Implementation of in-situ and geophysical investigation methods (ERT & MASW) with the purpose to determine 2D profile of landslide

***Stjepan Strelec*[1], *Josip Mesec*[1], *Kristijan Grabar*[2] and *Jasmin Jug*[3]**

Influence of region lithostratigraphic composition has a dominant effect on the development and shape of the landslide slide surface. Paper contemplate 2D profile model of a retrogressive landslide along with the implementation of "in situ" and geophysical investigation methods for the landslide instability with the complex slide surface. Purpose of the investigation works is to obtain insight in the ground composition and geotechnical characteristics of the soil on the area of the manifested landslide, identification of the slide surface as well as the creation of geotechnical soil profiles. To determine position and size of the surface of rupture, investigation works have been undertaken involving: core drilling, dynamic probing by the light dynamic probe (DPL), flat dilatometer probing (DMT) and static probing by CPTu probing. Used geophysical methods comprised geoelectrical tomography (ERT) and multichannel seismic analysis of surface waves (MASW). ERT tomography method can be considered appropriate for geometrical landslide outlining, as well as recognising potentially unstable slope areas. From the other side, electrical profiling results need to be calibrated with data obtained from other in-situ investigations. This was accomplished by use of In-situ probes, CPTu and DMT. Obtained 2D profile of the landslide model was additionally arithmetically confirmed by the use of limit equilibrium methods. Investigation results accomplished by the combination of proposed investigation methods have been proved as a good base for the exploration of the complex landslide instabilities.

Keywords: *landslide, slide surface, CPTu, Marchetti dilatometer (DMT), in situ horizontal stress index (K_D), electrical resistivity tomography (ERT), multichannel analysis of surface waves (MASW), limit equilibrium methods.*

1. Introduction

For the purpose of the landslide remediation on the local road LC37069 corridor in the place Samarica, geotechnical field investigations were carried out as well as laboratory sample analysis. Investigation works gave insight into geotechnical soil characteristics of the unstable landslide area, as the base for the landslide remediation design.

When considering landslide geotechnical investigations, the first thing that is considered is the core drilling, which is the most direct technique to obtain subsurface information of a particular site. Drilling is rather expensive and only gives information at a particular point or location. Information between the boreholes are merely based on interpretation and correlation.

Geophysics investigations involve measurements of the physical properties, usually conducted from the Earth's surface, to obtain useful information (2D & 3D interpretation) of the structure and composition of the concealed subsurface through a comprehensive interpretation of the measured geophysical data.

Geophysical methods have been widely and successfully employed for many years, primarily in the search for oil and gas deposits, and are becoming increasingly popular in tackling problems associated with geotechnical, geological engineering and environmental issues. Along with the drilling and implemented geophysical explorations (ERT & MASW), investigation program included newer in-situ probing technique methods to determine the depth of the landslide slip surface, particularly:

- static probing with CPTu probe,
- static probing with DMT flat dilatometer probe - Marchetti type,
- dynamic probing with light probe - DPL (dynamic probing light).

The landslide in the subject was geotechnically identified by core drilling in November 2015 by 6 (six) core investigation boreholes, marked B1 to B6. Drill core was identified according to USCS classification. At the drilling process, disturbed and undisturbed soil samples were taken for the laboratory testing, which is a determination of the physical and mechanical soil properties.

Along with the classic investigation core drilling, other in-situ probing were carried out: CPTu, DMT and DPL. Detailed investigation plan is illustrated in Fig. 1.

To obtain 2D soil profile through the main landslide body, two geophysical methods were implemented: geoelectrical tomography (ERT) and multichannel analysis of surface waves (MASW). 2D electrical tomography

[1] *Stjepan Strelec, Assoc. Prof., PhD Josip Mesec Assoc. Prof., PhD*, University in Zagreb, Faculty of Geotechnical Engineering, Hallerova 7, Varazdin, Croatia, stjepan.strelec@gfv.hr , josip.mesec@gfv.hr
[2] *Kristijan Grabar ,MSc*., SPP d.o.o. Trstenjakova 3, Varazdin, Croatia, spp@spp.hr
[3] *Jasmin Jug, MSc,* University in Zagreb, Faculty of Geotechnical Engineering, Hallerova 7, Varazdin, Croatia, jasmin.jug@gmail.com

gives more detailed subsurface soil model in comparison to vertical electrical profiling (VES) because tomography interpretation accounts for vertical and lateral differences of soil electrical resistivity.

By utilising this, elongated geological structures can be quality distinguished, particular to separate contact between depleted mass body from the more impermeable bottom soil sediments in 2D imaging.

MASW is a seismic geophysical method, which is determined soil stiffness based on measured S wave propagation velocity, and by that quality distinguish border between landslide depleted mass and bedrock firm soil, also by 2D imaging.

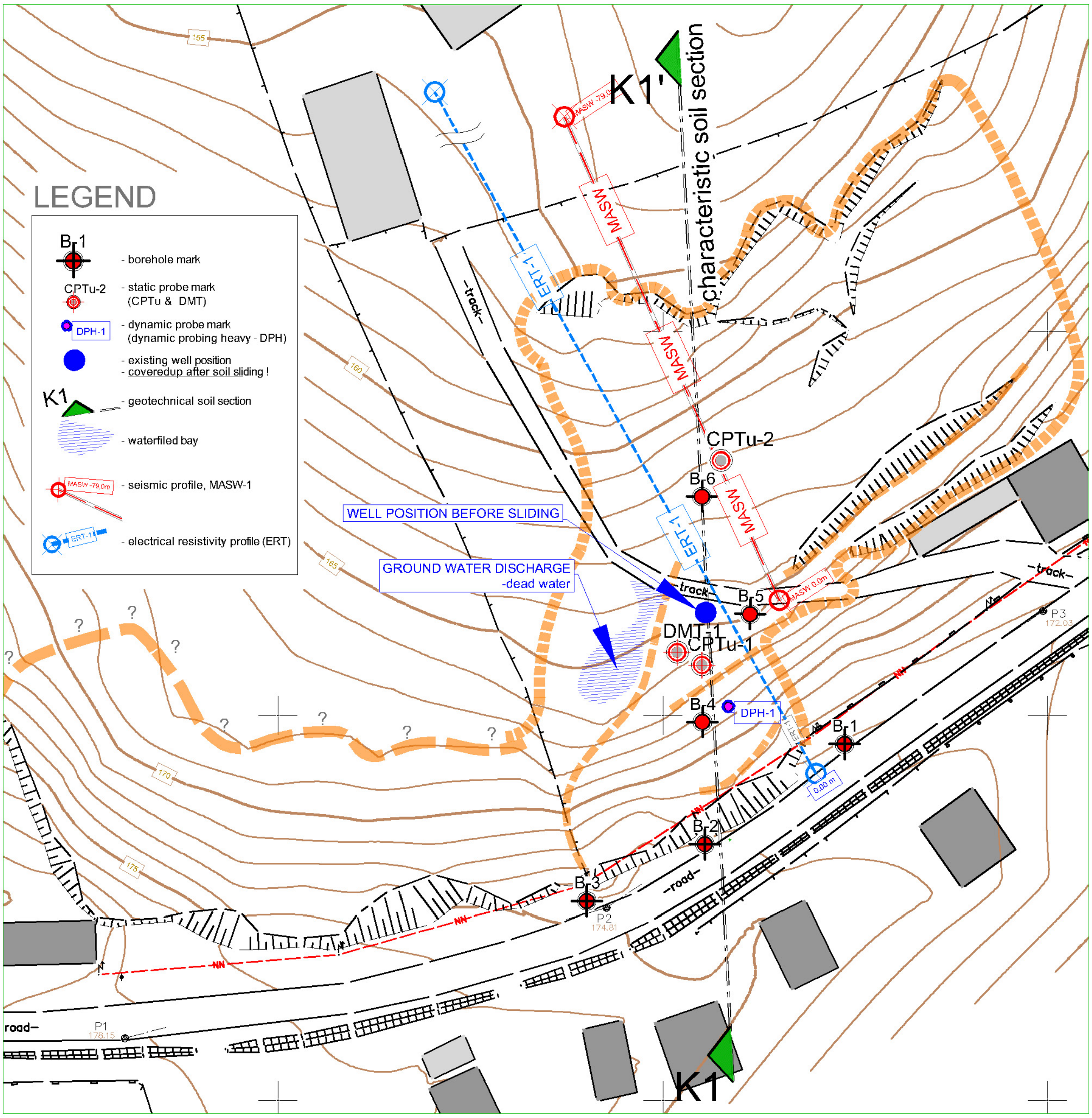


Fig. 1. Detailed plan of geotechnical investigation - "Samarica Landslide ".

