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ANALIZA SLEGANJA TEMELJA USLED PROGRESIVNOG PROVLAŽAVANJA PRAŠINASTIH GLINA[****]

Izvod

Nisu retki primeri vidljivih oštećenja (pukotine i prsline po fasadnim zidovima) na starijim građevinama Beograda koje su fundirane plitko. Najčešće je to posledica neravnomernih sleganja koja često nastaju usled nepredviđenog provlažavanja tla ispod temelja. Ovo provlažavanje je uglavnom lokalnog karaktera i u takvim uslovima je neminovna pojava diferencijalnih sleganja. U ovom radu su izloženi rezultati istraživanja koji ukazuju da pored veličine zone uticaja promene vlažnosti u tlu u horizontalnom pravcu i proračunatih vrednosti sleganja, treba anlizirati i ukupnu promenu zapremine tla ispod temelja.

Ključne reči: *provlažavanje tla, sleganje, statička penetracija, promena zapremine.*

1. UVOD

Voda predstavlja vitalnu i najaktivniju komponentu tla jer je stalno u pokretu. Njeno prisustvo u tlu zavisi od brojnih faktora, a pre svega od raspoložive količine (padavine, kvašenja, curenja drenažnih sistema, i sl.), ali i od brzine kojom voda osvaja tlo (razvijen proces evapo-transpiracije, prisustvo vegetacije, postojanja prirodnih i veštačkih drenažnih sistema i sl.). Manje promene vlažnosti događaju se tokom godine kao rezultat sezonskih promena usled jakih kiša, česte promene temperature i sl. Međutim, veći uticaj na tlo one imaju za vreme dugotrajnih padavina odnosno dugačkih sušnih perioda. Ovaj efekat klimatskih promena postaje važniji ukoliko je prisutna i vegetacija (npr. neke posebne vrste drveća dnevno mogu da iscrpe preko stotine litara vode u vrelim danima) izazivajući u određenoj meri

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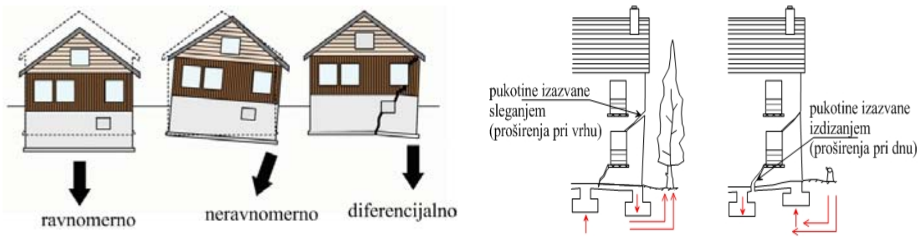
skupljanje tla, što opet može da izazove vidljive deformacije na objektu. Suprotno tome, odsustvo vegetacije vodi ka povećanju vlažnosti, ne retko i bubrenju glinenih tla, koje ponekad može izazvati izdizanje objekta. Zbog toga se na objektima mogu javiti različiti vidovi deformacija [1] (slika 1). Napominjemo da za vreme dužih sušnih perioda i korenje drveća može prouzrokovati uglavnom mehaničke štete na podzemnim delovima objekta usled njihovog rasta u dublje i vlažnije delove terena.



a) *b)*

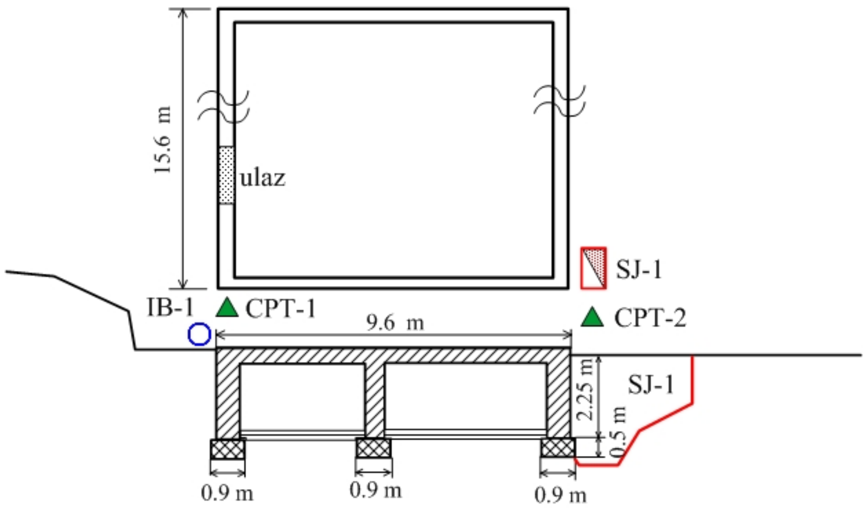
Sl. 1. a) *Opšti tipovi sleganja temelja, b) Pojava dijagonalnih pukotina u zidovima zbog diferencijalnih sleganja*

Samo kretanje vode, bilo da se obavlja pod uticajem gravitacionih odnosno negravitacionih sila (prirodnog ili antropogenog porekla), u najtešnjoj je interakciji s čvrstom komponentom, menjajući često mehanička svojstva a samim tim i fizičko stanje, pa i ponašanje. U početku su promene lagane, gotovo neprimetne, ali vremenom mogu biti i nepredvidljivih razmera dovodeći često objekat u stanje koje zahteva hitne intervencije. Zato, ako se ispolje ove promene (npr. ispod temelja objekata) stvaraju se i uslovi za pojavu diferencijalnog sleganja u temeljnoj konstrukciji, što dalje može da dovede do velikih oštećenja objekta [2], [3]. Međutim, nepoznavanje konkretnih uslova koji postoje u tlu posle provlažavanja, navodi na projektovanje i izvođenje tehnički i ekonomski neadekvatnih sanacionih mera. U tom slučaju od značaja je poznavati veličinu zone uticaja promene vlažnosti, odnosno veličinu zone u kojoj dolazi do promene fizičko-mehaničkih svojstava tla. Veličina ove zone je različita za različite vrste tla. Zato će se u ovom radu dati neki rezultati istraživanja u određivanju veličine zone uticaja promene vlažnosti u prašinastim glinama.

2. IZVEDENA ISTRAŽIVANJA

Predmet ovog rada je prizemni objeka u okolini Surčina, koji je u konstruktivnom smislu izgrađen od masivnih zidova. Osnova objekta je pravougaona, dimenzija 15,6 x 9,6 m. Zidovi su od opeke u krečnom malteru debljine d = 0,55 m. Krovna konstrukcija je drvena na dve vode sa krovnim pokrivačem od crepa. Objekat ima podrum koji je ukopan u zemlju 2,1 m, u odnosu na nultu kotu terena. Fundiran je na temeljnim trakama širine B = 0,9 m na koti fundiranja od 2,75 m. Kontaktno naprezanje ispod temeljnih traka je reda veličine Δq = 130 kN/m^2. Odvodnjavanje vode sa krova vrši se preko olučnih vertikala koje prikupljenu vodu izvode na površinu terena tako da dalji odvod vode nije regulisan. Posle dužeg korišćenja objekta (više desetina godina), došlo je do ozbiljnih oštećenja na mestima olučnih vertikala u vidu vrlo progresivnih i razvijenih pukotina koje ugrožavaju njegovu dalju eksploataciju [4].

