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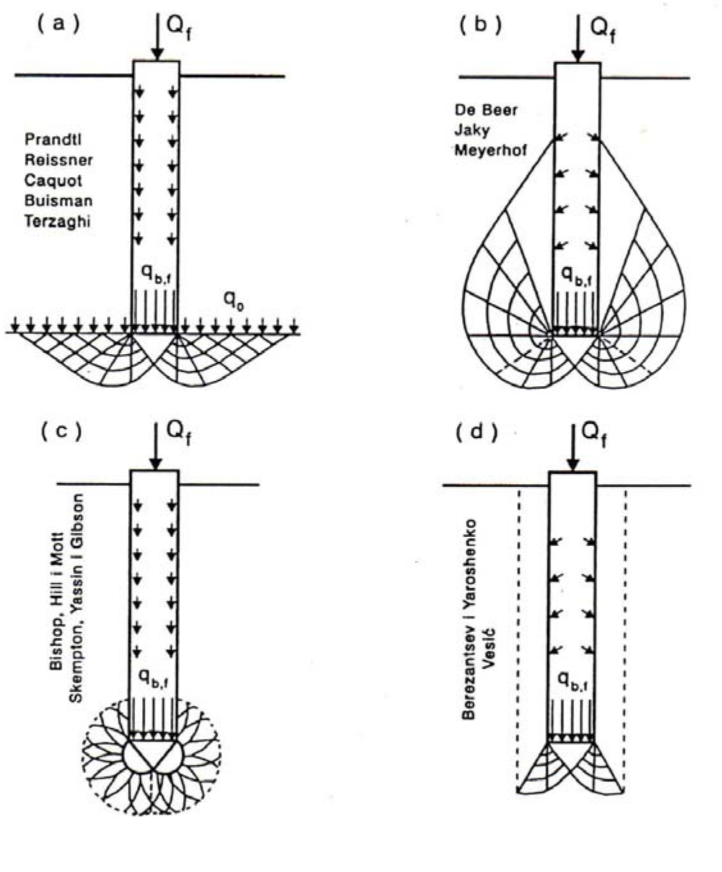
GRAĐEVINSKI MATERIJALI I KONSTRUKCIJE

1

BUILDING MATERIALS AND STRUCTURES

ČASOPIS ZA ISTRAŽIVANJA U OBLASTI MATERIJALA I KONSTRUKCIJA
JOURNAL FOR RESEARCH OF MATERIALS AND STRUCTURES



DRUŠTVO ZA ISPITIVANJE I ISTRAŽIVANJE MATERIJALA I KONSTRUKCIJA SRBIJE
SOCIETY FOR MATERIALS AND STRUCTURES TESTING OF SERBIA

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U V O D N I K

E D I T O R I A L

Ovaj broj časopisa posvećen je akademiku SANU profesoru Dušanu Miloviću, diplomiranom inženjeru građevine i redovnom profesoru Fakulteta tehničkih nauka Univerziteta u Novom Sadu u penziji.

Ove godine, Savez inženjera i tehničara Srbije slavi 150 godina postojanja i rada, u različitim uslovima i s različitim intenzitetom aktivnosti (videti sažet istorijat posle objavljenih radova). S tim u vezi, jedna od najvažnijih aktivnosti pojedinih članica i samog saveza jeste rad na planiranoj publikaciji „Znameniti inženjeri Srbije“. Reč je o svojevrsnom dugu jedne generacije prema stvaraocima u minulim periodima, pa je rukovodstvo Saveza građevinskih inženjera Srbije predložilo da se i jedan broj časopisa „Građevinski materijali i konstrukcije“, za sada vodećeg u oblasti građevinarstva u Srbiji, posveti jednom od vodećih srpskih naučnika u toj oblasti. Ovaj predlog je jednoglasno prihvatila Skupština Srpskog društva za mehaniku tla i geotehničko inženjerstvo.

Dušan Milović je naučnik sa izuzetno zapaženim rezultatima na osnovu kojih je doprineo afirmaciji i ugledu isprva bivše SFR Jugoslavije, a zatim i Srbije, u svetu. Njegovo svestrano angažovanje u oblasti mehanike tla i geotehničkog inženjerstva veoma je značajno, posebno zato što je praćeno laboratorijskim i terenskim geomehaničkim istraživanjima, na različitim lokacijama. Naročito se ističu njegova istraživanja lesa, koji je veoma osetljiv na uticaj vlage, s brojnim rezultatima i predlozima za fundiranje različitih konstrukcija na njemu, što je detaljnije navedeno u Biografiji i daljem tekstu uvodnika.

Nesumnjivo je da je izbor da se časopis u celosti posveti akademiku Dušanu Miloviću, proizašao iz vrednovanja rezultata koje je postigao u svojoj dugogodišnjoj karijeri, jer su njegovi dometi, kao plod višedecenijskog upornog rada, poznati, visoko cenjeni i priznati i kod nas i u svetu. S obzirom na to što je Dušan Milović od početka svoje interesovanje usmerio na oblast Mehanike tla i fundiranja, on pripada pionirima ove, relativno nove, naučne oblasti u našoj zemlji.

Uslovi rada u toj oblasti bili su veoma složeni i slojeviti, jer je sredinom dvadesetog stoleća ova disciplina bila mlada i tek se razvijala u Jugoslaviji i Srbiji, te nije bila dostupna u nastavi za nekoliko celih generacija posle Drugog svetskog rata. Moto profesora Milovića, na početku rada pripremljenog za publikovanje

This volume of the Journal is dedicated to Professor Dusan Milovic, Ph.D. in civil engineering, member of the SANU and full professor of the Faculty of Technical Sciences at the University of Novi Sad in retirement.

This year, the Union of Engineers and Technicians of Serbia celebrates 150 years of existence and work, under different conditions and intensity of activity (see the concise history after the published works). One of the most important activities of individual members and the General Union in this year is working on the future publication named *Famous Engineers of Serbia*. It is considered as responsibility of this generation towards the creators living in the past periods, so in addition to the above publication, the leadership of the Union of Civil Engineers of Serbia decided to dedicate one volume of the journal *Building Materials and Structures*, which is now the leading magazine in the field of civil engineering in Serbia, to one of the leading Serbian scientists in the field. This decision was unanimously adopted by the Assembly of the Serbian Society for Soil Mechanics and Geotechnical Engineering.

Dusan Milovic is a scientist with extremely notable results that contributed to the affirmation and reputation of both the former SFR Yugoslavia and Serbia in the world. His comprehensive engagement in the field of soil mechanics and geotechnical engineering is very important, especially because it was accompanied by laboratory and field geomechanical research at various locations. His research of loess, which is very sensitive to the influence of moisture, is particularly important with a number of results and proposals for founding various structures on it, which is detailed in the Biography and the further text of this editorial.

Undoubtedly, the choice to devote the entire journal to the academician Dusan Milovic was based on the evaluation of the results he has achieved during the years of his career, because his achievements, resulting from several decades of persistent work, are highly valued and recognized both in our country and abroad. Given that from the very beginning Professor Milovic has focused his attention on the field of soil mechanics, he belongs to the pioneers in this relatively new scientific field in our country.

Working conditions in this field were extremely difficult, because in the middle of the 20^{th} century the Soil Mechanics was the youngest branch in Civil Engineering u ovom broju časopisa, podseća nas na upozorenja čuvenog Terzagija, tvorca nove naučne discipline Mehanike tla i fundiranja, koja se proučava i koristi radi uspešnog građenja svih građevinskih objekata u svetu: „Temelji građevina uvek su bili pastorčad zato što nema slave u temeljenju. Ali dela osvete zbog nedovoljne pažnje oko njih mogu biti katastrofalna“. Odabrani moto pokazuje entuzijazam i veru koji su profesora Milovića podsticali da desetinama godina posvećeno radi u ovoj oblasti. Pre svega se to odnosi na uspešno rešavanje aktuelnih problema u građevinskoj praksi čija je kompleksnost zahtevala teorijska rešenja vrednovana i verifikovana eksperimentalnim istraživanjima.

Dušan Milović je svoje aktivnosti posebno usmerio na naučno-istraživački rad. On je u svojoj višedecenijskoj karijeri vodio šesnaest istraživačkih projekata za Fond za nauku Srbije i Vojvodine, ali i istraživanja za Nacionalni fond Kanade i nacionalne naučne fondacije SAD, te za Fond za nauku SANU.