2. Geotechnical properties of the ground - case study area

By identifying borehole core, static and dynamic probing results as well laboratory soil sample testing, lithological composition of the ground was determined (Strelec at al., 2016a):

1. Base ground on location consist of low plasticity clay (CL, w_L = 40-50%), of yellow-brown color and firm consistency state, q_c = 110-400 kN/m^2, N = 8-12 blows/foot, c_u = 50-200 kN/m^2, γ = 19,5 kN/m^3, M_v = 7,0 MN/m^2, c = 10 kN/m^2 and ϕ = 28°.
2. Depleted mass of landslide main body (CL, c_u = 10 - 30 kN/m^2, γ = 18,8 kN/m^3).
3. Soil parameters on the slide surface, identified by static probes: c_u = 6 kN/m^2, γ = 18,0 kN/m^3.

Boreholes B1 to B3 were carried out on the crown of the landslide (Fig. 1) using the truck rig boring machine, while the boreholes on the main landslide body because of inaccessible terrain were completed by hand boring. At the main landslide body, static and dynamic probing was completed using portable equipment (CPT, DMT and DPL) shown on photography in Fig. 6. At the drilling process on boreholes B1 to B3 located at the crown of the landslide, no groundwater was identified, while at the borehole B3 only moist zone at depth interval 2 to 5 m was identified. Fig. 2 shows B3 borehole core in which moist interval was identified. Fig. 3 display results of laboratory soil sample testing, plasticity chart for the borehole B3. The correlation between the index of plasticity and angle of internal friction is given in Fig. 4, as developed by various authors, systemized by Ortolan & Mihalinec (1998) and enriched by new carefully obtained data. From these correlations, the angle of internal friction of cohesive materials can be roughly determined, for the plasticity index values determined in the geotechnical laboratory.

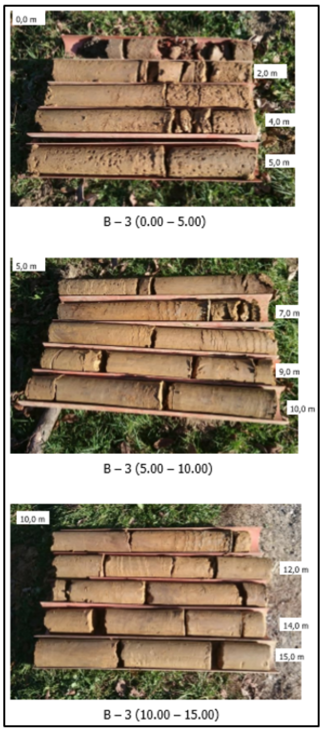
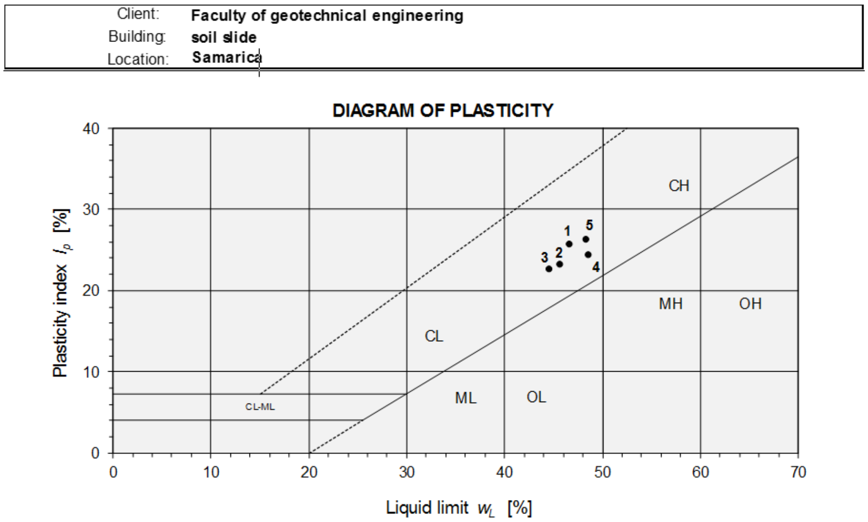


Fig. 2. B3 borehole core.



Position:	Sample description				Liquid limit w_L [%]	Plastic limit w_P [%]	Plasticity index I_P [%]	USCS clasification
	Laboratory sample mark:	Borehole:	Depth (m)	Percent passing sieve 0,425 mm				
1	37-10/15	B - 3	2,30 - 2,50	100%	46.6	20.9	25.7	CL
2	37-11/15	B - 3	4,80 - 5,00	100%	45.7	22.5	23.2	CL
3	37-12/15	B - 3	6,80 - 7,00	100%	44.6	22.0	22.6	CL
4	37-13/15	B - 3	10,40 - 10,60	100%	48.6	24.2	24.4	CL
5	37-14/15	B - 3	132,10 - 13,30	100%	48.3	22.0	26.3	CL

Fig. 3. Diagram of plasticity for the samples from the B3 borehole.

2.1. Lithostratigraphic structure

Location base ground consists of Pleistocene sediments (*lg*), represented by clayish silts with the thin layers of organic compounds. Base sediment from the geotechnical aspect consists mainly of low plasticity clay (CL), with liquid limit w_L = 40 – 50 %. Intermediate transitional interval towards the bottom mottled clays identified deeper than 7,0 m is gradual, without the strict sedimentation border. Clay contains little to middle of dispersed organic substance (black stains). On the B3 borehole core within the 2,0 to 5,50 m interval, extremely porous sediment structure was discovered, which proves geological association to the group of clayish silts. Unstable slope structure in the case study is a landslide in a uniform soil body.

2.2. Main reasons for the slope instability

The main reason for recent retrogressive development along the road corridor is a yielding of slide surface the caused by water buildup and high levels of seepage waters. The mentioned process is active in the investigated area for a long time, which resulted in landslide movements for a long period of years and formed surfaces of rupture to be predisposed to recent landslide movements. It is concluded that slope structure in combination with a lithostratigraphic composition of the ground, along with the hydrogeological conditions in soil with additional quantities of rainfall waters from the road, are the main reasons for the development of landslide on the affected road section. Considered through landslide instability development dynamic, restricted part of the landslide is characterised as retrogressive and depicted in Fig. 5, marked as "B".

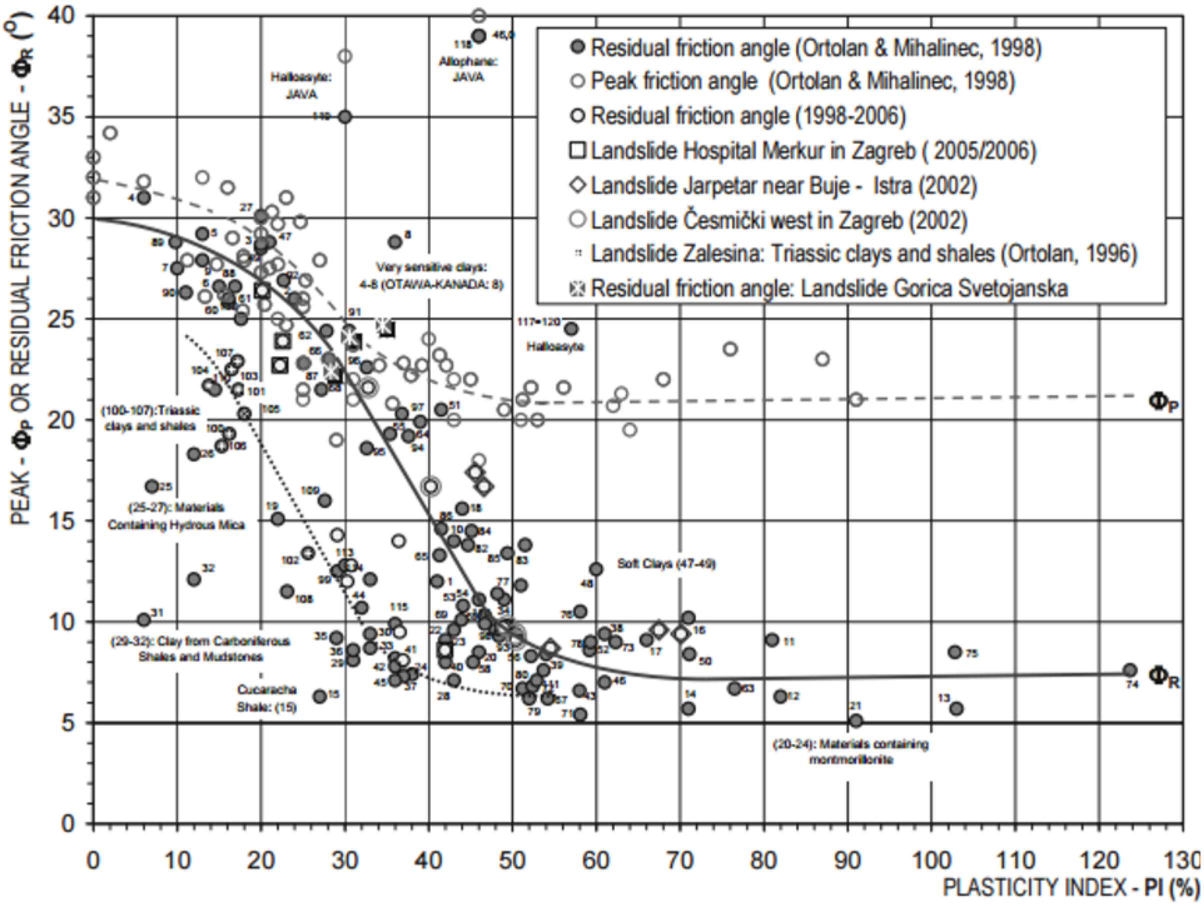


Fig. 4. Correlation between index of plasticity and angle of internal Friction - both peak and residual (Ortolan & Mihalinec, 1998).