U cilju odredjivanja fizičko-mehaničkih karakteristika kao i veličine zone uticaja promene vlažnosti u tlu, izvršena su određena geomehanička istraživanja na mestima olučnih vertikala gde su oštećenja i najveća tj. na mestima gde je najverovatnije došlo do provlažavanja tla. Sprovedena geomehanička istraživanja su obuhvatala iskop jedne istražne jame (SJ-1) u zoni najvećih oštećenja tj. pretpostavljenoj zoni provlažavanja, izvođenja jedne istražne bušotine (IB-1) van zone provlažavanja, dve statičke penetracije (CPT-1 i CPT-2) i geomehanička laboratorijska ispitivanja. Situacija objekta sa položajem istražnih radova prikazana je na slici 2.

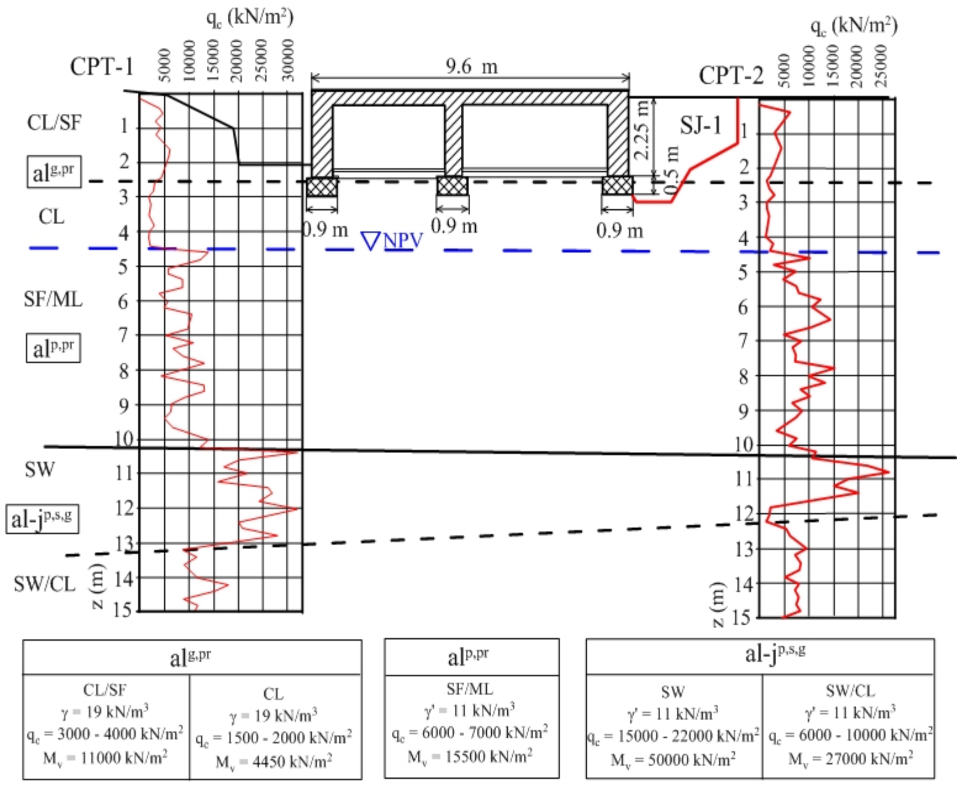


Sl. 2. *Prikaz načina fundiranja objekta sa položajem istražnih radova*

Istražna jama (SJ-1) je ručno kopana do dubine od 3,5 m tj. oko 0,7 m ispod kote fundiranja obuhvatajući tako i zonu provlažavanja. Nakon iskopa istražne jame izvršena je provera dimenzija temelja, inženjerskogeološko kartiranje tla i uzimanje 5 neporemećenih uzoraka za geomehanička laboratorijska ispitivanja. Pored istražne jame, urađena je i jedna istražna bušotina (IB-1) i dva opita statičke penetracije (CPT-1 i CPT-2). Dubina sondiranja terena je bila 15 m. Prva statička penetracija (CPT-1) je izvedena pored istražne bušotine IB-1 u zoni koja nije bila zahvaćena provlažavanjem. Druga statička penetracija (CPT-2) iz tehničkih razloga nije izvedena u samoj zoni provlažavanja, tj. na mestu istražne jame (SJ-1), već na oko 2.0 m od objekta. U cilju utvrđivanja fizičkomehaničkih svojstava tla izvršeni su identifikaciono-klasifikacioni opiti, opiti direk-tnog smicanja i edometarski opiti stišljivosti.

3. REZULTATI ISTRAŽIVANJA

Rezultati statičko penetracionog sondiranja, inženjerskogeološko kartiranje istražne bušotine i istražne jame, pokazali su da je teren izgrađen od sledećih litoloških članova (slika 3):



Sl. 3. *Geotehnički presek terena*

- Prašinaste gline, prašine i prašinast pesak ($al^{g,pr}$), smeđe i sive boje, sa značajnim udelom organskih primesa i pojavom tankih proslojaka peska; srednje stišljive; srednje i slabije vodopropusne. U istražnoj jami (SJ-1) od dubine 2,0 m pa nadalje u zoni fundiranja, utvrđena je zona provlažavanja sa izrazito vlažnim, mekanim, i jače stišljivim prašinastim glinama.
- Pesak prašinast ($al^{p,pr}$), finozrn do krupnozrn sive i smeđe boje sa neujednačenim udelom sitnozrne frakcije i karakterističnom pojavom proslojaka mulja sa dosta organskog detritusa. Postoji nanos sa izraženom finom stratifikacijom materijala, srednjeg stepena zbijenosti sa karakterističnim i učestalim gradacionim prelazima ka šljunku.
- Srednjezrn i krupnozrn pesak, šljunak i prašinasta glina ($al\text{-}j^{p,š,g}$) – rečno jezerski sedimenti (sa Corbicula flumi-nalis) međusobno izpreplitani slabo sortirani sa čestim lateralnim gradacionim prelazima, a izrazito heterogeni po parametru otporno-deformabilnih i filtracionih karakteristika.

Kako je terenskim istraživanjima pouzdano utvrđeno da je oštećenje objekta nastalo usled provlažavanja sloja prašinastih glina i prašina, to se u tabeli 1 daju zbirni rezultati laboratorijskih ispitivanja samo za ovu sredinu [5].