Profesor Milović je u teorijskim radovima, koristeći duple Furijeove redove, metodu konačnih i graničnih elemenata i konačnih razlika, formulisao brojna originalna teorijska rešenja. Ona predstavljaju najvažnija dostignuća njegovog istraživačkog rada, koja su dostupna pošto je publikovao petnaest monografija i udžbenika, 137 članaka u časopisima u bivšoj Jugoslaviji i Srbiji i zbornicima radova s brojnih kongresa, 73 članka u međunarodnim časopisima i kongresnim publikacijama, s preko 3500 stranica. Ovi članci citirani su 194 puta do 2007. godine u 18 zamalja (Sci citation index, u knjigama u regionu i u doktoratima u SAD i Kanadi).

Probleme plitkih temelja Dušan Milović je široko razmatrao uz primenu teorije elastičnosti. Njegova rešenja komponentalnih napona i deformacija dobijena su metodom konačnih elemenata. Rešenja koja obuhvataju različite oblike i relativne krutosti temelja, i za različita opterećenja i veoma složene modele tla, uključivši višeslojne sisteme, anizotropnih svojstava, ograničene debljine stišljivih slojeva, procenjena su kao pionirski radovi. Radovi saopšteni na Međunarodnim kongresima u Londonu (1957), Parizu (1961), Visbadenu (1963) i Parizu (1963) prvi su radovi iz Srbije u toj oblasti. Njegova originalna rešenja dobijena metodom konačnih elemenata (MKE), šest radova u zemlji (u periodu 1971– 1974) i u međunarodnim naučnim časopisima (osam radova u Londonu, Parizu, Berlinu i Moskvi) ubrajaju se među prve s rezultatima dobijenim metodom konačnih elemenata u mehanici tla. Posebnu vrednost predstavlja knjiga „Naponi i deformacije plitkih temelja“, jedina knjiga srpskog autora štampana u Roterdamu (Elsevier), u kojoj su data teorijska rešenja iz mehanike tla.

Tokom boravka u Kanadi, Dušan Milović je posebnu pažnju posvetio proučavanju nestabilnih, takozvanih Leda glina u Kvebeku. Bitno svojstvo ove vrste glina jeste potpuni gubitak smičuće otpornosti pod cikličkim opterećenjem i vibracijama. U ovim slučajevima, događaju se pokreti, klizanja i propadanje tla, što ugrožava stabilnost konstrukcija. Radi boljeg razumevanja ponašanja ovih glina, obavljeno je mnoštvo terenskih i laboratorijskih testova. Zapaženo je da mehanički poremećaji osetljivih glina bitno utiču na preciznost rezultata. Zato je Milović uveo test statičke penetracije u inženjersku praksu radi dobijanja rezultata

in Yugoslavia and Serbia and this subject was not yet included in the regular study programme for several post-war generations.

The motto of Professor Milovic, at the beginning of the paper prepared for publication in this volume, reminds us of the warnings of the famous Terzaghi, who is considered to be the creator of the new scientific discipline of soil mechanics and foundation engineering, which is being studied and used in the construction of all buildings in the world: "Building foundations have always been treated as step children because there is no glory attached to the foundations, but their acts of revenge for the lack of attention can be very embarrassing," and shows his enthusiasm and faith that prompted him to work tens of years in this field.

Therefore, in order to be able to solve the current problems in civil engineering practice, and the complexity of these problems, it was essential to develop the theoretical solutions and verify the validity of these solutions by means of the experimental investigations.

Dusan Milovic directed his activity towards scientific and research work. He has been the leader and principal investigator of 16 research projects, financed by the Fund for scientific work of Serbia, SIZ for scientific work of Vojvodina, National Research of Canada, American National Science Foundation and Fund for research work of the Academy of Sciences and Arts.

In his theoretical studies he used double Fourier’s series, power series method, finite and boundary element method, finite difference method and finite element method. Numerous original theoretical solutions represent one of the most important achievements in his research works. He published 15 monographs and textbooks, 137 papers in Yugoslav and Serbian journals and congress volumes, 73 in international journals and congress proceedings with over 3500 pages. These papers have been cited 194 times until 2007 in 18 countries (Science Citation Index, textbooks in foreign countries and in doctoral thesis in USA and Canada).

In the field of shallow foundations he considerably broadens the application of the Theory of Elasticity. His solutions obtained by means of the finite element method for calculation of componential stresses and displacements for various shapes and any relative stiffness of foundations, for any type of loading and very complex soil models, including multilayer systems, anisotropic properties, limited thickness of the compressible layers have been estimated as pioneer works. Papers presented at international congresses in London (1957), Paris (1961), Wiesbaden (1963) and Paris (1963) are the first Serbian papers. In addition, his original solutions, obtained by the finite element method, published in the country (6 papers over the period from 1971 to 1974), and in the international scientific journals (8 papers in London, Paris, Berlin and Moscow over the period from 1970 to 1973) are among first with the solutions obtained by finite element method in the field of Soil Mechanics. It is also worth mentioning that his book "Stresses and displacements for shallow foundations" is the only Serbian book published in English (Ed. Elsevier), in which are given the theoretical solutions related to Soil Mechanics.

During his stay in Canada, Dusan Milovic paid particular attention to the investigation of sensitive Leda clay in Quėbec. The essential property of this kind of za glinu u prirodnim uslovima.

Sa zadovoljstvom ističem da su se svi autori kojima sam se obratio da učestvuju svojim radovima u ovom broju časopisa, veoma rado odazvali i svojim doprinosima pokazali koliko cene ličnost i rad profesora Milovića. Ovaj stav potkrepljujem rečima profesora Henza Brandl-a, koji mi je napisao: „It gives me great honour to publish in a volume that is dedicated to Prof. Dusan Milovic. I knew him personally, because between 1968 and 2015 I was the official representative of Austria in the Council Meeting of the ISSMGE (International Society for Soil Mechanics and Geotechnical Engineering)”.

Veliki uspesi, ponekad, mogu da izazovu i odbojnost u okruženju, pa je tako akademik Milović bio povređen činjenicom da je u Katalogu nauka i tehnika – Realizovana rešenja članova Odeljenja tehničkih nauka SANU 1841–2016, umesto adekvatnog predstavljanja njegovih rezultata i dostignuća, bio potpuno izostavljen. U vezi s tim nemilim događajem, akademik Milović mi je napisao: „Dugo sam oklevao da spominjem ona događanja ili bolje rečeno pohvale koje su bile upućene na moj rad i moje uspehe. Mislim da nije neumesno, jer imamo ružne primere onih koji sami sebe veličaju bez ikakve istinite osnove, pa ne vidim da nemam moralnog prava da spomenem samo ono što su drugi rekli o meni.“ Zbog toga smatram da je od interesa za stručnu javnost da ovde iznesem šta su izuzetne ličnosti u oblasti građevinarstva rekle o radovima Dušana Milovića.

Akademik Đorđe Lazarević svojevremeno je uputio dr Miloviću, autoru monografije „Analiza napona i deformacija u Mehanici tla” svoj stav da smatra korisnim savet da se ona štampa na našem i jednom od svetskih tehničkih jezika. Iz monografije se inače ne bi mogla ni izbliza izvući ona korist koja bi bila u skladu sa autorovim doprinosom postupka konačnih elemenata u obradi novih rešenja. Dr Milović je nastavio da daje priloge teoriji elastičnosti, koje je počeo još Boussinesq – s naponima i deformacijama elastičnih polu-prostora.

Za isto delo štampano na srpskom i engleskom jeziku, **Arpad Kezdi**, član Mađarske akademije nauka, profesor Univerziteta u Budimpešti, navodi: „Autor ovog značajnog naučnog dela ’Stresses and Displacements in Soil Mechanics’ u kome se obrađuje primena Teorije elastičnosti pri proračunu napona i deformacija ispod temelja, uvodeći u razmatranje i aelotropski poluprostor, dao je rešenje za mnoge slučajeve opterećenja, koji se javljaju u inženjerskoj praksi. Na taj način, autor je riznicom podataka koji se ne mogu naći u udžbenicima**,** znatno proširio polje primene Teorije elastičnosti.“

Još prilikom odbrane doktorske disertacije Dušana Milovića, **Milan Luković**, član SANU, profesor na Rudarsko-geološkom fakultetu u Beogradu, izjavio je u svojstvu predsednika komisije za odbranu doktorske teze: „Vi ste bez sumnje najbolji poznavalac lesa i problematike fundiranja na njemu u čitavoj zemlji“.