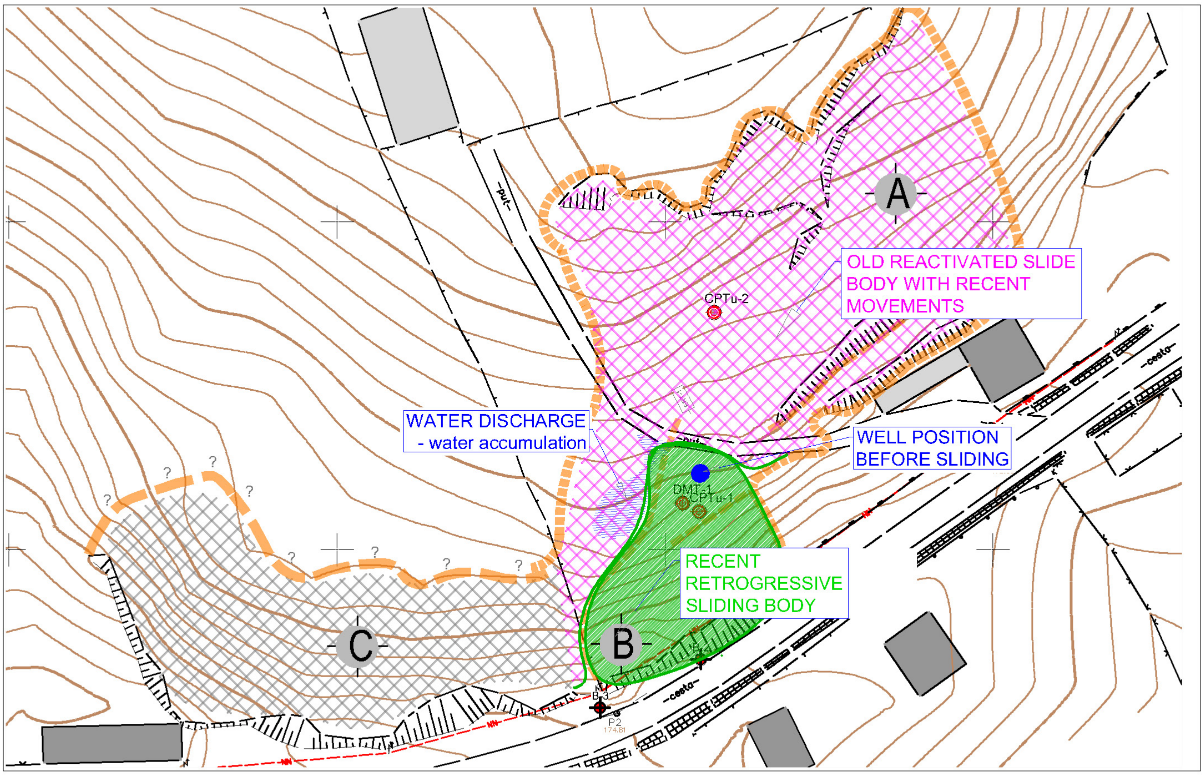


Fig. 5. Separation of the landslide parts according to movement kinematics.

Recent retrogressive part of landslide directly endangered road profile. Landslide retrogressive region marked as "B" was investigated by two static probes CPTu-1 and DMT-1, resulting with the clearly identified sliding surface with minimal measured shear strength c_u = 6,0 kN/m^2, confirmed on both static probes on the same value and on the same depth, average 4,10 to 4,40 m. Besides the confirmed depth to the surface of rupture and stable bedrock, probes also identified the small value of shear strength within the landslide depleted mass, also confirming local minimums as the results of composite slide surfaces (secondary sliding). Shear strength within the landslide depleted mass was identified in a range of c_u = 10 to 25 kN/m^2, with the distinctive difference in strength of depleted mass on "new" retrogressive part marked "B" and "older" part of the landslide marked "A" in the Fig. 5. Shear strength profiles acquired from the static probes are displayed in Fig. 7.

Kinematics of the Samarica landslide marked in Fig. 5:

A - Old landslide with recent movements related to the retrogressive development of sliding along the road corridor. New movements on the old part of the landslide are clearly restricted by visible markings, accumulation of material in the landslide toe.

B - Main depleted mass body with very low measured shear strengths. Located approximately at the foot of the retrogressive landslide part "B", former well was constructed with self-discharge after the heavy rainfall, which also confirms high seepage water potentials as the main reason for the slope sliding.

C - Expected to be an old part of the landslide, which because of dense vegetation is inaccessible and hard to characterise, without visible markings. On this area, exact landslide borders were not established except for the main scarp at the landslide crown, distant from the road corridor about 4.0 m. The area was not covered by investigation works, but it can be presumed retrogressive development progress as on the other parts of the landslide. At the found state before the investigation works, landslide endangered 35 m of road corridor with clearly visible main scarp and elevation difference at the landslide crown. By visual survey, it is clear that unstable area is larger than just the part endangering the road profile. All other landslide markings are present and detected by the visual survey, accumulation of depleted mass in the foot region, other micro terrain markings, the toe of rupture, as well as locations of groundwater discharge.

Fig. 6. Portable equipment for static probing, CPTu, on site.

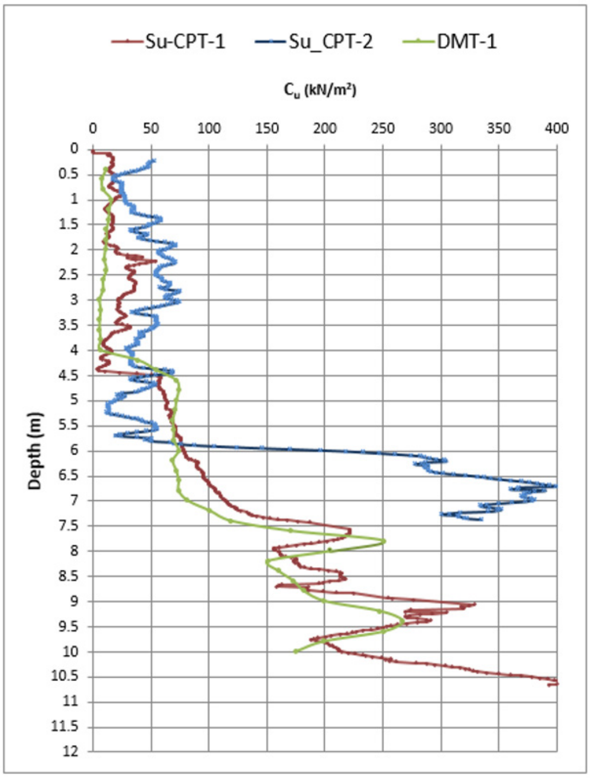


Fig. 7. Undrained shear strength (c_u) profiles acquired with CPTu and DMT probes.

2.3. Hydrogeological soil conditions

Layers that were sedimentary predisposed permits easier water seepage to deeper soil layers, which is characteristic of the silt sediments, as well as silty clays. Water is recharged in these layers at the higher ground area, so pressure and potential fluctuations are related to ground topography and frequently gains high-pressure values. This layer gradually tends to thin out, and seepage water is trapped in the slope ground, so the pressure starts to build up. Finally, a combination of high build up pore pressures and additional ground saturation from the rainfall drained from the road area, cause the slope to become unstable (Fig. 8). Located approximately at the foot of the retrogressive landslide, well was constructed with self-discharge after the heavy rainfall, which also confirms high seepage water potentials as the main reason for the slope sliding. Groundwater discharge from the landslide depleted mass creating a dead water pond proves described process on the affected site.