Tabela 1. *Rezultati identifikaciono-klasifikacionih i deformabilno-otpornih karakteristika prašinastih glina*

Istražni rad	**Prirodna vlažnost**	**Plastičnost i konzistencija**				**(USCS)**
	***w* (%)**	**w_L (%)**	**w_P (%)**	**I_p**	**I_c**	
SJ-1	27.4 – 34.6	34.8-40.5	21.8-22.7	13.0-17.8	0.12-0.49	CL, CL/SF
IB-1	22.1 – 26.8	30.0-38.0	20.0-22.0	13.0-17.8	0.58-0.59	
Istražni rad	**Zapr. težina**	**Modul stišljivosti Mv (kN/m^2)**			**Čvrstoća smicanja**	
	γ_d (kN/m^3)	**50-100**	**100-200**	**200-400**	**$\varphi'(^0)$**	**c' (kPa)**
SJ-1	13.5-14.6	1850-2120	2810-3110	4150-6020	28	5
IB-1	15.1-16.0	3450-4450	4500-6250	5500-11000	19-22	15-40

Upoređujući rezultate fizičko-mehaničkih svojstava tla pre i posle provlažavanja (SJ-1 sa IB-1), vidi se da su vrednosti svih fizičko-mehaničkih svojstava tla znatno smanjene. Ovo je potvrđeno i na osnovu rezultata kartiranja istražne jame (SJ-1) jer je pouzdano utvrđeno da je temeljno podtlo provlaženo. Naime, prirodna vlažnost (w) i indeks konsistencije (I_c), pokazuju da se sloj provlaženih prašinastih glina ($al^{g,pr}$) nalazi u vrlo mekom konzistentnom stanju:

$$w_p\,(21.8 - 22.7\ \%) < w\,(27.4 - 34.6\ \%) < w_l\,(34.8 - 40.5\ \%)$$

$$0.12 < Ic < 0.49$$

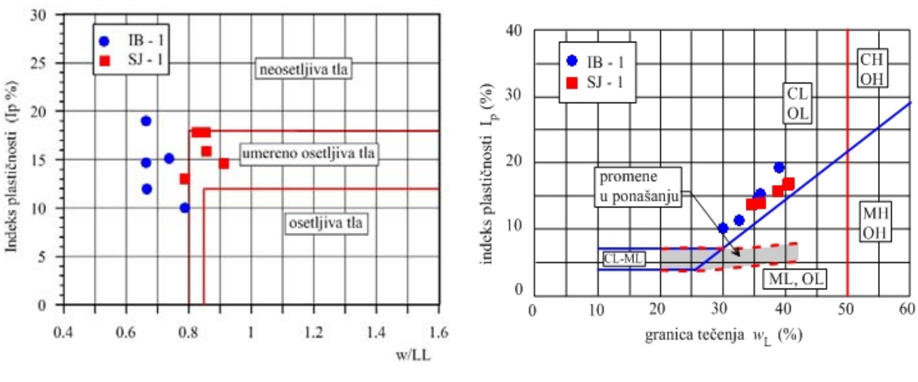
Saglasno tome i vrednosti zapreminske težine u suvom stanju (γ_d), efektivne kohezije (c') i modula stišljivosti (Mv), su izrazito niske:

$$\gamma_d = 13.50\ kN/m^3$$

$$c' = 5.0\ kN/m^2$$

$$Mv = 1850\ kN/m^2$$

Pored toga treba napomenuti da dobijene vrednosti granica konzistencije i odgovarajućih indeksnih pokazatelja, ukazuju da se prašinaste gline u provlaženoj zoni, mogu prema kriterijumu Braya i Sancia [6], [7], svrstati umereno osetljiva tla u pogledu opasnosti od pojave likvefakcije koji se odnosi na sitnozrne sredine.



Sl. 4. *Analiza procene opasnosti od pojave likvefakcije prema kriterijumima a) Braya i Sancia (2006) i b) Boulangera i Idrissa (2006)*

Dobijeni rezultati poslužili su za formiranje geotehničkog modela terena (GMT-1), koji reprezentuje temeljno podtlo dela objekta na kome je došlo do oštećenja. I po rezultatima opita statičke penetracije (CPT-1 i CPT-2), može se videti da postoje izvesna odstupanja u pogledu vrednosti q_c, a s obzirom da se ova istraživanja izvode direktno na terenu, može se reći da ona reprezentuju prirodne uslove koji vladaju u podtemeljnom tlu do dubine od 15 m. Rezultati dobijeni iz CPT-1 opita, iskorišćeni su za formiranje geotehničkog modela terena (GMT-2) koji reprezentuje zonu objekta u kojoj nisu vidljiva oštećenja. Merodavne vrednosti fizičko-mehaničkih parametara tla za analizirane geotehničke modele terena, prikazane su u tabeli 2.

Za ovako usvojene geotehničke modele terena izvršen je proračun dozvoljene nosivosti tla i sleganja temelja. Proračun dozvoljene nosivosti tla je izveden na osnovu "Pravilnika o tehničkim normativima za temeljnje građevinskih objekata", a prema modelu GMT-1 koji karakteriše provlaženu zonu tla. Rezultat proračuna je pokazao da je dozvoljena nosivost tla veća od stvarnog opterećenja jer iznosi:

$$Q_a = 164 \text{ kN/m}^2 > \Delta q = 130 \text{ kN/m}^2$$

Tabela 2. *Usvojeni geotehnički modeli terena GMT-1 i GTM-2*

GTM-1					
Litološki član	h (m)	φ' (^{0})	c' (kN/m^2)	Mv (kN/m^2)	γ (kN/m^3)
$al^{g,pr}$	2.75	27	5	2000	19
GTM-2					
Litološki član	**h (m)**	**d (m)**	**q_c (kN/m^2)**	**Mv (kN/m^2)**	**γ, γ' (kN/m^3)**
$al^{g,pr}$	2,5	2,5	3000-4000	11000	19
	4,5	2,0	1500-2000	4450	19
$al^{p,pr}$	10,0	5,5	6000-7000	15500	11
$al\text{-}j^{p,š,g}$	12,5	2,5	15000-22000	50000	11
	/	/	6000-10000	27000	11

Pošto ovaj rezultat nije pokazao da je došlo do sloma tla, izvršen je i proračun sleganja. Proračun sleganja je sproveden za oba geotehnička modela tla uključujući tako i zonu u kojoj je došlo do provlažavanja. Analiza sleganja je izvršena za karakterističnu tačku temelja, a za geotehnički model terena GMT-1 tj. za provlaženu zonu i oštećeni deo objekta, dobijene su računske vrednosti sleganja od:

$$(\text{GTM-1})\ \rho_1 = 3{,}8 \text{ cm}$$

Za geotehnički model terena (GMT-2), tj. za uslove koji vladaju u delu objekta gde nije došlo do oštećenja, ili bolje reći za uslove koji su vladali u temeljnom tlu nakon izgradnje objekta, računske vrednosti sleganja su reda veličine:

$$(\text{GTM-2})\ \rho_2 = 1{,}8 \text{ cm}$$

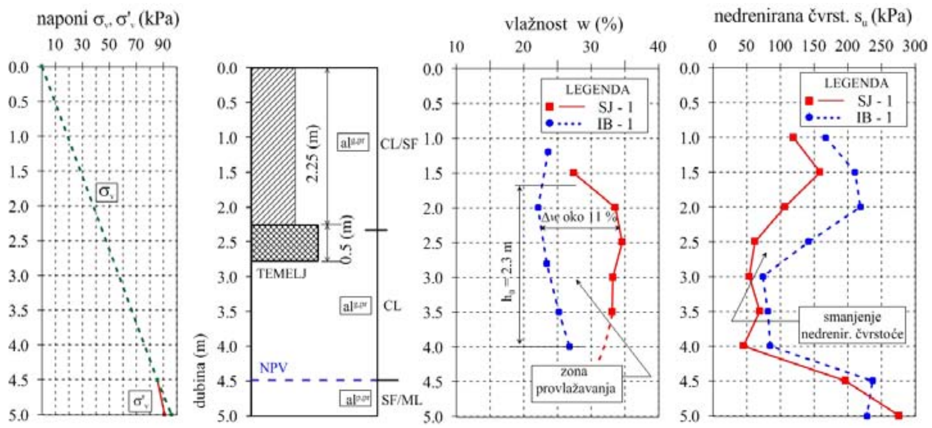
Ovi rezultati pokazuju da su sračunate vrednosti sleganja u dozvoljenim granicama ali da je naknadno sleganje usled provlažavanja izazvalo i pojavu diferencijalnog sleganja.

$$\Delta\rho = \rho_1 - \rho_2 = 2{,}0 \text{ cm}$$

4. DISKUSIJA

Rezultati veličina otpora konusa statičkih penetracija CPT-1 i CPT-2, ne pokazuju bitno odstupanje, što navodi na zaključak da je tlo u zoni CPT-2 u sličnom stanju kao tlo u zoni IB-1, tj. da u zoni CPT-2 nije došlo do provlažavanja tla. Na osnovu ovoga može se zaključiti da veličina zone uticaja promene vlažnosti u tlu u horizontalnom pravcu nije velika i da je ona za prašinaste gline ($al^{g,pr}$) manja od 2,0 m. Takođe, i razlika u računskim sleganjima temelja se javila na relativno malom rastojanju do 2,0 m što može da bude jedan od bitnih uzroka pojave diferencijalnog sleganja.

Međutim, pretpostavka je da razlika u sleganju od 2 cm, nije preterano velika da bi se sa sigurnošću moglo zaključiti da je to i jedini uzrok nastanka pukotina i oštećenja na objektu usled lokalnog provlažavanja tla. Zato se mora potražiti još neki od eventualnih uzroka oštećenja. Ukoliko se analiziraju rezultati promene vlažnosti tla sa dubinom, videlo bi se da je u zoni provlažavanja, vlažnost povećana u proseku za oko w = 11 %. Ovi rezultati promene vlažnosti tla sa dubinom, za oba geotehnička modela, prikazani su na slici 5 u vidu dijagrama. Iako je istražna jama (SJ-1) izvedena do dubine od 3,5 m, može se sa dijagrama zapaziti da se povećanje vlažnosti javlja i u vertikalnom pravcu za neku određenu dubinu h_0. U konkretnom slučaju prosečna vertikalna dubina promene vlažnosti iznosi oko h_0 = 2,3 m (od recimo 1,7 m do 4,0 m). Takođe se može uzeti u obzir i činjenica da je u zoni provlažavanja tla stepen zasićenja S_r = 100 %. Nedre-nirana čvrstoća smicanja određena je na osnovu rezultata opita statičke penetracije, korišćenjem teorijske zavisnosti u obliku $q_c = N_k\, s_u + \sigma_v$ (za N_k usvojena maksimalna vrednost od 25).



Sl. 5. *Primena fizičko-mehaničkih karakteristika u funkciji dubini*

Pošto ukupna zapremina tla (V) zavisi i od zapremine čvrstih čestica (V_s), možemo da definišemo specifičnu zapreminu v kao odnos V/V_s, odnosno

$$V = v \cdot V_S \quad \text{..................................(1)}$$

Ovo se može napisati i kao:

$$v = \frac{V}{V_S} = \frac{V_S + V_p}{V_S} = 1 + e \quad \text{...........(2)}$$

Pošto se zapremina čvrstih čestica ne menja, to promena ukupne zapremine (ΔV) zavisi od promene specifične zapremine (Δ*v*), pa se može napisati:

$$\Delta V = \Delta v \cdot V_s \quad \Rightarrow \quad \Delta v = \frac{\Delta V}{V_s} \text{ ....(3)}$$

Kako se radi o zasićenom tlu sa stepenom zasićenja S_r = 1, onda je na osnovu (2):

$$S_r = \frac{w \cdot G_s}{e} = \frac{w \cdot G_s}{v-1} \text{ ....(4)}$$

odnosno za S_r = 1

$$v = 1 + w \cdot G_s \text{ ....(5)}$$

Iz ovog sledi da promena specifične zapremine zavisi od promene vlažnosti tj.:

$$\Delta v = \Delta w \cdot G_s \text{ ....(6)}$$

Zamenom ove vrednosti u jednačini (3) dobija se promena ukupne zapremine:

$$\Delta V = V_s \cdot \Delta w \cdot G_s \text{ ....(7)}$$

Veza jednačina (1) i (5) definiše ukupnu zapreminu tj.:

$$V = (1 + w \cdot G_s) \cdot V_s \text{ ....(8)}$$

Ukoliko sada posmatramo tzv. blok tla jedinične površine ali do dubine h_0, ukupna zapremina je:

$$V = 1 \cdot h_0 = (1 + w \cdot G_s) \cdot V_s \text{ ....(9)}$$

ili

$$V_s = \frac{h_0}{(1 + w \cdot G_s)} \text{ ....(10)}$$

Zamenom ove vrednosti u (7) dobija se promena zapremine jediničnog bloka tla o dubine u kojoj je izražena promena vlažnosti:

$$\Delta V = \frac{h_0 \cdot \Delta w \cdot G_s}{1 + w \cdot G_s} \text{ ....(11)}$$

Ovako izračunata promena zapremine u vertikalnom pravcu u neku ruku predstavlja vertikalno izdizanje ρ (za slučaj smanjenja vlažnosti javilo bi se dopunsko sleganje). Međutim, kada se radi o glinenom tlu, poznato je da ono usled povećanja vlažnosti povećava i zapreminu (bubri). Zbog toga na ovako izračunatu prosečnu promenu zapremine u vertikalnom pravcu treba uzeti u obzir i bočno širenjene (za slučaj gubljenja vode, bočno skupljanje) pa je:

ΔV = ρ + *bočno širenje...............(12)*

što navodi na zaključak da je:

$$\rho < \Delta V$$

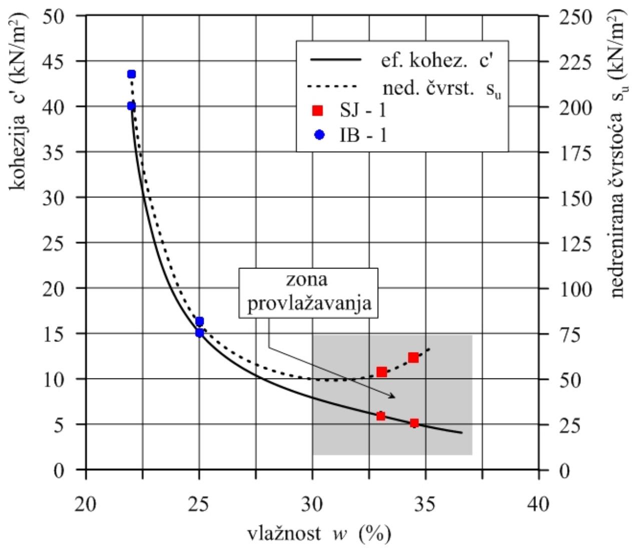
Na osnovu empirijskih rezultata, Driscoll [8] je predložio da se za bubrenje/sleganje usvoji redukovana vrednost promene zapremine od:

$$\rho \le \frac{\Delta V}{3} \quad do \quad \frac{\Delta V}{4}$$

Primenjujući ovo rešenje, dobijena je promena jedinične zapremine od ΔV = 35.7 cm, odnosno:

ρ *= 8.9 – 11.9 cm*

Kako je zidana konstrukcija od opeke u krečnom malteru i stara više desetina godina, može se zaključiti da se konsolidaciono sleganje završilo, pa ovako dobijene vrednosti, predstavljaju dopunsko izdizanje usled intenzivnog provlažavanja. Ono se desilo na kratkom rastojanju, pa objekat verovatno nije mogao da prihvati ovolike deformacije. Na to ukazuju i pukotine čija je širina u prizemnom delu veća. U svakom slučaju, glavni uzrok oštećenja objekta je svakako provlažavanje temeljnog tla, i to najvećim delom usled oštećenja olučnih sistema. Međutim, kada se radi o prašinastim materijalima sa provla-žavanjem je vrlo moguće i ispiranje finih čestica, što takodje dovodi do promene zapremine [9]. Ovaj proces promene fizičko-mehaničkih karakteristika tla usled provlažavanja, šematski je ilustrovan na slici 6.



Sl. 6. *Smanjenje čvrstoće smicanja tla ispod objekta usled provlažavanja*

5. ZAKLJUČAK

Na osnovu sprovedenih istraživanja i analiza može se zaključiti da se neplanirano provlažavanje tla dešava u određenoj zoni uticaja promene vlažnosti koja je recimo u horizontalnom pravcu za slučaj ispitivanih prašinastih glina ($al^{g,pr}$) relativno mala, L < 2.0 m. Kako je provlažavanje tla uglavnom lokalnog karaktera, to je i zona naknadnog sleganja lokalnog nivoa. Posledica ovoga je pojava diferencijalnih sleganja na vrlo kratkom rastojanju, što može da izazove manja ili veća oštećenja na objektu. Da bi se objekat vratio u eksploataciono stanje potrebno je izvršiti njegovu sanaciju koja mora obuhvatiti sanaciju temelja, sanaciju sistema za kontrolisano odvođenje površinskih i atmosferskih voda i sanaciju same konstrukcije. Sve ove mere sanacije zahtevaju znatno veća finansijska sredstva od onih koja su bila potrebna za izgradnju održavanje sistema koji treba da spreči provlažavanje temeljnog tla.

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ANALYSIS OF FOUNDATION SETTLEMENT FROM PROGRESSIVE MOISTENING OF SILTY CLAY****

Abstract

Examples of visible damage (cracks on the facade walls) on the shallow founded older buildings in Belgrade are not rare. Usually, this is a consequence of unequal settlement resulting from the unexpected moistening of soil under foundation. Moistening usually occurs locally and differential settlement is inevitable in such conditions. This paper presents the research results, which indicate that besides the size of impact zone of moisture change in the soil in horizontal direction and calculated settlement values, total change of soil volume under foundations should be analyzed.

Key words: *soil moistening, settlement, static penetration, volume change*

1. INTRODUCTION

Water is vital and the most active component of the soil because it is constantly in movement. Its presence in the soil depends on many factors, primarily on the available quantity (rainfall, wetting, leaking of drainage systems, etc.) and the rate of water penetration into ground (developed process of evapotranspiration, the presence of vegetation, the existence of natural and artificial drainage systems etc.). Less moisture changes occur during the year as the result of seasonal changes due to heavy rains, frequent temperature changes and similar. However, it has more influence on the ground during prolonged rainfall and long dry periods. The effect of climate changes becomes more important if the vegetation is present (eg. a particular tree species can exhaust per day over hundreds of liters of water on hot days), causing in a certain

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degree the soil shrinkage, which in turn can cause visible deformation on a facility. Contrary to this, the absence of vegetation leads to the increased humidity, often swelling of clay soils, which can sometimes cause elevation of the building. Therefore, different types of deformations may appear on building [1] (Figure 1). Please note that during prolonged dry periods, the tree roots can mainly cause mechanical damages of the underground parts of a facility due to their growth in deeper and wetter parts of terrain.

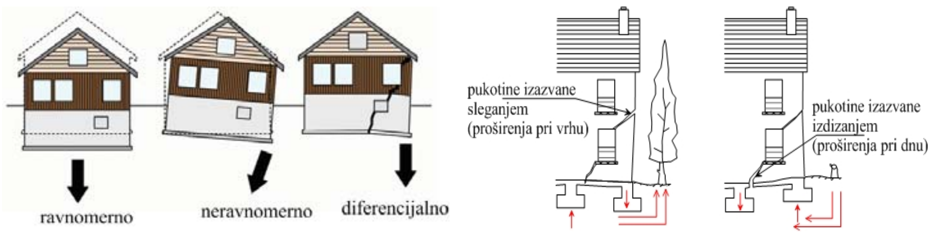

Fig. 1. *a) The general types of soil settlement, b) appearance of diagonal cracks in the walls due to differential settling*

The water movement, whether it is done under the influence of gravitational forces or non-gravitational forces (natural or anthropogenic), is in the closest interaction with the solid component, often changing the mechanical properties and therefore the physical condition, and behavior. At the beginning, the changes are light, almost imperceptible, but over time they can be with unpredictable scales leading often the facility in a condition that requires emergency intervention. Therefore, if these changes are expressed (e.g. under the foundations of buildings), the conditions are created for the occurrence of differential settlement in the basic structure, which can still cause great damages to the facility [2], [3]. However, the lack of specific conditions that exist in the soil after moistening leads to a design and implementation of technically and economically inadequate rehabilitation measures. In this case, it is important to know the size of influence zone of moisture changes, or the zone size in which there is a change of physical-mechanical properties of soil. The size of this zone is different for different types of soil. Therefore, this paper will present some research in determining the size of influence zone of moisture changes in silty clay.