Akademik Božidar Vujanović, profesor Fakulteta tehničkih nauka u Novom Sadu rekao je: „Od srca Vam čestitam na vrlo impresivnim podacima, koji pokazuju koliko ste truda, energije i volje uložili u stvaralački i originalni rad, koji zaslužuje svako poštovanje. Ja se veoma dobro sećam Vaše monografije štampane na engleskom jeziku, a Vaši inženjerski naučni radovi proneli su ugled jugoslovenske i svetske nauke i

clay is the complete loss of shear strength under the influence of cyclic loading and vibrations. In these cases movements and sliding of soil occur and endanger the stability of structures. In order to better understand the behaviour of these clays numerous field and laboratory rests have been performed. It has been also observed that the mechanical disturbance of sensitive clays has a considerable influence on the precision of the obtained results. Therefore, he introduced the static penetration test in engineering practice in order to get the results for clay in the natural state.

It is my pleasure to point out that all authors whom I asked to participate with their papers in this volume were very happy to contribute, showing they respect to the personality and work of Professor Milovic. This position I support by the words of Professor Hens Brandl, who wrote to me: "It gives me great honour to publish in a volume that is dedicated to Professor Dušan Milovic. I knew him personally, because between 1968 and 2015 I was the official representative of Austria in the Council Meeting of the ISSMGE (International Society for Soil Mechanics and Geotechnical Engineering)."

Great success sometimes can provoke reverence in the environment, so Professor Milovic was hurt by the fact that, instead of adequately presenting his results and achievements, he was completely omitted in the Catalogue of Science and Technology - Realized solutions of Members of the Department of Technical Sciences of the SANU 1841-2016. In connection with this unwelcome event, Professor Milovic wrote to me: "I have long hesitated to mention those events, more specifically praises that were addressed to my work and my successes. I think it is inappropriate, because we have ugly examples of those who glorify themselves without any real basis, so I think that I have moral right to mention only what others have said about me." For this reason, I believe that it is of interest of professional community to cite here what other important individuals in the field of civil engineering have said about the work of Dusan Milovic.

Djordje Lazarevic, member of the SANU, once expressed his opinion to Dr. Milovic, the author of the monograph "Analysis of Stresses and Displacements in Soil Mechanics", that it is useful for the Council to print the monograph in Serbian and in one of the world's technical languages as well. Otherwise, it will be impossible to derive the benefit from the monograph which is in accordance with the author's contribution to the process of finite element method in processing of new solutions. Dr. Milovic continued his contributions to the theory of elasticity, which began with Boussinesq about the stresses and strains of elastic half-spaces.

Arpad Kezdi, a professor at the University of Budapest and member of the Hungarian Academy of Sciences, wrote about the same paper printed in Serbian and English: The author of this important scientific paper "Stresses and Displacements in Soil Mechanics", in which he analyzes the application of theory of elasticity for the calculation of the stresses and displacements below the foundations, introducing the aelotropic half-space into consideration, has provided a solution to many cases of loading that occur in engineering practice. In this way, offering a repository of data that cannot be found in textbooks, the author significantly expanded the field of application of theory of elasticity.

zaslužuju najveće priznanje i poštovanje.“

Akademik Đorđe Zloković, profesor na Arhitektonskom fakultetu u Beogradu: „Vaša impresivna bibliografija zadivljuje i obimom i kvalitetom. Vaš opus Vas stavlja u vrh svetske nauke“.

Jedan od vodećih naučnika u svetu **Harry Poulos**, profesor Univerziteta u Sidneju, Australija, ističe: „Radovi prof. Milovića, u kojima su prikazana rešenja metodom konačnih elemenata jesu pionirski, jer su u to vreme oni bili retkost u geotehnici. Njegova izvanredna knjiga, koju je objavio izdavač Elsevier iz Holandije, poslužila bi mi često pri rešavanju raznih problema. Iz izvanrednih radova prof. Milovića, prikazanih tokom više godina, prof. Davis i ja smo neke od rezultata uvrstili u našu monografiju o naponima i pomeranjima.

Alan Lutenegger, profesor Univerziteta u Masačusetsu, Amherst, Sjedinjene Države, navodi: „Izvanredni radovi prof. Milovića o lesu sadrže takve podatke kakvi još nigde u svetu nisu do sada objavljeni“.

P. Habib, profesor Politehničke škole u Parizu: „Eksperimentalni radovi prof. Milovića predstavljaju izvanrednu proveru teorijskih rešenja u Mehanici tla.“

Gaston Denis, dekan Građevinskog fakulteta Šerbruk u Kanadi rekao je: „Za mene je laka i prijatna dužnost da izrazim moje najdublje poštovanje kako za vrednost dr Dušana Milovića kao naučnika, tako i za njegovu kompetenciju kao predavača i njegovu odanost kao saradnika. Dr Miloviću smo poverili zadatak da organizuje Odeljenje za Mehaniku tla na našem građevinskom odseku, i da njime upravlja. Taj zadatak obavio je do te mere briljantno, da se samo posle tri godine Univerzitet u Šerbruku mogao ponositi laboratorijom za naučna istraživanja koja je priznata kao jedan od centara izvrsnosti u Kanadi u domenu Mehanike tla i fundiranja.“

Prof. Milović uživao je velik ugled i lično je dobio 110 000 dolara kao sredstva za naučni rad od Nacionalnog saveta Kanade i Ministarstva za obrazovanje provincije Kvebek. Rezultati njegovih naučnoistraživačkih radova omogućili su mu da publikuje četrnaest članaka u časopisima i da prikaže šest radova na internacionalnim kongresima. Vredno je pomena da je odajući priznanje dr Miloviću za kvalitet rada i za veliku reputaciju koju je stekao u Kanadi i u inostranstvu, Univerzitet u Šerbruku ubrzanim promocijama njemu dodelio zvanje vanrednog profesora, a 1969. godine najviše zvanje – redovnog profesora. Profesor **Claude Hamel,** na Građevinskom fakultetu Univerziteta Šerbruk izjavio je „da pored toga što je naučnik velike vrednosti, dr Milović je i najprijatniji saradnik. Njegovi studenti veoma ga poštuju i njegove kolege duboko ga uvažavaju. U mnogobrojnim kontaktima koje smo imali, uvek sam se uveravao u njegovu izvanrednu ljubaznost i neumornu predanost, njegov marljiv i metodičan radni elan**.** Milovićeva međunarodna reputacija u oblasti Mehanike tla, i više publikovanih radova za vreme njegovog boravka u Šerbruku, doneli su našem fakultetu izuzetan ugled u ovom domenu. Dr Milovića su priznali kao izvanredanog profesora, kako studenti na nivou redovnih studija, tako i oni na nivou magistrature i doktorata.“

Ovde ću, sa zadovoljstvom, navesti ono što za sve nas koji smo upoznati s njegovim rezultatima i dometima, predstavlja najveći uspeh akademika Milovića, a koji se može potvrditi neoborivim dokazima:

1. Proširio je primenu teorije elastičnosti na

During the defence of the doctoral thesis of Dusan Milovic, **Milan Lukovic**, member of the SANU, professor at the Faculty of Mining and Geology in Belgrade, stated as the President of the Commission for the defence of the doctoral thesis: "You are undoubtedly the best connoisseur of loess and the issues of founding on it in the whole country".

Bozidar Vujanovic, member of the SANU and professor at the Faculty of Technical Sciences in Novi Sad, said: "I congratulate you on the very impressive data that show how much effort, energy and commitment you invested in your creative and original work, which deserves all respect. I remember very well your monograph printed in English, and your engineering research papers have made the Yugoslav science globally acknowledged and deserve the utmost recognition and respect."

Djordje Zlokovic, member of the SANU and professor at the Faculty of Architecture in Belgrade: "Your impressive bibliography is amazing both in scope and quality. Your opus brings you to the top of the world science."