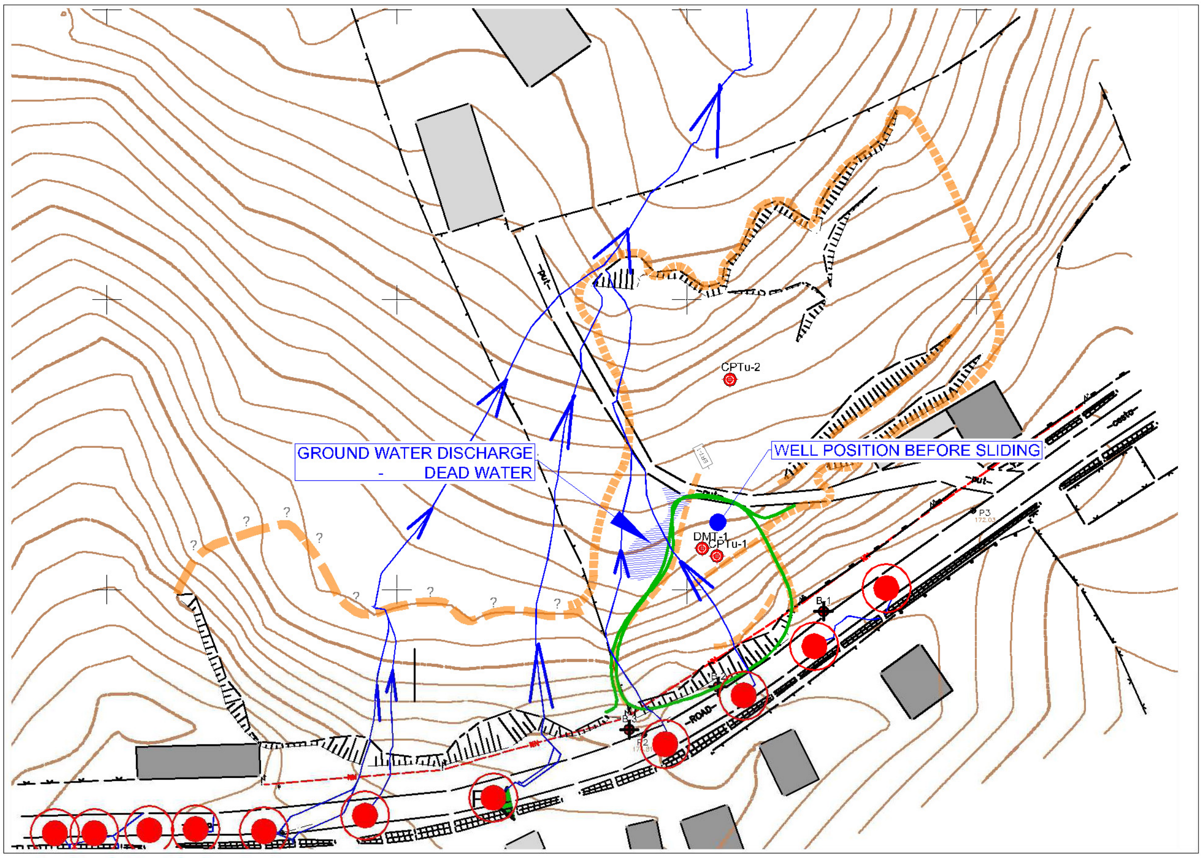


Fig. 8. Analysis of surface rainfall paths from the road corridor onto landslide area.

3. Field investigation works (subsurface investigation)

3.1. Geophysical explorations

The study site in Samarica was investigated using two geophysical methods ERT and MASW. These methods are applied in order to obtain high-quality 2D interpretation below the surface, and therefore a better understanding of the processes within the landslide. The geoelectrical and seismic measurement results were also compared to evaluate whether ERT or MASW is more appropriate for landslide investigations.

The Electrical Resistivity Tomography (ERT) is a geophysical method that provides 2D images of the distribution of the electrical resistivity in the subsoil. The imaging technique can also become more useful if the measured resistivity could be correlated with geotechnical parameters and changes in those parameters can be estimated by the proper combinations of field procedures. The analysis and interpretation of tomograms allow the identification of resistivity contrasts that can be found in some area due to the lithological pattern of the terrains or the water content variation. To determine the subsurface resistivity characteristic for different zones or layers from investigated areas an "inversion" routine of the measured apparent resistivity values must be carried out. This resistivity profile inversion was achieved using RES2DINV software, based on the smoothness constrained least-square method proposed by Loke, (1997) considering a quasi-Newton optimisation technique.

The MASW method, based on the spectral analysis of surface waves has been actively used since the early 1990s to solve the problems of engineering geophysics (Park et al., 1998, 1999; Foti et al., 2002; Roma and Pescatore, 2005). The method is based on the dispersion properties of Rayleigh surface waves, which directly provides important information about the shear wave velocity propagation in the ground to a depth of 30-50 m and even up to 100 m with the integration with passive low-frequency sources. In the early 2000s, the MASW (Multichannel Analysis of Surface Waves) method came into popular use among the geotechnical engineers.

The term "MASW" originated from the publication made on Geophysics by Park et al., (2001). The multichannel analysis of surface waves (MASW) method provides one of the most critically important geotechnical parameters the stiffness of ground materials. It gives this information in terms of seismic shear-wave velocity (V_s) distribution in both vertical and horizontal directions. From an elastic theory viewpoint, shear wave velocity (V_s) is the most powerful indicator of a material's stiffness.

Shear-wave velocity (V_s) is one of the elastic constant and closely related to Young's modulus. Under most circumstances, V_s is a direct indicator of the ground strength (stiffness) and therefore commonly used to derive load-bearing capacity. Most important properties of R waves is a wave velocity frequency dispersion. Wave propagation velocity at each particular frequency is called phase velocity, whereas curve representing phase velocity in dependence of frequency is called phase velocity curve or dispersion curve. Spatial P an S seismic waves do not have dispersion characteristics. Receiver spread for the MASW profile consisted of 24 vertical 4,5 Hz geophones, with receiver spaced out 3,0 m. For the interpretation purpose fundamental mode of the dispersion curve is used, M0. Experimental dispersion curve measured on the field was interpreted using SeisIMAGER 4.0.1.6. OYO Corporation 2004-2009 computer software.

3.2. In-situ investigations (CPTu, DMT, DPL)

In-situ testing is fast, continuous, and provides immediate results for use in analysis. Thus, the optimal site characterisation program involves a series of soil borings, complemented by a series of in-situ testing and laboratory reference testing. Over 50 different in-situ field testing devices have been developed for either general or specific measurements in soil (Lunne et al., 1992). Some of the common in-situ tests are: including the standard penetration test (SPT), cone penetration test (CPT), piezocone (PCPT), flat dilatometer (DMT), pressuremeter (PMT), vane shear test (VST), dynamic penetration heavy test (DPH), dynamic penetration light test (DPL) and several geophysical methods: crosshole test (CHT), downhole test (DHT), spectral analysis of surface waves (SASW & MASW) and electrical resistivity tomography (ERT).

The utilisation and interpretation of these tests are well-established (Kulhawy & Mayne, 1990). The routine in-situ tests (SPT, CPT, DMT, PMT) have at least two decades of experience in practical applications. The development of hybrid devices, particularly penetration tests with geophysical measurements, is attractive because of the interpretative procedures for P, S, and R waves are fairly well-established (Mayne, 2000).

3.2.1. CPTu test

Complete reports on the interpretation of piezocone tests focusing on the several aspects of engineering practice are given by Jamiolkowski et al., (1985). For a general review of the subject reference to Lunne et al., (1997), CPT in Geotechnical Practice, is advised. Field CPTu probing procedure includes hydraulic pushing of the steel cone penetrometer with constant speed to obtain a vertical profile of stress, pressures and other measured parameters. To conclude the test no borehole is needed, but often predrilling in superficial rock fill is necessary. The testing procedure is carried out in accordance with ISO 22476-1:2012. A result of probing process is supervised online by the operator at the surface, so the decision on dissipation test is done in particular soil layer. Pore pressure is measured through the filter at the U2 position on the probe penetrometer cone was also recorded.

3.2.2. Flat dilatometer test

The DMT was developed in Italy by Silvano Marchetti in 1980. It is recognized here that Marchetti's efforts were of a pioneering nature and subsequent works on this subject followed his original basic conceptual framework. From a historical point of view, the DMT was conceived to establish a reliable operational modulus for the problem of laterally loaded piles. Later, the interpretation of test results was extended to provide measurements of in situ stiffness, strength and stress history of the soil. The equipment, test method and original correlations described by Marchetti (1980) were the essential initial steps towards establishing a procedure capable of supporting geotechnical applications which then become transformed into ISO/TS 22476-11 norm.

Flat dilatometer probing is based on pushing the steel blade probe with membrane into the ground and periodically stopping to measure specific pressures at designated depths. For dilatometer testing also no boreholes are needed although test can be performed in the borehole by advancing DMT blade into undisturbed part of borehole bottom. Testing is usually performed at every 200 mm depth interval distance. At each test, 60 mm diameter membrane is deformed under pressure to take A and B reading at designated deformation, and optionally C reading is taken in sandy soil material.

3.2.3. Dynamic penetration light (DPL)

Soil testing with Dynamic probe light (DPL) is a dynamic penetration test at which probe cone is rammed into the ground by 10 kg hammer falling from the height of 50 cm. Probe cone point is at 90° angle and has a surface area of 10 cm^2.

Dynamic penetration resistance is defined as a number of blows to penetrate 10 cm (N_{10L}). In cohesive soils, friction between soil and pushing rods builds up and has an important influence on the measured penetration resistance, as also increases with the length of the pushing rods. To reduce the rods-soil friction, pushing rods are turned for 1.5 turns, overcoming maximum torsional moment, at each addition of new rod (every 1 m depth), while taking readings of the torsional moment (T) needed to overcome rod-soil friction.

