential based on SPT data. Their framework for SPT-based assessments of liquefaction potential was developed in a series of papers that includes Seed and Idriss (1971), Seed et al. (1977), Seed (1979), Seed and Idriss (1981, 1982), Seed et al. (1983), significant contributions were also suggested in the work of Tokimatsu and Yoshimi (1983), Seed et al. (1985).

The empirical method in evaluating the soil liquefaction resistance from Standard Penetration Test blow counts is based on corrected values of $(N_1)_{60}$ and cyclic resistance ratio, CSR.

The first step in evaluating the soil liquefaction resistance is to correct the measured SPT blow counts N_{SPT} . The measured SPT blow counts is first normalized for the overburden stress at the depth of the test and corrected to a standardized value of $(N_1)_{60}$.

$$(N_1)_{60} = N_{SPT} \cdot C_N \cdot C_E \cdot C_B \cdot C_S \cdot C_R \quad (1)$$

where: N_{SPT} represents the blow counts necessary for 30 cm soil penetration; C_N , C_E , C_B , C_S , C_R are the correction factors

The next step in the liquefaction analysis procedure is to find the cyclic resistance ratio (CRR) for the soil based on the computed clean-sand equivalent ($(N_1)_{60cs}$). This is done using the empirical base curve drawn from the liquefaction catalogue for a magnitude 7.5 earthquake. The mathematical expression implemented for determining the cyclic resistance ratio for soil is:

$$100 \cdot CRR_{M=7.5} = \frac{95}{34 - (N_1)_{60cs}} + \frac{(N_1)_{60cs}}{1.3} - \frac{1}{2} \quad (2)$$

where: CRR is the cyclic resistance ratio for a $M=7.5$ earthquake; $(N_1)_{60cs}$ represents the clean-sand equivalent SPT value.

The value of $CRR_{M=7.5}$ must be adjusted for the magnitude of the earthquake under consideration. This is done with a magnitude scaling factor, MSF:

$$CRR = CRR_{M=7.5} \cdot MSF \quad (3)$$

where: CRR is the cyclic resistance ratio of the soil for an earthquake magnitude corresponding to MSF, which can be considered by assuming that the main effect of different magnitude earthquakes on liquefaction resistance is the number of significant stress cycles generated (Seed et al., 1983).

The magnitude scaling factors is computed using the following equations:

- for $M_w < 7.0$: $MSF = 10^{3.00} \times M_w^{-3.46}$
- for $M_w > 7.0$: $MSF = 10^{2.24} \times M_w^{-2.56}$

where: M_w is the moment magnitude of the earthquake.

In case of present soil liquefaction analysis procedure, we use the NCSRR earthquake records from 2004, 27 October, magnitude $M_w=6.0$ (EMSC – European-Mediterranean Seismological Centre). The seismic records are presented in Figure 18.