One of the world's leading scientist **Harry Poulos**, professor at the University of Sydney (Australia) declares that "Professor Milovic's papers which present solutions obtained based on the finite element method are pioneering because at that time they were rare in geotechnics. His extraordinary book, published by Elsevier from the Netherlands, has often served me to solve a variety of problems. From extraordinary papers presented by Professor Milovic over the years, Professor Davis and I have included some of the results in our monograph on stresses and displacements."

Alan Lutenegger, professor at Massachusetts University, Amherst (USA) states that "The extraordinary works of Professor Milovic about loess contains data which have not yet been published in the world."

P. Habib, professor at the Polytechnic School in Paris declares that "Experimental works of Professor Milovic represent an extraordinary test for theoretical solutions in soil mechanics."

Gaston Denis, dean of the Sherbrooke School of Engineering in Canada said that "It is an easy and pleasant duty for me to express my deepest respect for Dr. Dusan Milovic as a scientist, as well as his competence as a lecturer, and loyalty as an associate. We entrusted Dr. Milovic with the task of organizing and managing the Department of Soil Mechanics at our civil engineering section, a task which he carried out in a brilliant way to the extent that only after 3 years, the University of Sherbrooke could have been proud of having a scientific research laboratory recognized as one of the Centers of Excellence in Canada in the field of Soil Mechanics and Funding.

Professor Milovic enjoyed a great reputation, and from the National Council of Canada and the Ministry of Education of the Province of Quebec he personally received $ 110,000 as a funding for the scientific work. The results of his scientific research enabled him to publish 14 papers in journals and present 6 papers at international congresses. It is worth mentioning that in recognition to Dr. Milovic's work and his great reputation in both Canada and abroad, the University of Sherbrooke, based on accelerated promotions, promoted him to the position of associate professor, and in 1969 he rešavanju problema u oblasti Mehanike tla i fundiranja;

2. Rešenja prikazana metodom konačnih elemenata i metodom Furijerovih dvostrukih redova smatraju se pionirskim (radovi objavljeni u Parizu, Berlinu, Londonu, Moskvi i Tokiju, u periodu od 1970. do 1973. godine);

3. Radovi provere teorijskih rešenja u Mehanici tla smatraju se izvanrednim;

4. Pionirski, izuzetni radovi o lesu sadrže takve podatke kakvi još nigde u svetu do sada nisu objavljeni;

5. Na svetskim kongresima za Mehaniku tla i fundiranje zapaženo je njegovih dvanaest radova: London, Pariz, Montreal, Meksiko Siti, Moskva, Tokio, Stokholm, San Francisko, Rio de Žaneiro, Hamburg, Osaka i Čikago.

6. Na svetskim kongresima za inženjersku geologiju prikazana su tri rada u Buenos Ajresu, Lisabonu i Vankuveru.

7. Na evropskim kongresima, internacionalnim regionalnim i dunavskim kongresima za Mehaniku tla i fundiranje prikazano je 27 radova.

8. Radovi akademika Milovića citirani su 205 puta (SCI) do 2009. godine.

9. Na internacionalnim kongresima u periodu od 1965. do 2014. godine bio je deset puta pozivan od Instituta za mehaniku tla, da u svojstvu člana panela održi predavanje po pozivu (invited speaker), da bude generalni izvestilac, potpredsednik sekcije za kolapsibilna tla, predsednik sekcije za makro-porozna tla, a na poziv Instituta za Mehaniku tla Kineske akademije nauka pripremio je Key Paper za Internacionalni kongres za Mehaniku tla i fundiranje u Vuhanu 2012. godine. Takođe, organizator Internacionalne Konferencije Geo SIN 2014. godine u Singapuru, poziva ga da pripremi Key Paper i da organizuje jednu sekciju po sopstvenom izboru.

Iz navedenih podataka može se zaključiti da energija i entuzijazam, svojstveni samo retkim stvaraocima, ne napuštaju akademika Milovića, te da, iako u poodmaklim godinama, daje zapažene doprinose nauci i struci. To još jednom potvrđuje i člankom koji je napisan za ovaj broj časopisa. Iz njega izviru bogato iskustvo i originalne ideje pretočene u predloženi proračunski model koji doprinosi realnijoj proceni nosivosti šipova. Sve to dokazuje da je izbor akademika Milovića kao prve ličnosti kojoj se posvećuje ceo broj časopisa – u potpunosti opravdan.

Svojim delovanjem uvek je izlazio iz uskih okvira oblasti Geotehničkog inženjerstva. To mu je omogućila široka kultura kakva dolikuje velikanima, pošto je pored izvanrednog poznavanja struke i nauke i svetskih jezika, pratio i druge oblasti, naročito konstrukterska ostvarenja. Mislio je i o drugima i pratio njihove domete s radošću i s divljenjem je govorio i pisao o dipl. inž. Iliji Stojadinoviću, projektantu mostova velikih raspona, i mosta Krk–Sv. Marko–Kopno, koji je dugo bio svetski rekord po ostvarenim rasponima. Izbegavao je intervjue novinarima, jer je smatrao da oni prihvataju mnoge izjave bez argumentacije, čemu se protivio i tražio je u svemu utemeljenost u činjenicama, a ne u frazi „Stručna javnost, to sam ja“.

Uvodničar je nekoliko godina radio sa akademikom Dušanom Milovićem u istoj instituciji iz koje je on 1992. godine otišao u zasluženu penziju. Naši kontakti nisu prekinuti ni posle njegovog preseljenja u Kanadu. Plod tih kontakata jeste objavljivanje većeg broja radova koje

was awarded the title of full professor. **Claude Hamel**, professor at the University of Sherbrooke's Faculty of Civil Engineering said that "in addition to being a great scientist, Dr. Milovic is a remarkable associate. He is greatly respected by his students and deeply appreciated by his colleagues. The many contacts we had repeatedly convinced me of his extraordinary kindness, tireless commitment, and diligent and methodical workmanship. His international reputation in the field of soil mechanics and a number of published papers during his stay in Sherbrooke provided our faculty with outstanding reputation in this domain. Dr. Milovic is recognized as an excellent professor by both graduate students and students at master and doctoral studies."

I am pleased to state here the greatest achievements of academician Milovic for all of us who are familiar with his results and achievements that can be confirmed with indelible evidences:

1. He expanded the application of theory of elasticity to solving problems in the field soil mechanics and foundation engineering;

2. Solutions presented by the finite element method and the Fourier double series method are considered pioneering (papers published in Paris, Berlin, London, Moscow and Tokyo over the period from 1970 to 1973);

3. Papers aimed at verifying the theoretical solutions in soil mechanics are considered extraordinary;

4. Pioneering and extraordinary papers on loess contain data that were previously not published in the world;

5. Twelve of his papers were recognized at World Conferences for Soil Mechanics and Funding: London, Paris, Montreal, Mexico City, Moscow, Tokyo, Stockholm, San Francisco, Rio de Janeiro, Hamburg, Osaka and Chicago.

6. Three of his papers were presented at International Conferences on Engineering Geology: Buenos Aires, Lisbon and Vancouver.

7. Twenty seven of his papers were presented at European, international, regional and Danube conferences on soil mechanics and foundation engineering.

8. By 2009, Professor Milovic's papers were quoted 205 times (SCI).

9. At international conferences in the period from 1965 to 2014, he was invited 10 times by the Institute of Soil Mechanics to speak as invited speaker and a panel member, to be a general reporter, vice president of the section for collapsible soils, president of the section for macro-porous soils, and upon the invitation of the Institute of Soil Mechanics of Chinese Academy of Sciences he prepared the Key Paper for the International Conference on Soil Mechanics and Foundation Engineering in Wuhan in 2012. Upon the invitation of the organizer of the International Conference Geo SIN 2014 in Singapore he also delivered a Key Paper and organized one section of his own choice.

From the above it can be concluded that the energy and enthusiasm, unique only to creative individuals, persisted in academician Milovic, and despite his old age he still contributes remarkably to science and profession. This is confirmed once again by the paper written for this volume of the journal. It reflects rich experience and original ideas that have been translated into the proposed calculation model, which contributes to a more realistic assessment of pile capacity. All this fully justifies je kao autor ili koautor napisao, u našem časopisu, a i u ovom broju, za šta smo mu veoma zahvalni, jer je time sadržajno obogatio naš časopis. Uz radost što i u ovim godinama kreativno stvara i deluje, želimo mu da i ubuduće ostane u dobrom zdravlju i u mogućnosti da nastavi sa svojim radom.