It is clear that much of the driving energy was being absorbed in overcoming skin friction build-up on the rods, but the prevailing theory currently embedded in standard ISO 22476-2:2008 for skin friction is deficient (Cope, 2010). It assumes that rod string has no mass and no concern was made of the conservation of momentum. As well, the question of the transferring of hammer energy from the anvil to the probing tip is also omitted from the standard. According to the recommendations from the author (Cope, 2010), adopting "R_d" is the appropriate symbolism given that it is historically associated with the more completed formula, opposite to q_d from the ISO 22476-2:2008, Annex E. Equation derived by Cope 2010, can now give normalized output results for all kind of dynamic testing equipment.

In the case study presented in this paper, results of dynamic probing are presented by probe dynamic penetration resistance R_d, as suggested by Cope 2010, and were calculated by the next expression:

$$\mathrm{R_d} = \frac{g}{A} \cdot \left[\left(\frac{M^2}{M + M'} \right) \cdot \frac{h}{0.1} \cdot N_{10} + (M + M') \right] - \frac{T}{A \cdot r} \qquad (1)$$

where:

R_d - dynamic resistance of the soil penetration probe [MPa],
M - mass of the weight, 10 kg [kg] (DPL),
M' - the mass of the driving rods, 3.0 kg [kg] (DPL),
h - the height of the fall of the weight, 0.50 m [m] (DPL),
N_{10} - the number of strokes needed to penetrate the probe 10 cm [-],
T - torque required to rotate the rod [Nm],
r - driving rods radius, 22 mm [m] (DPL),
A - probe cone surface, 10 cm^2 [m^2] (DPL),
g - acceleration from the gravitational force [m/s^2]

Interpretation results are corrected by taking into account influence of torsional moment due to the rod-soil friction. Dynamic probing in most cases is used along the core drilling to obtain soil profile. As well as for the static probing, numerous correlations are available for the dynamic probing as well, making available various soil parameters as well as correlations with others in-situ investigation methods, mostly standard penetration test (Strelec at al., 2016b).

Fig. 9 shows employed in-situ probes: CPTu static penetrometer cone, flat dilatometer blade probe for DMT and DPL dynamic probe with a portable pneumatic ramming device. Applicability of in-situ tests according to the soil type is illustrated in the Fig. 10.

Fig. 9. CPTu penetrometer cone - left, DMT blade - middle, DPL pneumatic ramming device- right.

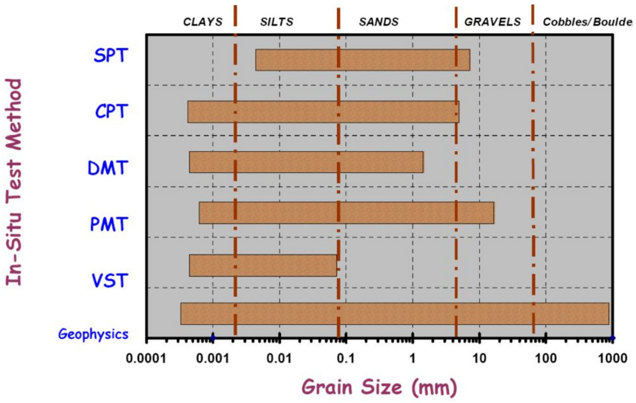


Fig. 10. Applicability of in-situ tests (Mayne et al., 2002).

4. Results and discussion

Geoelectrical tomography (ERT) in this paper were performed on single longitudinal profile along the main landslide body with the objective to gain as detailed imaging data about the landslide rupture surface or information about the stable bedrock depth. Fig. 11 presents a composite geotechnical profile of measured electrical resistivity overlapped with soil shear strength profiles obtained from CPTu probes. It is evident from the Fig. 11 that after 6 inversion iterations, resistivity profile model has only 0.84 error which proves good and reliable data interpretation. Location of electrical profile ERT can be seen on situation plan in Fig. 1. The dashed line in Fig. 11 separate area of landslide depleted mass which is in accordance with CPTu-1 and CPTu-2 static probe results. Resistivity tomography profile clearly identifies an unstable sliding mass, corresponding to increased electrical resistivity values which are in contrast to surrounding ground (CL) on tomography profile represented in bluish colours. Presented case shows how the depth of sliding plane perfectly aligns with data obtained from the static CPTu probes. Electrical resistivity values lower than 30 Ωm in deeper parts of profile indicate clays of lower permeability forming the bedrock of the landslide-sliding plane. Immediately above, is a depleted landslide mass of highly water-saturated silty clays with electrical resistivity 30 to 50 Ωm, low to medium permeability. At deeper parts (right-hand side of the profile), silty-sandy clays appear with increasing values of electrical resistivity (50-100 Ωm) adequate for the increasing sand content and by that of higher permeability.

Standard values obtained from the completed static probes at the site CPTu-1 and CPTu-2 are displayed in Fig. 12 (left side of the chart), total cone resistance q_t (MPa), friction ratio R_f (%) and pore pressure u (kPa). The right part of the Fig. 12 depicts interpreted lithological profile form the results of the static probing CPTu-1. Friction angle (Φ), constrained modulus (M_v) and shear wave velocity (V_s) are given in Fig. 13. In Fig. 12, contact of depleted sliding mass and firm bedrock is indicated on the depth of 4.4 m, at which all measured values are at a minimum. For deeper penetration into the firm soil, all parameters suddenly increase. At Samarica landslide, depth of the sliding plane acquired by static CPTu probes exactly aligns with the indicated depth on electrical tomography profiles, see Fig. 11. Geotechnical parameters interpreted from the values obtained from static penetration in Fig. 13 were calculated according to the expression given by Robertson (2014).

Marchetti flat dilatometer was also used for the determination of the depth of landslide sliding plane. A method in use is based on the determination of dilatometer detection of soil in situ horizontal stress index (K_D) (Totani et al., 1997). Clays at shallow ground layers are in most cases overconsolidated, whereas at the sliding plane material is disturbed, lose primary structure and any cementation, so it behaves as normally consolidated clay. In accordance with the stated, K_D method is based on the identification of zones of normally consolidated clays in a slope, where parameter $K_D \approx 2$. At the surface of rupture (sliding plane), in situ K_D adopts roughly named value and by that defines the depth of sliding plane. Overconsolidation index (OCR) is directly related to in situ horizontal stress index (K_D) (Špiranec et al., 2016). From the Fig. 14, OCR values according to various authors can be read off. From the presented Fig. 14, it is evident that OCR = 1 corresponds to coefficient $K_D \approx 2$.

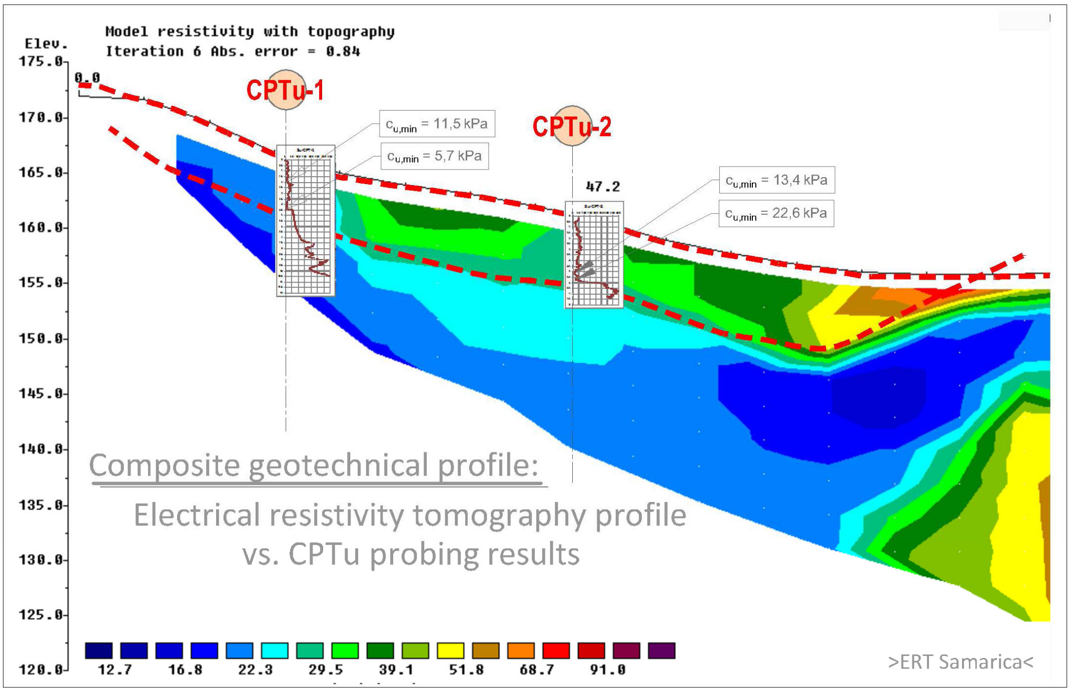


Fig. 11. Inversion soil model of electrical tomography profile "ERT-1 Samarica".