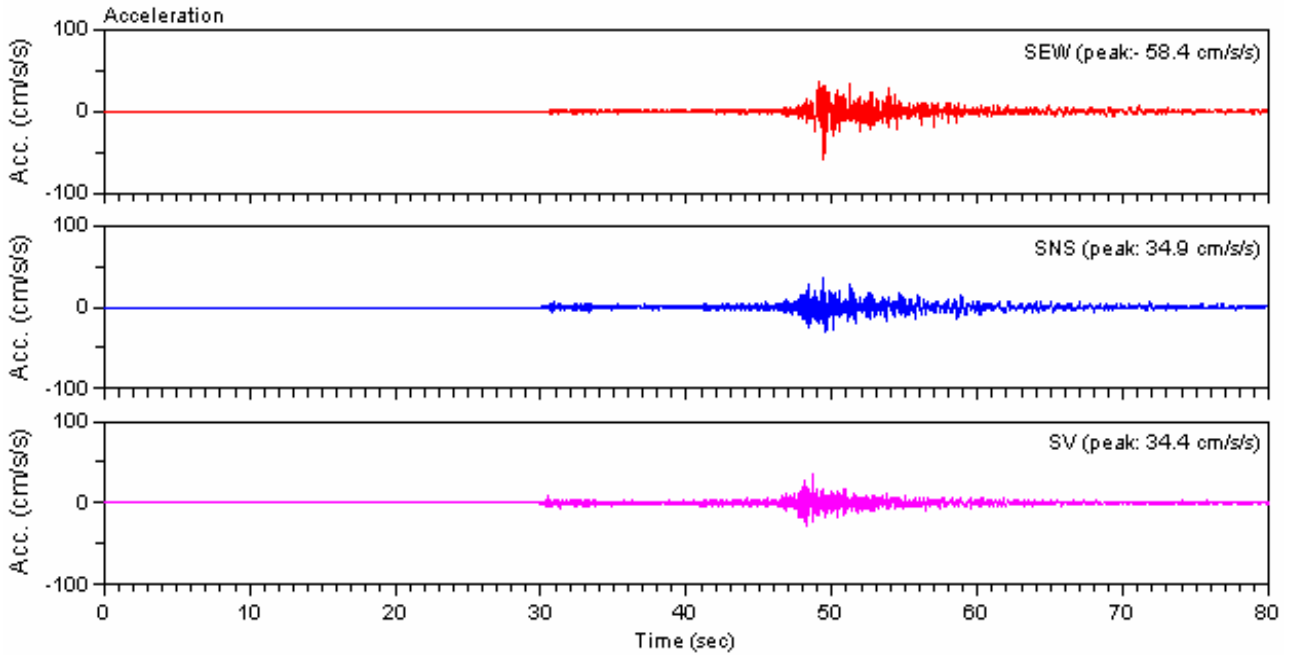


Figure 18: Seismic records (27.10.2004), magnitude $M_w=6.0$ (EMSC)

Once the liquefaction resistance is known at a certain depth, the average cyclic shear stress generated by an earthquake must be estimated. The representative horizontal shear stress is computed with a simplified equation suggested by Seed et al. (1983, 1985) and expressed in terms of the cyclic stress ratio (CSR):

$$CSR = 0.65 \cdot \frac{a_{max}}{g} \cdot \frac{\sigma_{vo}}{\sigma'_{vo}} \cdot r_d \quad (4)$$

where: $g=9.81\text{m/s}^2$ is the acceleration due to gravity, σ_{vo} is the total vertical overburden stress, σ'_{vo} is the effective vertical overburden stress at the depth of interest, a_{max} is the maximum horizontal acceleration that would occur at the ground surface in the absence of excess pore pressures or liquefaction generated by the earthquake. The last parameter is the stress reduction factor, r_d , which accounts for soil flexibility as a function of depth. Several researchers have used simple linear equations to approximate these average r_d values, Seed and Idriss (1971), Tokimatsu and Yoshimi (1983), Liao and Whitman (1986) and Kayen et al. (1992).

The last step in the liquefaction analysis is to compute the factor of safety at each *SPT* location and the liquefied thickness. If the computed cyclic resistance ratio (*CRR*) of the soil is less than or equal to cyclic stress ratio (*CSR*) generated by an earthquake, liquefaction is assumed to occur at that location. The factor of safety against liquefaction, FS_{liq} , is defined with (Ishihara 1985, 1993; Seed and Hardner, 1990):

$$FS_{liq} = \frac{CRR}{CSR} \quad (5)$$

- $FS_{liq} \leq 1.0$ indicates that the soil at the depth of the measured *SPT* blow counts is predicted to liquefy
- $FS_{liq} > 1.0$ indicates no liquefaction

Evaluation of soil liquefaction resistance using shear wave velocities results

The preferable practice when using V_s measurements to evaluate liquefaction resistance is to drill sufficient boreholes and conduct sufficient tests to detect and delineate thin liquefiable strata, to identify non-liquefiable clay-rich soils, etc.