Glavni i odgovorni urednik

Radomir Folić

the choice of academician D. Milovic as the first person to whom the entire volume of this journal is dedicated.

By his work, it always went beyond the narrow framework of the field of geotechnical engineering. This was facilitated by his broad culture that fits the giants, culture which, in addition to the excellent knowledge of the profession and science and world languages, accompanies other areas as well, especially achievements in civil engineering. He was also thinking of others and was happy for their success. He spoke and wrote about the Ilija Stojadinovic, BSc. designer of large span bridges, especially the Krk bridge, which has long held the world record for the achieved spans. He avoided interviews with journalists because he believed that they accepted many statements without argumentation, to which he opposed and sought a factual basis in everything and disagreed with the expression "Professional community, that's me."

The editor of this Journal worked with academician Dusan Milovic for several years in the same institution from which he went to a well deserved pension in 1992. Their contacts continued when prof. Milovic moved to Canada, and resulted in the publication of many papers, which he has written for this Journal either as the author or co-author, including this volume as well, enriching thereby our Journal, for which we are very grateful to him. Being happy for his ability to create and work in this age, we wish him good health in the future to be able to continue with his creative work.

Editor in chief
Radomir Folic

Biografija akademika prof. dr Dušana Milovića, dipl.inž.građ.

Biography of Academician Prof. Dr. Dusan Milovic, B.C.Eng.

Dušan Milović (1925) rođen je u Novoj Varoši, u Srbiji. Gimnaziju je završio u Beogradu 1943. godine. Diplomirao je 1954. godine na Građevinskom fakultetu u Beogradu - konstruktivni smer (oblast: armirano-betonski mostovi). Doktorsku disertaciju, pod naslovom "Inženjerske osobine lesa u Jugoslaviji", odbranio je 1959. godine na Rudarsko-geološkom fakultetu u Beogradu i prvi u Srbiji dobio je zvanje doktora tehničkih nauka iz oblasti mehanike tla i fundiranja. U periodu od 1954. do 1966. godine radio je kao naučni saradnik u Institutu za ispitivanje materijala Srbije. Od 1966. do 1971. godine radio je na Univerzitetu u Šerbruku (Kvebek, Kanada), isprva kao pozvani profesor, a kasnije kao vanredni, te ubrzo potom i kao redovni profesor i šef Katedre za mehaniku tla i fundiranje. Od 1972. do 1980. godine, bio je savetnik u Institutu za građevinarstvo Vojvodine u Subotici i redovni profesor na novoosnovanom građevinskom fakultetu, na kome je bio i prvi dekan. Od 1980. do 1992. godine, bio je redovni profesor u Institutu za industrijsku gradnju Fakulteta tehničkih nauka u Novom Sadu i šef Katedre za mehaniku tla i fundiranje, gde je i penzionisan. Za dopisnog člana Vojvođanske akademije nauka i umetnosti izabran je 1981. godine, a za njenog redovnog člana – 1987. godine. Srpska akademija nauka i umetnosti primila ga je 1991. godine kao redovnog člana. Bio je član Društva za mehaniku tla i fundiranje Srbije (predsednik), član Jugoslovenskog društva za mehaniku tla i fundiranje (član predsedništva), delegat Jugoslovenskog društva za mehaniku tla i fundiranje u Svetskom društvu za mehaniku tla i fundiranje, član Predsedništva SANU, savetnik u Komitetu za uzimanje uzoraka pri Svetskom društvu za mehaniku tla i fundiranje. Tokom dugogodišnjeg rada na Univerzitetu u Novom Sadu, držao je predavanja iz mehanike tla i fundiranja studentima na Građevinskom fakultetu u Novom Sadu i Subotici, kao i u Institutu za uređenje voda Poljoprivrednog fakulteta u Novom Sadu. Na Građevinskom fakultetu u Šerbruku držao je predavanja i na magistarskim i na doktorskim studijama. Bio je mentor prilikom izrade više magistarskih radova i doktorskih disertacija. Treba naglasiti i to da je sredinom XX veka mehanika tla bila najmlađa disciplina u građevinarstvu u Jugoslaviji i tek su tada prve posleratne generacije imale su taj predmet u programu studija.

Istraživački rad

U periodu od 1954. do 1995. godine, pored fokusiranja na nastavni i obrazovni rad, Dušan Milović usmerio je svoju aktivnost i na rešavanje teorijskih problema u oblasti mehanike tla, kao i na eksperimentalno proučavanje temeljnog tla i temeljnih konstrukcija pri dejstvu opterećenja od objekta. Taj rad ostvaren je u šesnaest naučnoistraživačkih projekata, čiji je bio nosilac i glavni istraživač. Pomenute projekte finansirali su Fond za naučni rad Srbije (3), Conseil National de Recherches Ottawa u Kanadi (3), Siz za naučni rad Vojvodine (7), Jugoslovensko-američki Joint Venture projekat (1) i Fond za naučni rad Srpske akademije nauka i umetnosti (2). Originalna teorijska rešenja i rezultate eksperimentalnih ispitivanja objavio je u 226 radova, od kojih je u 195 prvi autor (a u 138 – jedini autor). Do 2009. godine, njegovi radovi citirani su 205 puta (SCI). U oblasti direktnog fundiranja, proširio je primenu teorije elastičnosti i prikazao rešenja za

Dusan Milovic was born on 28 March 1925 in Nova Varoš, Serbia . He finished his primary school in Nis and grammar school in 1943, in Belgrade. He graduated from the Faculty of Civil Engineering, in 1954. on the subject of concrete bridges, at Belgrade University. In 1959 he defended his doctoral thesis entitled "Engineering properties of loess soils in Jugoslavia" and he was the first who received Ph. D. degree in the field of Soil mechanics and foundations in Serbia .

From 1959 he worked at the Serbian Institute for Testing materials, Department of Soil Mechanics and Foundations in Belgrade. He remained there until 1966 working as science associate and senior science adviser. From 1966 until 1971 he worked in Québec, Canada, where he occupied various functions at the University of Sherbrooke, as invited professor, associated professor and the Head of the Department of Soil Mechanics and Foundation Engineering. In 1969, he was elected full professor. After return from Canada, in the period from 1971 until 1980 he was a counsellor in the Institute for Civil Engineering in Vojvodina (Subotica) and full professor and the first Dean of the newly opened Faculty of Civil Engineering. From 1980 until 1992 he was full professor at the Institute for Industrial Building at the Faculty of Technical Sciences in Novi Sad, director of the Institute and Head of the Soil Mechanics Department. He retired in 1992.

He was elected corresponding member of the Vojvodina Academy of Sciences and Arts in 1981 and in 1987 he became its full member. In 1991 he was elected full member of the Serbian Academy of Sciences and Arts .

During the long period of active work he taught Soil Mechanics and Foundation at the Faculty of Technical Sciences in Novi Sad, Faculty of Civil Engineering in Subotica, Faculty of Agriculture in Novi Sad and Faculty of Civil Engineering in Sherbrooke, Canada, where he had held post graduate courses. He headed for several master's thesis and doctoral dissertations in Serbia and Canada.

He speaks English and French, and has a considerable knowledge of German.

In the field of deep foundations Dusan Milovic developed the procedure for determination of bearing capacity of piles, subjected to a vertical compression load, using the results of the cone penetration tests in the field. By means of the finite difference method, he solved theoretically the problem of calculation of horizontal displacements, bending moments, rotation and shear forces for any relative rigidity of free head or fixed head piles, produced by horizontal load and bending moment. The agreement between the theoretical and field test results was performed using field load tests in the scale 1:1. Experience gained in engineering practice confirms that his method provides more precise results than those obtained by static or dynamic methods and represents considerable improvement in prediction of pile behaviour subjected to vertical or horizontal load. During the long period of time it has been noticed that seismic forces can cause the liquefaction in sand layers with catastrophic consequences. Studying the behaviour of sand deposits under the influence of cyclic load he has found that severe damages and collapse of structure very often take place due to degradation of skin friction of piles.