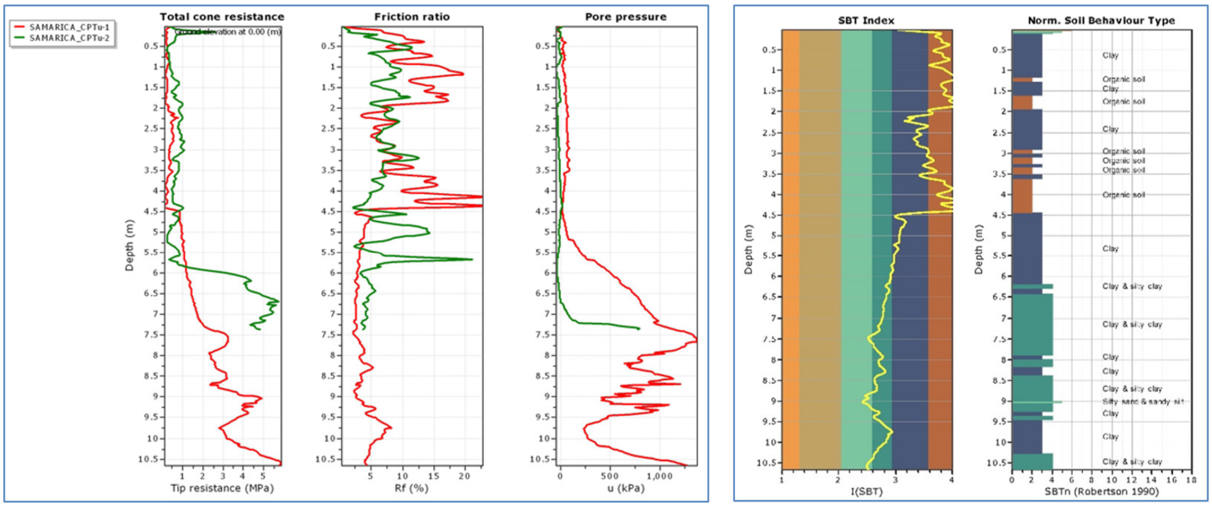


Fig. 12. Depth profiles of measured static probe parameters for Samarica Landslide (CPTu-1 & CPTu-2) - left and interpreted lithological profile for CPTu-1 probe - right.

Fig. 15 displays results of dynamic probing utilising DPL probe, defined by a number of blow count for the penetration of 10 cm (N_{10L}). To minimise the influence of driving rods friction in the soil, torsional moment (T) is recorded at each rod addition (1m interval), needed to overcome the pushing rod skin friction and turn the rods freely. Results of dynamic probing can be compared with the cone penetration resistance (Czado and Pietras, 2012) and they are expressed by probe dynamic penetration resistance R_d. Locations of the DPL probes are indicated on situation plan in Fig. 1.

In Fig. 16, measured dilatometer pressures p_0 and p_1 acquired by Marchetti flat dilatometer probe are displayed, as well as calculated dilatometer modulus E_D (location of DMT probe is indicated in Fig. 1). In situ acquired horizontal stress index (K_D) equals the value of 2 at a depth of 4 m which is an indication of sliding plane depth. From the CPTu probe results, Figures 12 and 13, it is indicated that the sliding plane depth is at 4.4 m. The small difference in depth is explained as elevation difference of probes, as well as the horizontal distance between them.

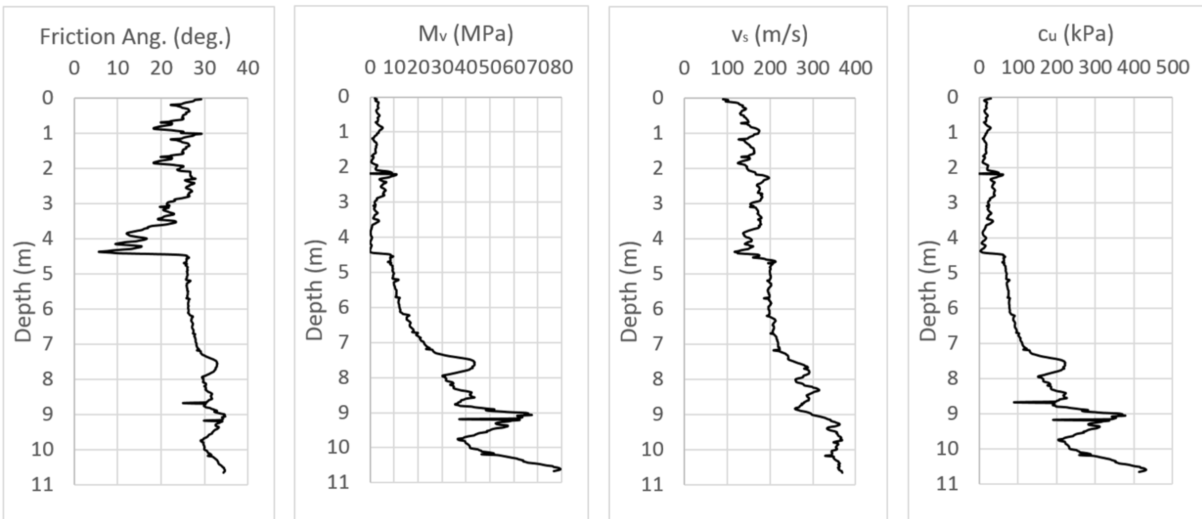


Fig. 13. Example of estimated geotechnical parameters from the static probe (CPTu-1) for friction angle - Φ , constrained modulus - M_v, shear wave velocity - v_s, undrained shear strength - c_u, - at the "Samarica landslide".

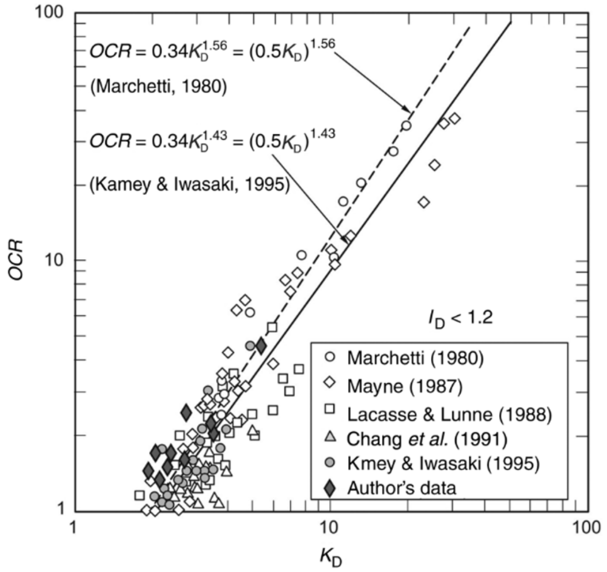


Fig. 14. Correlations of K_D and OCR for cohesive soils (Kamey and Iwasaki, 1995).

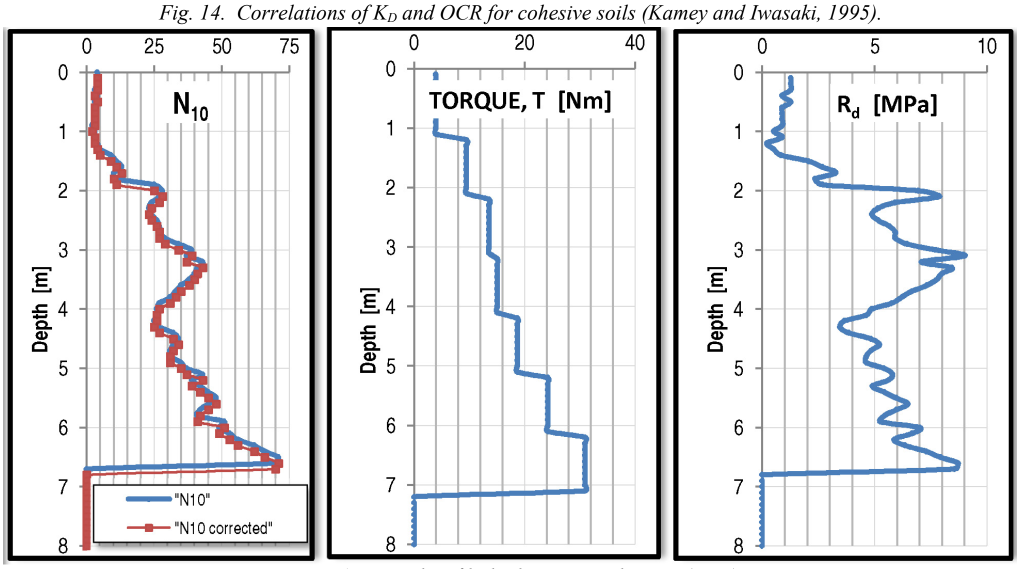


Fig. 15. Results of light dynamic probe N_{10L} (DPL).