2. REALIZED INVESTIGATIONS

The subject of this work is a ground-floor building near Surčin, which was built of massive walls in the constructive sense. The base of building is rectangular, size 15.6 x 9.6 m. The walls are of brick in a lime mortar, thickness d = 0.55 m. The roof structure is timber on two water with tile roof. The building has a basement that was dug into the ground 2.1 m, compared to zero above ground level. It was founded on the fundamental bands, width B = 0.9 m at funding height of 2.75 m. Contact stresses below the fundamental bands is $\Delta q = 130$ kN/m^2. Water drainage from the roof is done through gutters which run out the collected water on the surface so that further water drainage is not regulated. After long use of building (several decades), there were serious defects in places of vertical gutters in the form of progressive and developed cracks that threat its future exploitation [4].

In order to determine the physical and mechanical characteristics as well as the size of influence zone of moisture changes in the soil, the certain geomechanical testing was carried out in the places of gutter verticals where the damages are the greatest, i.e. the places with possible moistening of soil.

Conducted geotechnical investigations included the excavation of an exploration pit (NS-1) in the zone of greatest damage, i.e. assumed moistening zone, carrying out one exploration hole (IB-1) outside the zone of moistening, two static penetrations (CPT-1 and CPT-2) and geomechanical laboratory testing. The situation facilities with the exploration works is shown in Figure 2

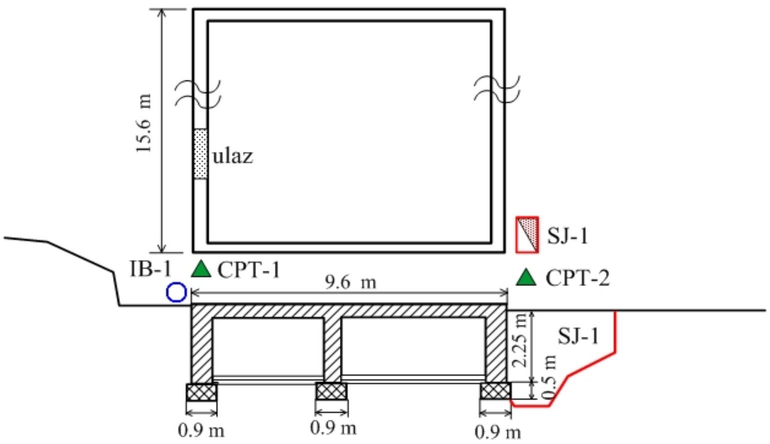

Fig. 2. *Review of facility founding with a position of exploration works*

Exploration (NS-1) was manually excavated to a depth of 3.5 m, i.e. about 0.7 m below the level of funding including the zone moistening After excavation the exploration pit, a check of foundation dimensions was carried out, then engineering mapping of soil and taking 5 undisturbed samples for geomechanical laboratory testing. In addition to the exploration pit, one exploration drill hole (IB-1) was made and two static penetration tests (CPT-1 and CPT-2).

Sounding depth of a field was 15 m.

The first static penetration (CPT-1) is derived next to the exploration drill hole in the IB-1 zone, which was not affected by moistening. The second static penetration (CPT-2), for technical reasons, was not performed inside the moistening zone, i.e. the place of exploration pit (NS-1), but at about 2.0 m from the facility. In order to determine the physic-mechanical properties of soil properties, the identification-classification experiments were carried out, the experiments with direct shear and oedometric compressibility experiments.

3. INVESTIGATION RESULTS

The results of static penetration sounding, engineering-geological mapping of exploration drill hole and exploration pit have shown that the ground is made of the following lithological members (Figure 3):

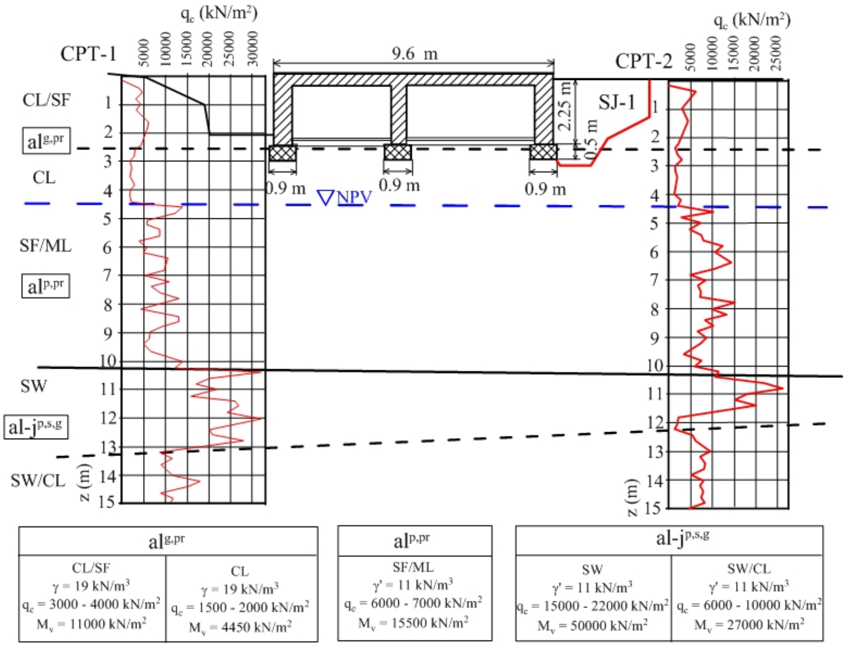

Fig. 3. *Geotechnical section of the ground*

- Silty clay, dust and dusty sand ($al^{g,pr}$), brown and gray in color, with significant organic impurities and occurrence of thin interlayers of sand; medium compressible; medium and poor water permeable. In the exploration pit (NS-1) from 2.0 m depth and further in the funding zone, the zone of moistening was found with distinctly very wet, soft, and more compressible silty clay.- Dusty sand ($al^{p,pr}$), fine grain to coarse grain and brown in color with an uneven share of fine-grained fraction and characteristic appearance of interlayers of mud with lot of organic detritus. There is a deposit with the expressed fine material stratification, medium degree of compaction with characteristic and frequent gradual transitions to gravel.
- Medium grain and coarse grain sand, gravel and silty clay ($al\text{-}j^{p,š,g}$) – river-lake sediments (with Corbicula fluminalis) interlaced, poorly sorted with often gradual lateral transitions and expressly heterogeneous per parameter of resistant-deformable and filtration properties.

As the field explorations have reliably established that the damage of facilities was caused by moistening the layer of silty clay and dust, Table 1 gives the summary results of laboratory tests only for this environment [5].