One of his very significant activities was directed određivanje veličine sleganja i ugaonih distorzija za sve oblike i sve relativne krutosti temelja, za razne slučajeve opterećenja i kompleksne modele tla, uključujući anisotropna svojstva tla, ograničenu debljinu deformabilne sredine nedeformabilnim substratumom, kao i višeslojne sisteme. Rešenja dobijena metodom konačnih elemenata i dvostrukim Fourier-ovim redovima prikazana su na internacionalnim kongresima geomehanike i objavljena u časopisima svetskog renomea (osam radova u Londonu, Parizu, Berlinu, Moskvi i Tokiju, u periodu od 1970. do 1973. godine), te se smatraju pionirskim.

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U oblasti dubokog fundiranja, Dušan Milović rešio je probleme određivanja veličine graničnog i dozvoljenog opterećenja šipova pomoću podataka dobijenih iz terenskih opita statičke penetracije. Metodom konačnih razlika, prikazao je rešenje za određivanje horizontalnog pomeranja šipa, momenata savijanja, rotacije i poprečnih sila za šip bilo koje krutosti, sa slobodnom i uklještenom glavom, usled dejstva vertikalnog i horizontalnog opterećenja. Tačnost teorijskih rešenja proveravana je terenskim opitima, probnim opterećenjem u razmeri 1:1. Eksperimentalni radovi predstavljaju izvanrednu proveru teorijskih rešenja u mehanici tla.

Značajnih aktivnosti Dušana Milovića bila su usmerene na teorijske studije i terenska ispitivanja lesnog tla. Osim u našoj zemlji, ova vrsta tla je veoma rasprostranjena u Rusiji, Kini, Americi, kao i u mnogim drugim zemljama. Kako su u svim pomenutim zemljama registrovana veoma teška oštećenja, pa čak i rušenja objekata i pri relativno niskim vrednostima delujućeg opterećenja, vrlo opsežnim terenskim i laboratorijskim ispitivanjima, odredio je parametre koji su od presudnog značaja za ponašanje lesnog tla. Na osnovu dobijenih rezultata, modifikovao je teoriju proračuna ukupnih i diferencijalnih sleganja. Novim predloženim postupkom, dokazano je da se dato rešenje može uspešno primeniti na bilo koju lokaciju u svetu, gde se javlja lesno tlo. Milovićevi izvanredni radovi o lesu sadrže takve podatke kakvi još nigde u svetu nisu do sada objavljeni.

Pored aktivnog višedecenijskog rada na nastavnom i naučnom planu, aktivno je učestvovao u rešavanju najsloženijih problema fundiranja mnogobrojnih objekata visokogradnje u građevinarstvu. Za više od 220 objekata dao je rešenje za siguran i ekonomičan način fundiranja (npr. za stambene zgrade s trinaest spratova, za stambene zgrade do devetnaest spratova, za silose za žito, mostove, administrativne zgrade, čeličane, energane, šećerane, sportske centre, luke, brodogradilišta). Osim u zemlji, radio je studije fundiranja i za objekte u Iraku, Čehoslovačkoj, Poljskoj i Kanadi.

Rezultate istraživačkih radova prikazao je na mnogim svetskim kongresima za mehaniku tla i fundiranje (London 1957, Pariz 1961, Montreal 1965, Meksiko Siti 1969, Moskva 1973,Tokio 1977, Stokholm 1981, San Francisko 1985, Rio de Žaneiro 1989, Hamburg 1997, Osaka 2005. g. i Čikago 2013).

Njegovi radovi prikazani su na tri svetska kongresa za inženjersku geologiju - u Buenos Ajresu 1986, u Lisabonu 1994. i Vankuveru 1998. godine.

Učestvovao je s radovima na sledećim evropskim kongresima, internacionalnim regionalnim kongresima i dunavskim kongresima za mehaniku tla i fundiranje (Budimpešta 1963, Visbaden 1963, Čikago 1965, Haifa

toward theoretical, field and laboratory studies of loess soils .This kind of soil covers about 9% of continent surface, reaching the thickness greater than 100 m Beside of our country, loess is widely spread in Russia, China, America and in other countries It has been reported that loess exhibits unusual properties . In many countries a great number of damaged or collapsed structures has been noticed, despite the fact that the applied load was relatively low. On the basis of the extensive laboratory and field investigations he defined the parameters which have the greatest influence on the loess behaviour. He modified the method of settlement calculation, involving the additional component of differential settlement, caused by wetting or saturation of loess soil and including the effect of anisotropy. During the laboratory testing of loess samples he established that the mechanical disturbance can lead to the quite erroneous results and conclusions concerning its bearing capacity and expected settlements. By means of the obtained solution it is possible to solve successfully foundation problems on loess soils in every country with loess deposits. These results have been estimated as exceptional achievement in this field, not earlier published anywhere else.

In the capacity of designer, expert and consultant he has made a considerable contribution in the field of foundation engineering, providing a safe and economical solutions to the geotechnical problems for more than 220 structures. Some of the most important are apartment buildings with 13 to 19 stories, silo groups, bridges, steel work, rolling mill building, factory of chemical products, halls of fair, hotels, sport centers, shipbuilding yard, harbours and others important structures. In addition, solutions of the foundation problems have been provided for structures in Iraq, Poland, Czechoslovakia and Canada.

Papers have been published in Journals Glas (Serbian Academy of Sciences and Arts), Our Civil Engineering, Publications of the Institute for Testing Materials, Buildings, Road and Traffic, Materials and Structures. He has taken part with papers at 29 Yugoslav and Serbian congresses on Soil Mechanics and Foundation Engineering .

Some papers were published in foreign countries in the most recognized international geotechnical journals such as Géotechnique (London, England), Soils and Foundations (Tokyo, Japan), Journal of the American Society for Testing and Materials ASTM USA, Sol Soils (Paris, France), L'Ingénieur Constructer (Paris, France), Le Génie Civil (Paris, France), Bauingenieur (Berlin, Germany).

Papers have been presented at World Conferences on Soil Mechanics and Foundation Engineering, in London 1957, Paris 1961, Montreal 1965, Mexico City 1969, Moscow 1973, Tokyo 1977, Stockholm 1981, San Francisco, 1985, Rio de Janeiro 1989, Hamburg 1997, Osaka 2005 and Chicago 2013.

Papers have been presented at 3 World Conferences on Engineering Geology, in Buenos Aires 1986, Lisbon 1994 and Vancouver 1998 .

He has participated with papers at 27 European Congresses, International Regional Congresses and Danube Congresses on Soil Mechanics and Foundation Engineering, Budapest 1963, Wiesbaden 1963, Chicago 1965, Haifa 1967, Belgrade 1970, Bangkok 1971, 1967, Beograd 1970, Bangkok 1971, Budimpešta 1971, Pariz 1971, Stokholm 1974, Beč 1976, Bratislava 1977, Brno 1979, Pariz 1980, Cirih 1982, Amsterdam 1982, Budimpešta 1984, Pariz 1984, Nagoja 1985, Peking 1986. i 1988, London 1989, Budimpešta 1990, Firenca 1990, Vankuver 1991, Dalas 1992, Gent 1993. i Kopenhagen 1995).

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Pored aktivnih učestvovanja na internacionalnom planu, objavio je i sledeće knjige i monografije: "Geomehanika" 1976. godine (148 strana); "Mehanika tla" 1977 (243); „Mehanika tla" 1982 (323); „Mehanika tla" 1987 (475); "Analiza napona i deformacija u mehanici tla" (na srpskom i engleskom jeziku) 1974 (264 strane); "Problemi fundiranja na lesnom tlu" 1987 (255); "Greške u fundiranju" 2005 (438); "Problemi interakcije tlo-temelj - konstrukcija" 2009 (428); *Stresses and displacements for shallow foundations* 1992, Elsevier, 620 strana.