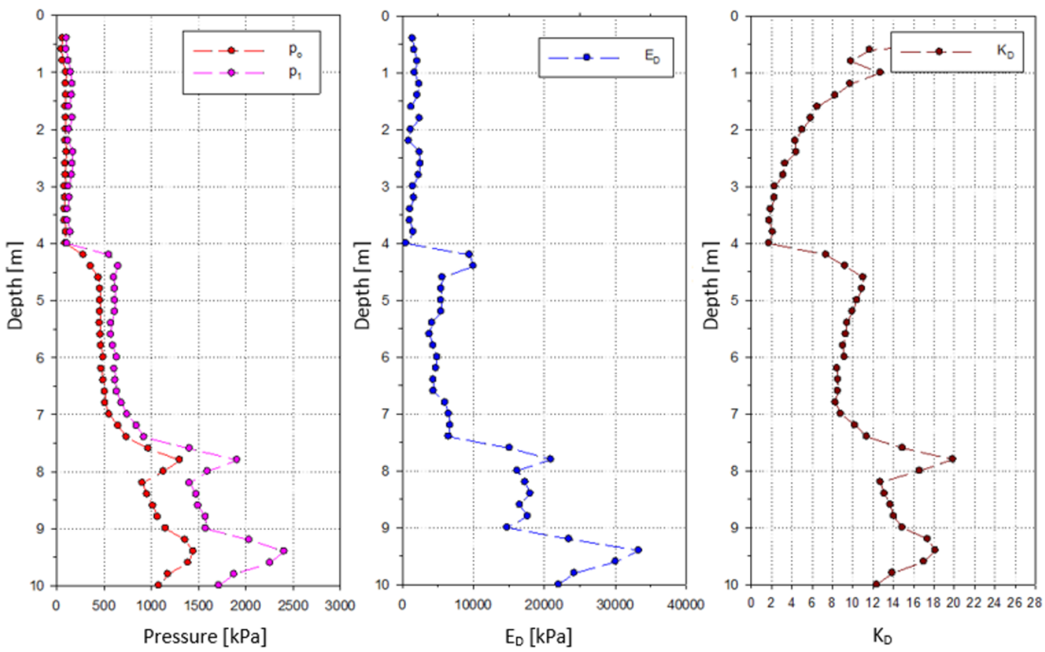


Fig. 16. Display of measured dilatometer pressures p_o i p_1 of Marchetti flat dilatometer, calculated Dilatometer Modulus (E_D) an in situ horizontal stress index (K_D), at Samarica landslide site.

Road instability at the place Samarica is a landslide with composite mass movements, and by the time of origin, it is an old landslide recently reactivated with new retrogressive development along the road corridor. From the Fig. 16, it can be observed the existence of two sliding planes, shallow at 2 m depth, and deeper at 4.1 to 4.4 m depth. Similar anomalies are also visible in Fig 13 from CPTu-1 results, where parameters M_v, v_s and c_u become minimal, as well as in the Fig. 16, where dilatometer modulus is also at a local minimum at 2 m depth.

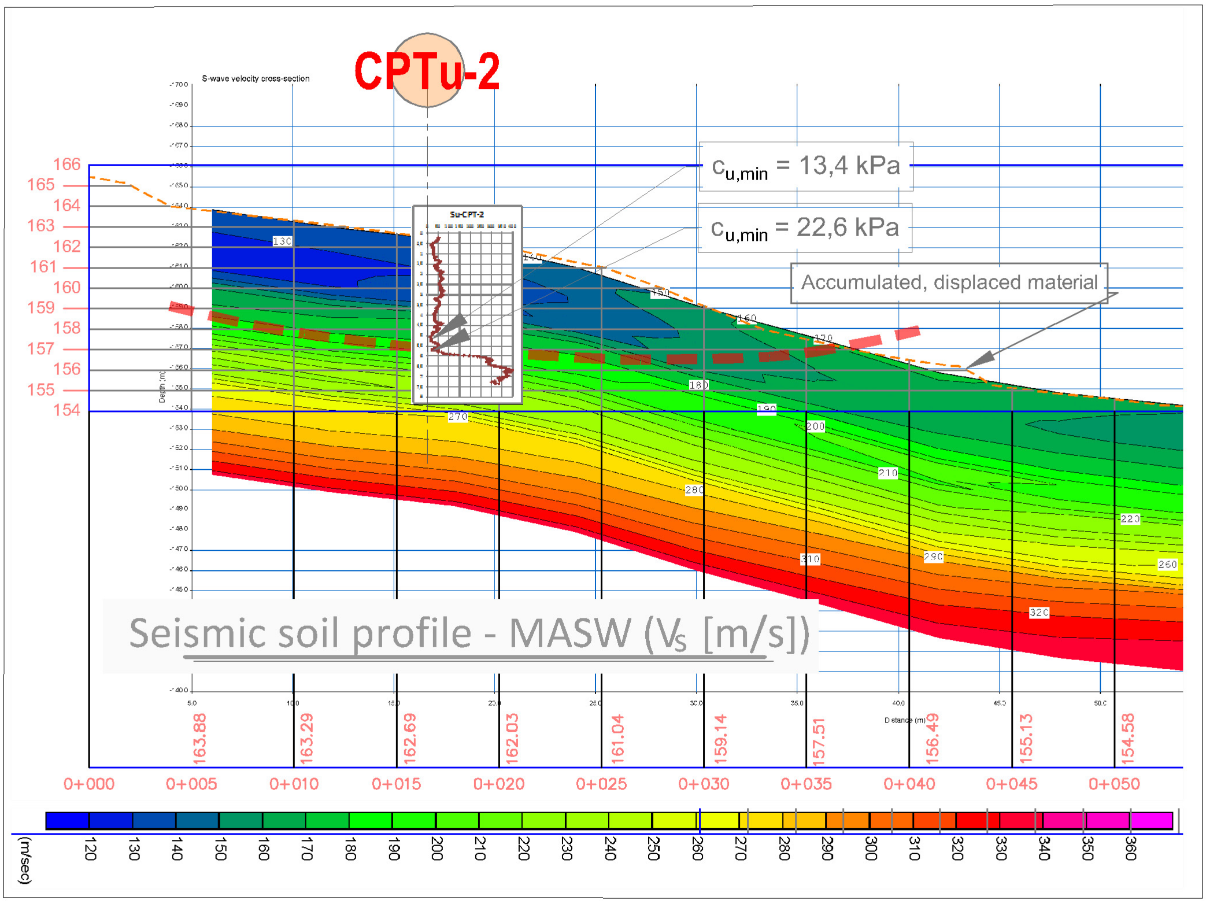


Fig. 17. MASW-1, the 2D profile of seismic shear wave propagation velocities V_s (Samarica landslide).

Seismic shear wave propagation velocities v_s shown in Fig. 17. is obtained from the MASW seismic profile and matches the seismic velocities interpreted from the CPTu-1 static probe measurements depicted in Fig. 13. As well, in Fig. 17, sliding plane depth indicated from c_u result of CPTu profile corresponds to the increase of shear wave velocity, as depicted in Fig 17 by the thick red line. Location of the seismic receiver MASW spread is illustrated in the Fig. 1.

Borderline of the landslide depleted mass depicted in Fig. 17, overlapping the 2D MASW seismic profile interpretation, is not as much evident as borderline in Fig. 11 acquired from the electrical tomography imaging, ERT.

5. Conclusion

Landslide model presented in Fig. 18 is a result of detailed investigation works and is displayed on a characteristic geotechnical profile. Findings have shown that the landslide shape is a result of multiple sliding processes that proves the retrogressive character of recent sliding along the road corridor. It is concluded that slope structure in combination with a lithostratigraphic composition of the ground, along with the hydrogeological conditions in soil with additional quantities of rainfall waters from the road corridor, are the main reasons for the development of landslide on the affected road section.

Paper gives an overview on the application of classic investigation methods (core drilling, SPT) and the newer in-situ investigation methods CPTu, DMT & DPL, as well as geophysical explorations (ERT & MASW). By combining results of in-situ methods, it is possible to calibrate/identify lithography data obtained from geophysical methods, enabling 2D/3D landslide modelling. In this case study, the engineering purpose of geophysical explorations was to identify geometrical outline between sliding mass and stable soil formations, as well as to determine elastic modulus needed in geotechnical modelling. By interpreting results of ERT tomography, information on electrical soil resistivity was acquired, that is in this case quality-identified landslide depleted mass from the stable bedrock formations. ERT tomography method can be considered appropriate for geometrical landslide outlining, as well as recognising potentially unstable slope areas. From the other side, electrical profiling results need to be calibrated with data obtained from other in-situ investigations.