Table1. *Results of identification-classificationand deformation-resistant properties of silty clay*

Exploration work	Natural moisture	Plasticity and consistency				(USCS)
	w (%)	w_L (%)	w_P (%)	I_p	I_c	
NS-1	27.4 – 34.6	34.8-40.5	21.8-22.7	13.0-17.8	0.12-0.49	CL, CL/SF
IB-1	22.1 – 26.8	30.0-38.0	20.0-22.0	13.0-17.8	0.58-0.59	
Exploration work	**Volumetric weight**	**Compressibility module Mv (kN/m²)**			**Shear strength**	
	γ_d (kN/m³)	50-100	100-200	200-400	w_P'(°)	c' (kPa)
NS-1	13.5-14.6	1850-2120	2810-3110	4150-6020	28	5
IB-1	15.1-16.0	3450-4450	4500-6250	5500-11000	19-22	15-40

Comparing the results of physico-mechanical properties of the soil before and after moistening (NS-1 to IB-1), it is seen that the values of all physico-mechanical properties of the soil are significantly reduced. This was also confirmed on the basis of results of mapping the exploration pit (NS-1) because it is reliably established that the basic subsoil is moistened. Namely, the natural moisture (w) and consistency index (I_c) show that the layer of moistened silty clay ($al^{g,pr}$) is in a very soft consistent state:

$$w_p\ (21.8 - 22.7\ \%) < w\ (27.4 - 34.6\ \%) < w_1\ (34.8 - 40.5\ \%)$$

$$0.12 < Ic < 0.49$$

According to this, the value of gravity in a dry state (γ_d), effective cohesion (c') and modulus of compressibility (Mv), are extremely low:

$$\gamma_d = 13.50\ kN/m^3$$
$$c' = 5.0\ kN/m^2$$
$$M_v = 1850\ kN/m^2$$

In addition, it should be noted that the obtained values of consistency limits and corresponding index indicators, suggest that silty clay in the zone, according to the criteria of Bray and Sancio[6], [7], can be classified into moderately sensitive soils, in terms of danger of liquefaction occurrence, related the fine-grained environments.

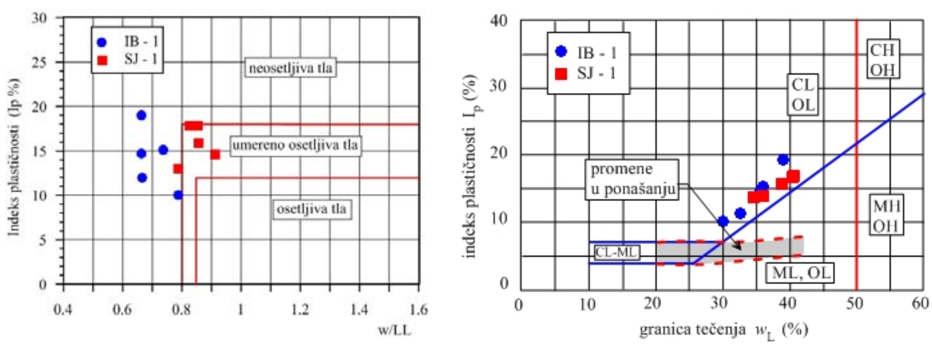

Fig. 4. *Analysis of risk assesment from liquefactionoaccurence according to the criteria a) Bray and Sancio (2006) and b) Boulanger and Idriss (2006)*

The obtained results were used to form a geotechnical model of field (GMT-1), which represents a fundamental subsoil of a facility part which was damaged. And the results of static penetration tests (CPT-1 and CPT-2) can show that there are some discrepancies regarding the q_c value of qc, and considering that these explorations are carried out directly in the field, it could be said that they represent the natural conditions prevailing in the sub-base soil to a depth of 15 m. The obtained results from CPT-1 experiment were used for formation the geotechnical field model (GMT-2) which represents the zone of facility in which the damages are not visible. Relevant values of physic-mechanical soil parameters for analyzed geotechnical field models are shown in Table 2.

For such adopted geotechnical field models, the calculation of allowable bearing capacity of soil and foundation settlement was done. Calculation of allowable bearing capacity is derived based on the "Rules of technical standards for construction facilities with foundation," and according to the model GMT-1, which characterizes the moistening zone of soil. The result of calculation showed that permitted bearing capacity of soil is greater than the actual load as follows:

$$Q_a = 164 \text{ kN/m}^2 > \Delta q = 130 \text{ kN/m}^2$$

Table 2. *The adopted geotechnical field models GMT-1 and GTM-2*

GTM-1					
Lithological member	**h (m)**	φ' (^{0})	**c' (kN/m^2)**	**Mv (kN/m^2)**	**(kN/m^3)**
$al^{g,pr}$	2.75	27	5	2000	19
GTM-2					
Lithological member	**h (m)**	**d (m)**	**q_c (kN/m^2)**	**Mv (kN/m^2)**	**γ, γ' (kN/m^3)**
$al^{g,pr}$	2.5	2.5	3000-4000	11000	19
	4.5	2.0	1500-2000	4450	19
$al^{p,pr}$	10.0	5.5	6000-7000	15500	11
$al\text{-}j^{p,š,g}$	12.5	2.5	15000-22000	50000	11
	/	/	6000-10000	27000	11

Since this result has not shown that there was a breakdown of the soil, a calculation of settlement was made. Calculation of settlement was carried out for both geotechnical models of soil including the zone of moistening. Analysis of settlement was carried out for the specific point of foundation, and the calculating value of settlement were obtained for a geotechnical field model GMT-1, i.e. for moistening zone and damaged part of the facility, as follows:

$$\text{(GTM-1) } \rho_1 = 3{,}8 \text{ cm}$$

For geotechnical field model (GMT-2), i.e. prevailing conditions in a part of the building where no damage has occurred, or rather the conditions that prevailed in the underlying soil after construction of the facility, the calculating values of settlement are as follows:

$$\text{(GTM-2) } \rho_2 = 1{,}8 \text{ cm}$$

These results show that the calculated values of settlement are within allowable limits, but that subsequent settlement, due to the moistening, has caused the occurrence of differential settlement.

$$\Delta\rho = \rho_1 - \rho_2 = 2{,}0 \text{ cm}$$

4. DISCUSSION

The results of cone resistance values of static penetrations CPT-1 and CPT-2, show no significant deviation, which suggests that the soil in zone of CPT-2 is in a similar state as the soil in zone IB-1, i.e. there was no moistening of soil in the CPT-2 zone. Based on this, it can be concluded that the size of the zone of influence of soil moisture changes in the horizontal direction is not great and that the influence zone size moisture changes in a horizontal direction is not large and it is less than 2.0 m for silty clay ($al^{g,pr}$). Also, the difference in computational soil settlements has occurred on a relatively small distance of 2.0 m, what can be one of important causes for differential settlement occurrence.

However, it is assumed that the difference in settlement of 2 cm is not too big to be concluded with certainty that this is the only cause of cracks and damages on a facility due to local soil moistening.