Tokom internacionalnih kongresa, bio je *Invited panel member* 1965. godine, potpredsednik sekcije za kolapsibilna tla 1969, *Invited lecturer* – 1969, *Invited lecturer* – 1979, *Invited speaker* – 1989, *Invited panel member* i *General reporter* – 1990, predsednik tehničke sekcije za kolapsibilna tla – 1992. i *Invited speaker* – 1995. godine. Nadalje, imao je poziv od Instituta za mehaniku tla Kineske akademije nauka da pripremi (*Key Paper*) za Internacionalni kongres za mehaniku tla i fundiranje u Vuhanu (Wuhan) 2012. godine i da organizuje rad jedne od sekcija, poziv od organizatora Internacionalne konferencije GEO SIN 2014. godine u Singapuru da bude njihov savetnik, da pripremi *Key Paper* i da organizuje jednu sekciju po sopstvenom izboru.

Priznanja i nagrade

Dušan Milović je dobitnik Oktobarske nagrade grada Beograda 1962. godine. Odlikovan je Medaljom zasluge za narod i Ordenom rada sa srebrnim vencem. Dušan Milović je počasni i zaslužni član Saveza građevinskih inženjera i tehničara Srbije.

Budapest 1971, Paris 1971, Stockholm 1974, Wien 1976, Bratislava 1977, Brno 1979, Paris 1980, Zurich 1982, Amsterdam 1982, Budapest 1984, Paris 1984, Nagoya 1985, Beijing 1986 and 1988, London 1989, Budapest 1990, Firenze 1990, Vancouver 1991, Dallas 1992, Ghent 1993 and Copenhagen 1995.

In addition, he published several books such as Soil Mechanics 1976, 148 pages, Soil Mechanics 1977, 243 pages, Soil Mechanics 1982, 323 pages, Soil Mechanics 1987, 475 pages, Analyses of Stresses and Deformations in Soil Mechanics, (Serbian and English) 1974, 264 pages, Foundation problems on loess soil, 1987, 255 pages, Mistakes in Foundations, 2005, 438 pages, Interaction problems soil - foundation - construction, 2009, pages 428 pages.

Stresses and displacements for shallow foundations, 1992, ELSEVIER, 620 pages.

He was invited for panel member in Chicago, 1965, vice president of the section for collapsible soils on the World Conference in Mexico 1969, lecturer at Ecole Polytechnique in Montreal 1969, panel member and lecturer at the Conference in Brno 1979, invited speaker in London 1989, panel member and General reporter in Budapest 1990, president of Technical section for collapsible soils at the International Conference held in Dallas 1992, invited speaker to deliver a lecture at the European Conference on Soil Mechanics and Foundation Engineering held in Copenhagen 1995; Invited speaker to deliver a Key Lecture at the International Conference on Problematic Soils - CHINESE

ACADEMY OF SIENCES Institute for Soil Mechanics, Wahan, 2012, and to be president of one Technical section; Invited speaker to deliver a Key Paper at the International Conference GEO SIN 2014 in Singapore.

Dusan Milovic was President of Serbian Society of Soil mechanics and Foundation Engineering, member of the Presidency of Yugoslav Society of Soil Mechanics, Representative of the Yugoslav Society at the World Society of Soil Mechanics and Foundation Engineering, member of the European Society of Numerical methods, member of the European Committee of Penetration Testing and advisor in the Committee of World Society for soil sampling.

NOSIVOST ŠIPOVA - TEORIJSKE I TERENSKE METODE

BEARING CAPACITY OF PILES - THEORY AND FIELD TESTS

Dušan MILOVIĆ

ORIGINALNI NAUČNI RAD
ORIGINAL SCIENTIFIC PAPER
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Karl Terzaghi, 1948 god.
"Temelji građevina uvek su bili pastorčad, zato što nema slave u temeljenju. Ali dela osvete zbog nedovoljne pažnje oko njih mogu biti katastrofalna". '

1 UVOD

Za pravilno dimenzioniranje temelja na šipovima, potrebno je zadovoljiti više kriterijuma, među kojima su najvažniji oni u vezi sa slomom tla i pojavom nedozvoljeno velikih sleganja. Pri svemu tome, potrebno je primeniti najekonomičnije rešenje – koje podrazumeva optimalan broj šipova odgovarajućeg poprečnog preseka i dužine.

Zbog značaja što tačnijeg određivanja veličine graničnog opterećenja šipova, razvijene su brojne metode – kako teorijske, tako i eksperimentalne – koje se koriste u inženjerskoj praksi. Međutim, pokazalo se da postoje znatne razlike u veličinama dobijenih rezultata. Stoga, za 48 izvedenih šipova izvršeno je i terensko ispitivanje probnim opterećenjem do sloma tla, kako bi se teorijski određene veličine graničnog opterećenja uporedile s realnom veličinom i kako bi se utvrdio stepen tačnosti najčešće korišćenih teorijskih metoda.

Potrebno je pomenuti i to da je na jednom nedavno održanom svetskom kongresu za mehaniku tla i fundiranje generalni izvestilac obavestio skup svetskih stručnjaka da još nema rešenja kojim bi se mogla odrediti veličina graničnog opterećenja, a da pritom greška bude manja ili veća za 30% od veličine dobijene probnim opterećenjem. Veća vrednost od realne vrednosti ima za posledicu da umanji stepen sigurnosti objekta, ili – u drugom slučaju – da poveća troškove gradnje.

Akademik prof. dr Dušan Milović,dipl.ing.građ. SANU

Karl Terzaghi, 1948 th.
"Foundations of structures always were orphans because there is no glory in foundation. But the works of revenge because of this neglect can be catastrophic”.

1 INTRODUCTION

For successful design of the foundations on piles it is necessary to satisfy some criterion. Amongst the most important are soil rupture and unacceptable great settlement. Also it is very important to apply the most economical solution, which consist of the optimal number of piles with the corresponding cross section and length.

In order to determine the values of the bearing capacity of piles numerous theoretical and experimental methods were developed, which are used in the engineering practice. However, it was observed that the obtained results were very different. For that reason 48 concrete piles were in situ tested in order to determine the real values of the ultimate load and to compare it with the theoretical results. In this way it was possible to evaluate the level of precision of the used theoretical solutions.

It is necessary to mention that in the recent World Conference on Soil Mechanics and Foundation Engineering the General Reporter informed that the Society does not have a solution to determine the ultimate bearing capacity of pile without making the error ± 30 % from the real value obtained by field load test of a pile.

Academician Professor Dr. Dusan Milovic, SASA

2 METODE ZA ODREĐIVANJE GRANIČNOG I DOZVOLJENOG OPTEREĆENJA ŠIPOVA

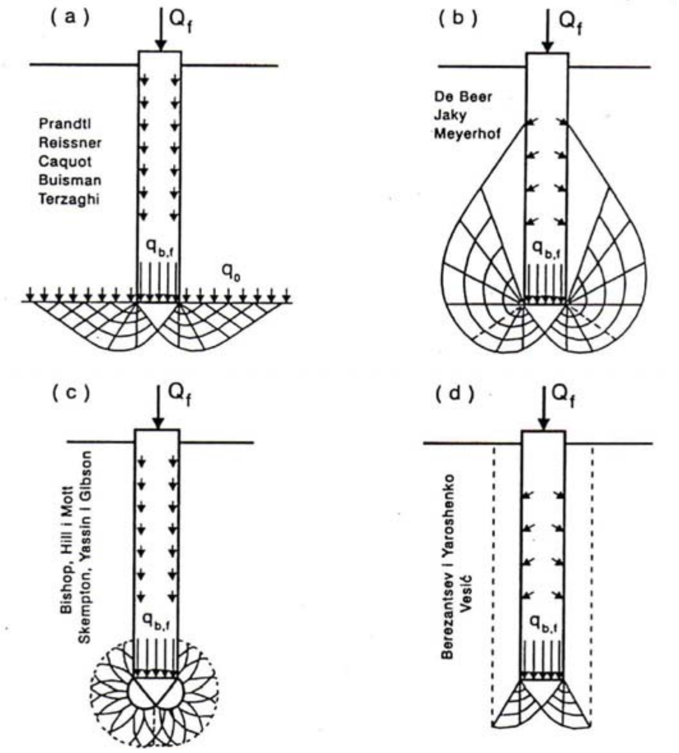
2.1 Statičke metode

Pri određivanju graničnog i dozvoljenog opterećenja šipova koriste se parametri koji se određuju laboratorijski sa raznih dubina. U teorijskom proučavanju problema autori pretpostavljaju razne oblike kliznih površina u zoni baze šipa, sto je prikazano na sl. 1.