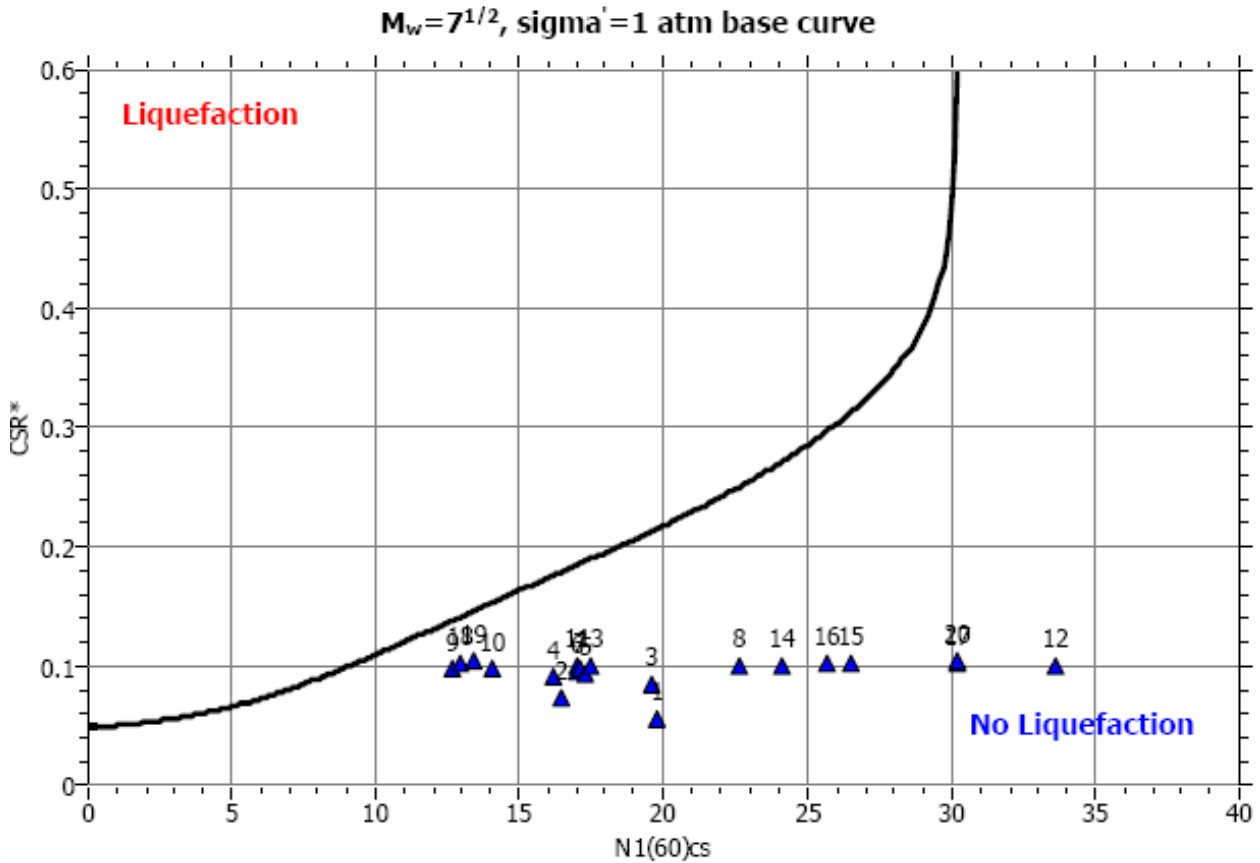


Figure 19: The assessment of soil liquefaction potential from SPT data at UTCB site (applying Eurocode 8 method)

One method of direct determination of dynamic soil properties in the field is to measure the velocity of shear waves in the soil. The waves are generated by impacts produced by a hammer or by detonating charges of explosives, and the travel times are recorded. This is usually done in or between boreholes. The use of V_s as an index of liquefaction resistance is justified since both V_s and liquefaction resistance are influenced by many of the same factors (void ratio, effective confining pressure, stress history, geologic age).

The resistance of the soil, expressed as the cyclic resistance ratio is generally established by separating liquefied cases from non-liquefied cases in actual earthquakes. Here, following empirical equation defined by Andrus et al. (1999) is used:

$$CRR = \left\{ a \left(\frac{V_{s1}}{100} \right)^2 + b \left(\frac{1}{c - V_{s1}} \right) - \frac{1}{c} \right\} \quad (6)$$

where: a, b, c are parameters (a=0.022, b=2.8, c=200~215m/s), V_{s1} is the overburden stress corrected shear wave velocity defined as:

$$V_{s1} = V_s \left(\frac{Pa}{\sigma'_v} \right)^2 \quad (7)$$

where: V_s = measured shear-wave velocity (m/s), Pa =reference stress (100kPa), σ'_v = initial effective overburden stress (kPa).

The parameter c in the equation (6) represents the limiting upper value of V_{s1} for liquefaction occurrence. Generally, in the SPT based simplified method, a corrected blow count (N) of 30 is assumed as the limiting upper value for liquefaction occurrence.