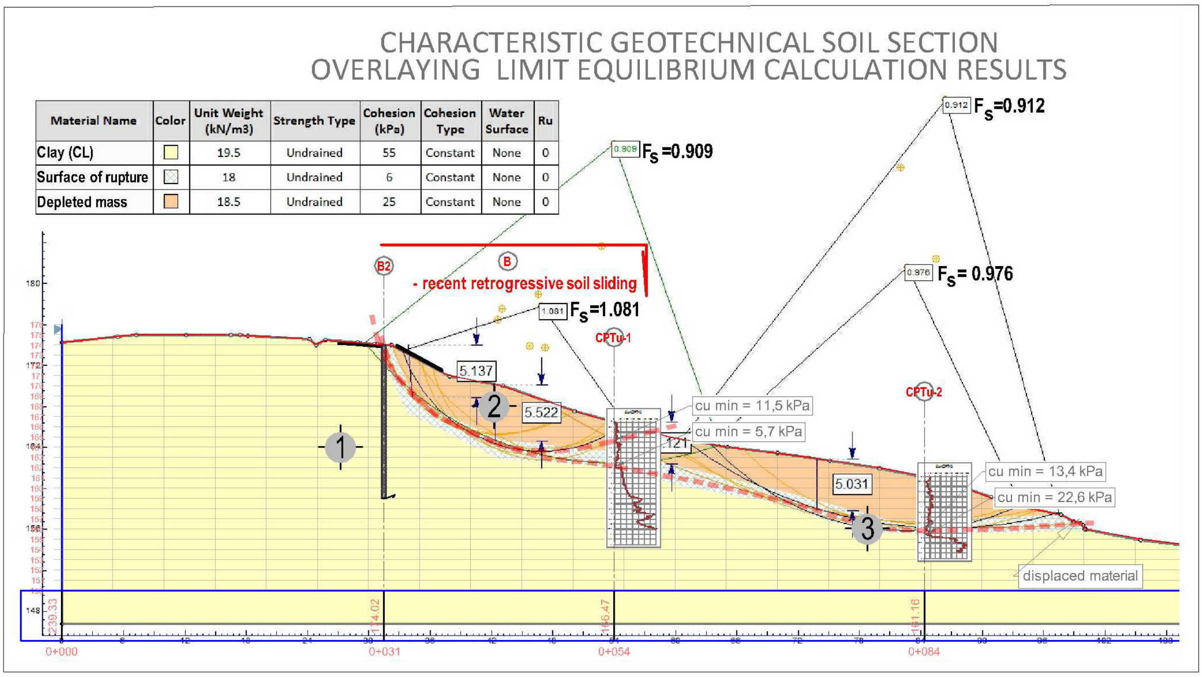


Fig. 18. Landslide model displayed on a characteristic geotechnical profile.

By the geophysical MASW method, the subsurface propagation velocity of seismic S wave has been obtained (Fig. 17). From the elasticity theory point, shear velocity (v_s) is the best indicator of material stiffness, while obtained borderline is not as much evident with the depleted sliding mass as borderline shown in Fig. 11 acquired from electrical profiling.

Application of in-situ investigation techniques (CPTu, DMT, DPL), made available physical and mechanical parameters immediately after the probe testing (Fig. 12 to 16), which are then needed for calculations of bearing capacity, settlements, sliding stability, liquefaction and various geotechnical calculations. Light dynamic probing DPL, have also proven to be very suitable for detection of sliding planes. Equipment for DPL probe is light and can also be used in coarse, gravelly materials. Dynamic probing offers a wide variety of parameters correlated with N_{10L}, especially for noncohesive materials. Borehole for DPL dynamic probing is also not needed. Best interpretations results from the dynamic penetration are obtained using the expression for the dynamic penetration resistance *Rd* (1), developed by Cope (2010).

Advantages of DMT and CPT in-situ probing are fast and continuous soil profiling, repeatability of measurements and reliability of acquired data. Main disadvantages are relatively high start capital investments, need for a highly skilled operator, unable to take soil samples while testing, and limited use in gravelly and cemented soil layers.

Investigation results accomplished from the combination of proposed investigation methods used on the presented case study will be the base for the additional investigation works to be carried out on the western parts of Sljeme urbanised area (town Zagreb). In order to obtain confirmation of the depths of the investigated sliding surface, additional investigative works will include measurements of horizontal displacements by installing inclinometers and consequent comparison with the methods described in the article.

References

Cope, M.: Dynamic probe theory revisited – effective interpretation of dynamic test results, *CETANZ Conference 2010 proceeding, Auckland, New Zeland,* www.cetanz.org.nz

Czado, B. and Pietras, J.S.: Comparison of the cone penetration resistance obtained in static and dynamic field tests. *AGH Journal of Mining and Geoengineering, 2012, vol.36, no. 1, 97-105.*

Foti, S., Lai, C.G., Rix, G., Roma, V.: Simultaneous measurement and inversion of surface wave dispersion and attenuation curves, *Soil Dyn. Earthquake Eng., 2002, vol. 22, pp. 923–930*.

Kamey, T. and Iwasaki, K.: Evaluation of undrained shear strength of cohesive soil using a flat dilatometer. *Soils and Found, 1995, 35(2):111–116.*

Kulhawy, F.H. and Mayne, P.W.: Manual on estimating soil properties for foundation design. *Report EL6800. Electric Power Research Institute, Palo Alto, 1990, 306 p.*

Mayne, P.W.: Enhanced geotechnical site characterization by seismic piezocone penetration tests. Invited Lecture, Fourth International Geotechnical Conference, *Cairo University, January 2000, pp. 95-120*.

Jamiolkowski, M., Ladd, C.C., Germaine, J., Lancellotta, R.: New developments in field and lab testing of soils, Proceedings, 11th International Conference on Soil Mechanics and Foundations Engineering, *1985, Vol. 1, San Francisco, 57-154.*

Lunne, T., Lacasse, S., Rad, N.: General report: SPT, CPT, PMT, and recent developments in in-situ testing. Proceedings, 12th Intl. *Conf. on Soil Mechanics & Foundation Engrg., 1992, Vol. 4, Rio de Janeiro, 2339-2403.*

Lunne, T., Robertson, P.K., Powell, J.J.M.: Cone Penetration Testing in Geotechnical Practice. *Blackie Academic/Chapman-Hall Publishers, U.K., 312 pages, 1997.*

Loke, M. H.: Res2DINV software user's manual, *University Sains Malaysia, Penang, 173p, 1997*

Marchetti, S.: In situ tests by flat dilatometer. J. Geotech. Engng Div., ASCE, 106(3):299–321, 1980.

Mayne, P.W., Christopher, B., Berg, R., and DeJong, J.: Subsurface Investigations - Geotechnical Site Characterization. *Publication No. FHWA-NHI-01-031, National Highway Institute, Federal Highway Administration, Washington, DC: 301 pages, 2002.*

Ortolan, Ž. & Mihalinec, Z.: Plasticity index - indicator of shear strength and a major axis of geotechnical modelling. *Proc. of the XIth Danube–European conference on soil mechanics and geotechnical engineering, May 25–29, 1998, Poreč, 743–750, A.A. Balkema, Rotterdam*.

Park, C.B., Miller, R.D. and Xia, J.: Multichannel analysis of surface waves. *Geophysics, 64, 3, 800-808, 1999.*

Park, C.B., Xia, J., and Miller, R.D.: Imaging dispersion curves of surface waves on multichannel record. Expanded Abstracts of 68th Annual International Meeting of the Society of Exploration Geophysicists, *New Orleans, USA, pp. 1377–1380, 1998*.

Park, C.B., Miller, R.D., and Xia, J.: Offset and resolution of dispersion curve in multichannel analysis of surface waves (MASW), *Proceedings of the SAGEEP2001, Denver, USA, pap. SSM4.*

Robertson, P.K., Cabal, K.L.: Guide to Cone Penetration Testing for Geotechnical Engineering, *2014*.

Roma, V. and Pescatore, M.: Environmental impact caused by high speed train vibrations, International Geotechnical Conference "Soil-Structure Interaction: Calculation Methods and Engineering Practice", *St. Petersburg, Russia, St. Petersburg, 2005. pp. 26–28.*

Strelec,S., Gazdek, M., Mesec, J., Jug, J., Stanko, D. (a): Primjena in situ i geofizičkih metoda u istraživanju cestovnog klizišta. (Implementation of in situ and geophysical methods for road landslides investigations). *Geotechnical aspects of damages caused by natural phenomena, 7th conference of Croatian Geotechnical Society with international participation, 10. – 12 .11.2016. Varaždin, Croatia.*

Strelec, S., Stanko, D., Gazdek, M. (b): Empirical correlation between the shear-wave velocity and the dynamic probing heavy Test; Case study Varaždin, Croatia*, Acta Geotechnica Slovenica (1854-0171) 13 (2016), 1; 3-15.*

Špiranec, M., Grabar, K., Strelec, S.: Određivanje klizne plohe in situ pomoću dilatometarskog (DMT) ispitivanja ("Kd metoda"), (Determination slip surface in situ by dilatometer (DMT) test ("Kd method")). *Geotechnical aspects of damages caused by natural phenomena, 7th conference of Croatian Geotechnical Society with international participation, 10. – 12 .11.2016. Varaždin, Croatia.*

Totani G., Calabrese M., Marchetti S., Monaco, P.: Use of in situ flat dilatometer (DMT) for ground characterization in the stability analysis of slopes. *Proceeding of 14th International Conference on Soil Mechanics and Foundation Engineering, Hamburg, September 1997, vol. 1, pp. 607–610.*