Therefore, some other possible causes of damage have to be found out. If the results of soil moisture changes with depth are analyzed, it will be seen that, in the moistening zone, the moisture is increased of about w = 11%. These results of soil moisture changes with depth, for both geotechnical models, are presented in Figure 5 in the form of diagrams. Although the exploration pit (NS-1) was made to a depth of 3.5 m, it can be observed from a diagram that increased soil moisture also appears in the vertical direction to the certain depth h_0. In this case, the average vertical depth of moisture changes is about h_0 = 2.3 m (from e.g. 1.7 m to 4.0 m). It can also be taken into account the fact that the zone of soil moistening the degree of saturation is S_r = 100%. The nondrained shear strength was determined on the basis of the results of static penetration tests, using the theoretical dependence in the form $q_c = N_k s_u + \sigma_v$ (for N_k the adopted maximum value of 25).

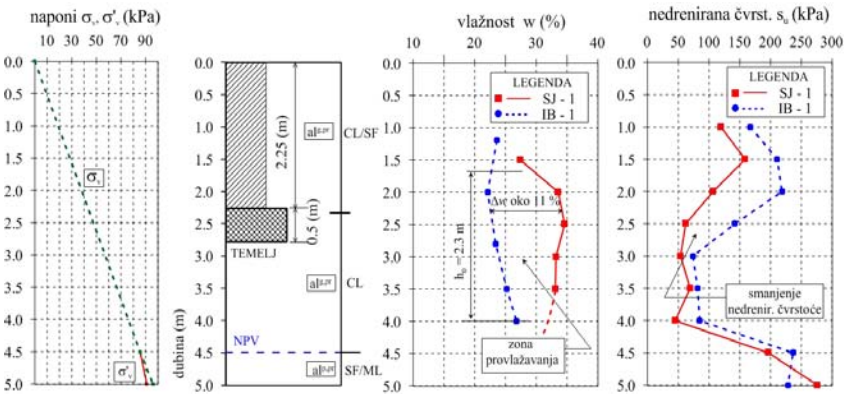

Fig.5. *The use of physico-mechanical properties in the function of depth*

Since total volume of soil (V) depends on the volume of solids (V_s), the specific volume v can be defined as the ratio V/V_s, that is:

$$V = v \cdot V_s \quad \text{.....................................(1)}$$

This can be written as

$$v = \frac{V}{V_S} = \frac{V_S + V_P}{V_S} = 1 + e \text{ ....(2)}$$

Since the volume of solid particles does not change, so the change of total volume (ΔV) depends on the specific volume change (Δ*v*), and it can be written

$$\Delta V = \Delta v \cdot V_S \quad \Rightarrow \quad \Delta v = \frac{\Delta V}{V_S} \text{ ......(3)}$$

Since this is a saturated soil with saturation degree $S_r = 1$, then based on (2)

$$S_r = \frac{w \cdot G_s}{e} = \frac{w \cdot G_s}{v - 1} \text{ ....(4)}$$

that is for $S_r = 1$

$$v = 1 + w \cdot G_s \text{ ....(5)}$$

From this it follows that the specific volume change depends on humidity change, i.e.

$$\Delta v = \Delta w \cdot G_s \text{ ....(6)}$$

Substitution of this value in equation (3) gives total volume change

$$\Delta V = V_s \cdot \Delta w \cdot G_s \text{ ....(7)}$$

The connection of equations (1) and (5) defines total volume, i.e.

$$V = (1 + w \cdot G_s) \cdot V_s \text{ ....(8)}$$

If so called block of the unit surface is observed, but to a depth h_0, total volume is:

$$V = 1 \cdot h_0 = (1 + w \cdot G_s) \cdot V_s \text{ ....(9)}$$

or

$$V_s = \frac{h_0}{(1 + w \cdot G_s)} \text{ ....(10)}$$

Substitution of this value in equation (7) gives the volume change of unit soil block to a depth where the change of humidity is expressed:

$$\Delta V = \frac{h_0 \cdot \Delta w \cdot G_s}{1 + w \cdot G_s} \text{ ....(11)}$$

Thus the calculated volume changes in the vertical direction in a way represents a vertical rise ρ (in the case of moisture reduction of moisture, the additional settlement will occure). However, when it is a clay soil, it is known that, due to the increased moisture, it also increases the volume (swelling). Because of this, the lateral spreading (in the sace of water loss, lateral shrinkage) should be taken into account on such calculated average volume change in the vertical direction, so

ΔV = *ρ + lateral spreading*.........(12)

Suggesting the conclusion that

$$\rho < \Delta V$$

Based on empirical results, Driscoll [8] suggested that the reduced value of volume change has to be adopted for swelling/settlement, as follows

$$\rho \le \frac{\Delta V}{3} \quad do \quad \frac{\Delta V}{4}$$

Applying this solution, the ochange of unit volune was obtained of $\Delta V = 35.7$ cm, that is

ρ = 8.9 – 11.9 cm

As the construction was built of a brick in lime mortar, and it is several decades old, it can be concluded that the consolidated settlement ended, and thus obtained values represent additional uplift due to intense moistening. It happened in a short distance, so the facility could not possibly accept such deformations. The cracks point out to it with a width in the ground floor is higher.In any case, the main cause of damage of the facility is certainly moistening the foundation soil, and mostly due to a damage of gutter systems. However, when it is a fact with dusty materials with moistening, the washing of very fine particles is possible, which also leads to the volume change [9]. This process of changes the physico-mechanical characteristics of the soil due to moistening, is schematically illustrated in Figure 6.

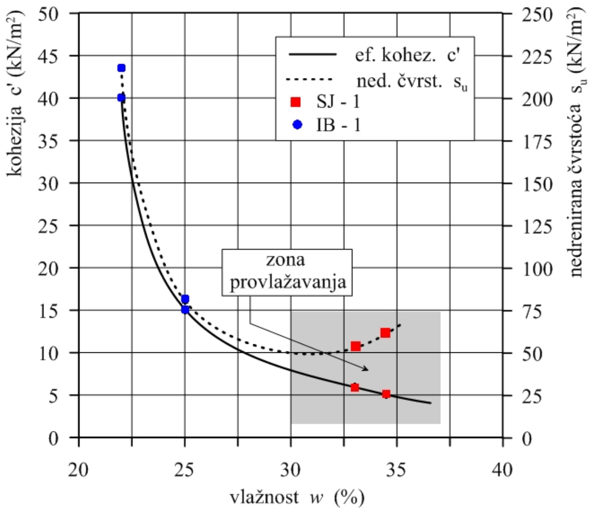

Fig. 6. *Reduction of shear strength of soil under facility due to moistening*

5. CONCLUSION

Based on the conducted investigations and analyses, it can be concluded that the unintended soil moistening occurs in a particular affected zone by changes in moisture, which is, for example in the horizontal direction, in the case of the tested silky clay ($al^{g,pr}$) relatively small, L < 2.0 m. As the soil moistening has mainly the local character, so the zone of subsequent settlement has the local level. The consequence of this is the appearance of differential settlement in a very short distance, which can cause minor or major damage on the facility. To return the facility in exploitation condition, it is necessary to make its rehabilitation, which must include the foundation repair, rehabilitation of system for controlled discharge of surface and storm water and rehabilitation of the structure. All these measures of rehabilitation require much higher financial resources than those needed to build and maintain a system that should prevent the moistening of foundation soil.

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