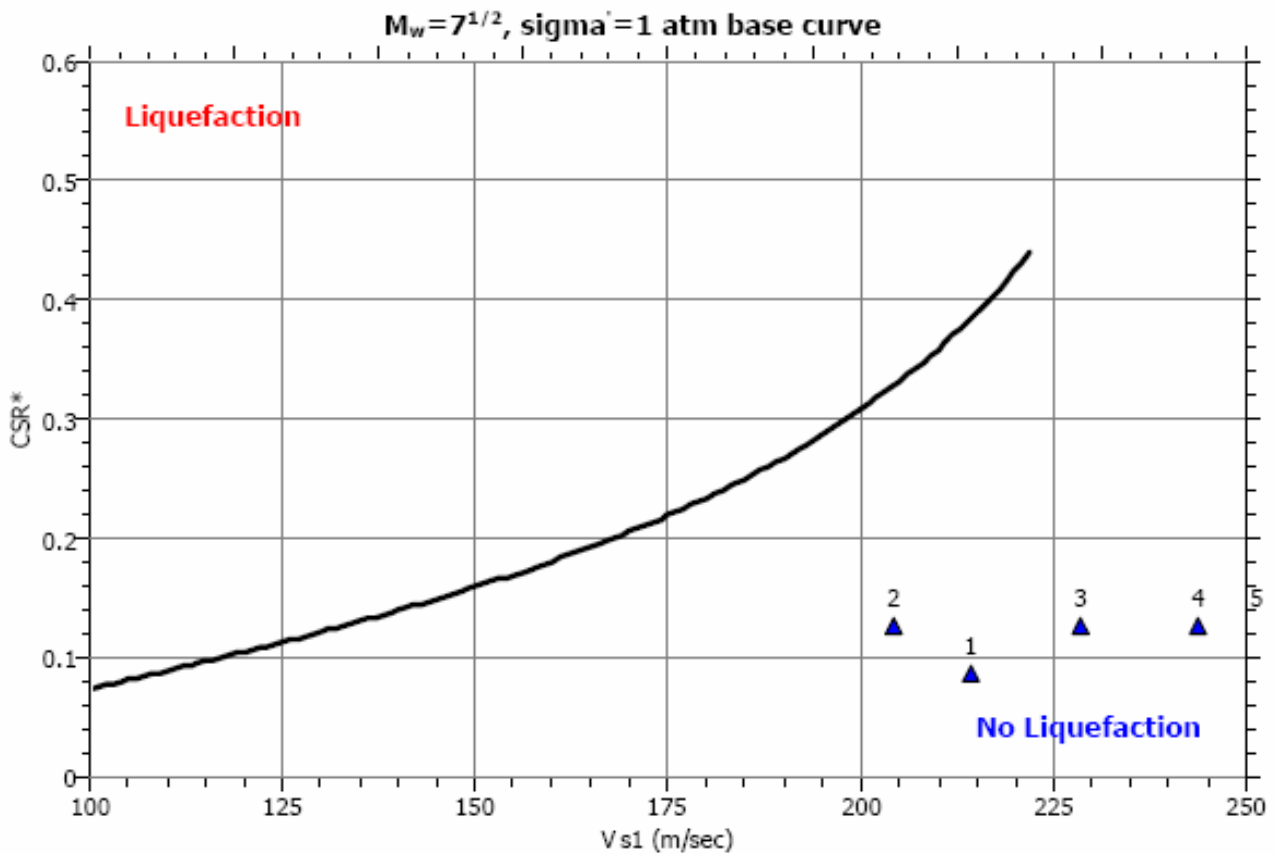


Figure 20: The assessment of soil liquefaction potential from V_s data at Municipal Hospital site (applying Eurocode 8 method)

CONCLUSIONS

The different local soil conditions in Bucharest area lead to a variability of the soil response at seismic ground motions, which can occur at small distances and can be observed even for the same city area. This is the reason why in the case of the large urban areas microzonation studies must take into account the mapping of the soil profile and the ground parameters. We have applied two methods of seismic investigation to evaluate the liquefaction potential of the site.

Creating an informational database concerning the characteristics of superficial geology of Bucharest is the basis for determining the correlation between seismic velocities, SPT and CPT results and soil parameters at seismic motions. The corroboration of resulted information is useful for the characterization of local site conditions from the geological and geotechnical point of view. The database will set basis for the requirements of Romanian seismic codes, harmonized with the European and international codes.

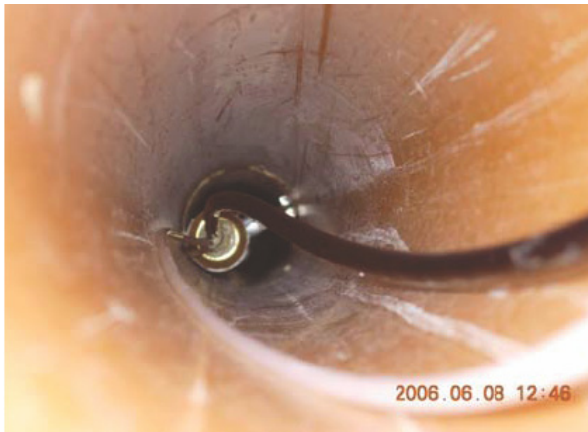
The extended soil investigation by using those equipments will provide valuable data correlated with prior studies concerning seismic data processing, should be analysed in the future in order to establish the microzonation parameters to be used for urban planning and earthquake risk reduction. The microzonation of averaged shear wave velocity for Bucharest shows a relative uniformity of the values in the range between 210m/s and 310m/s.

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Photos during the field investigation of the CNRRS