2 METHODS FOR DETERMINATION OF THE ULTIMATE AND ADMISSIBLE LOADING OF PILES

2.1 Static methods

For determination of the ultimate and admissible loading of piles several parameters are used, which are decided by laboratory tests of the mechanically undisturbed samples taken from various depths. In the theoretical study of problems the authors assumed various shapes of sliding surfaces in the zone of pile base, as shown in figure 1.



Slika 1. Pretpostavljeni mehanizmi sloma u zoni baze šipova
Figure 1. Assumed failure mechanisms in zone of the piles base

Granično opterećenje šipa prikazano je kao zbir komponente nosivosti bazom šipa i komponente nosivosti trenjem po omotaču šipa i može se napisati u sledećem obliku:

The ultimate load of a pile is shown as the sum of the component bearing by base of pile and by component bearing by skin friction of a pile, and can be written in the following form:

$$P_f = pA_p + f_{sk} A_{sk} \tag{1}$$

Gde je:

- P_f = granično opterećenje šipa;
- p = granični pritisak u nivou baze šipa;
- A_p = površina baze šipa;
- f_{sk} = specifično trenje po omotaču šipa;
- A_{sk} =površina plašta šipa.

Pri tome, treba imati na umu da je vrlo teško doći do neporemećenih uzoraka iz nekoherentnih slojeva tla radi određivanja njihovog ugla unutrašnjeg trenja.

Dobijene veličine graničnog opterećenja šipa, određene statičkim metodama, znatno odstupaju od rezultata terenskih opita probnog opterećenja.

where is:

- P_f = ultimate load of a pile;
- p = ultimate pressure/ load of a pile base;
- A_p = surface of a pile base;
- f_{sk} = specific skin friction of a pile;
- A_{sk} = skin surface of a pile.

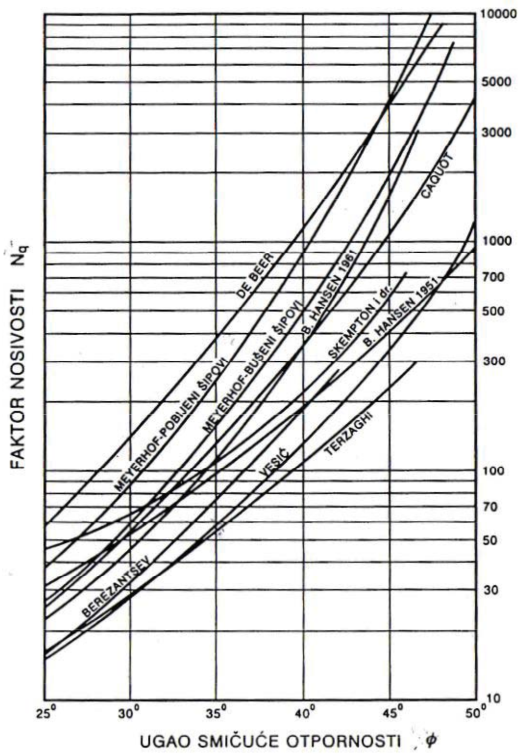
It is worth mentioning that it is very difficult to get the mechanically undisturbed samples from non cohesive soils and to determine consequently the real values of the angle of their internal friction of soil.

The obtained values of the ultimate load by static

Primera radi, na sl. 2 pokazana je zavisnost veličine koeficijenta N_q od ugla φ.

methods considerably differ from the results got by the in situ tests.

Therefore, in Fig 2 is shown the dependence of coefficient N_q on the angle of internal friction φ.



Slika 2. Zavisnost koeficijenta N_q od ugla unutrasnjeg trenja tla
Figure 2. Variation of coeficient N_q with soil friction angle

Ove razlike jednim delom potiču i od primene različitog koeficijenta sigurnosti za mobilisan ugao unutrašnjeg trenja. Radi ilustracije, može se zapaziti da se za ugao trenja od 30 stepeni faktor *Nq* kreće u granicama od 30 do 140 i za ugao od 35 stepeni u granicama od 55 do 400.

These differences are caused by using the various values for the coefficient of safety for the mobilized angle of internal friction. For illustration, if angle of friction is 30 degrees, the coefficient Nq varies between the limits 30 - 140 and if the angle is 35 degrees this coefficient varies from 55 to 400.

2.2 Dinamičke metode

Radi povećanja tačnosti teorijskih metoda za proračun nosivosti šipova, istovremeno su razvijane i dinamičke metode u kojima opšti izraz ima sledeći oblik:

2.2 Dynamic methods

In order to increase the precision of the theoretical methods, the dynamic methods were developed, using the following general expression:

$$WH = P_d s + E_1 \tag{2}$$

Gde je:

W = težina malja;

H = visina pada malja;

P_d = dinamička otpornost šipa;

s = utiskivanje šipa usled pada malja;

E_1 = gubitak uložene energije, određen teorijom udara prema Newton-u.

Na osnovu sprovedenih analiza dobijenih rezultata, dinamičkim metodama, te rezultata terenskih opita probnog opterećenja, zaključilo se da je disperzija rezultata izrazito velika, što je uzrokovalo vrlo retku upotrebu ove metode.

where is:

W = the weight of the hammer;

H = height of the hammer drop

P_d = dynamic resistance of pile

s = penetration of pile due to hammer drop

E_1 = loos of the applied energy, determined by Newton's shock theory.

On the bases of the analysis the results obtained by dynamic methods and the results of the in situ tests of a pile, it is concluded that the dispersion of the results is very significant, which resulted in a very sporadic usage of this method.

2.3 Određivanje graničnog opterećenja šipa terenskim opitom probnog opterećenja

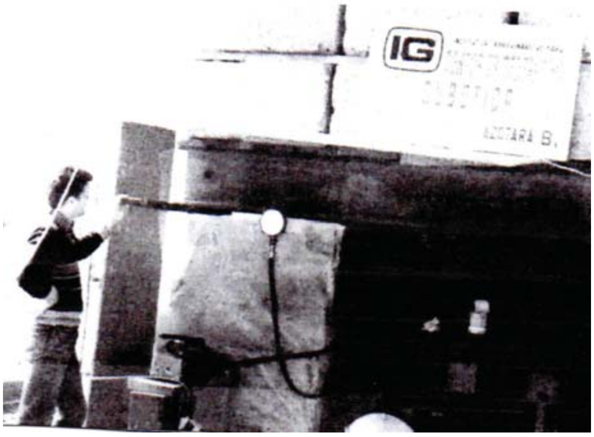
Terenski opit probnog opterećenja šipa – u razmeri 1:1 – smatra se najpouzdanijim načinom za određivanje graničnog opterećenja šipa. Na glavu šipa nanesu se najčešće betonske kocke do opterećenja koje odgovara projektovanoj sili. Ona se nanosi na šip u etapama i povećava tek kada se nanetom silom postigne konsolidacija tla.

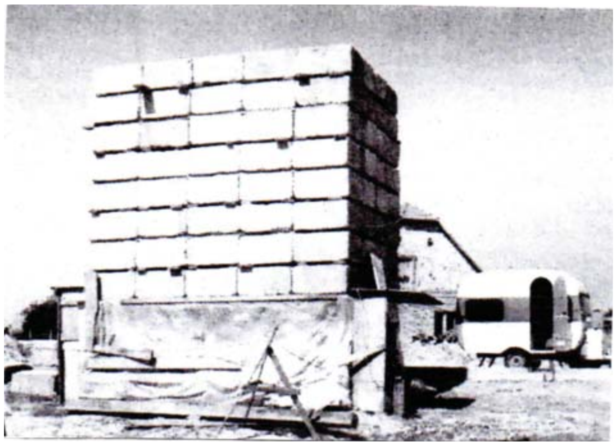
Na slici 3 prikazan je kontrateret za probno opterećenje.

2.3 Determination of the ultimate loading of pile by in situ load tests

In situ test loads of piles in a 1:1 correlation is considered the best way to determine ultimate loading of pile. The head of a pile is most often loaded by concrete blocks in order to reach the designed force. It is loaded on the pile gradually and is being increased only when soil consolidation has been achieved by the applied force.

Figure 3 shows the loaded pile by concrete blocks.





Slika 3. Kontra teret postavljen na glavu šipa
Figure 3. Field load test of the pile

2.4 Terenske metode statičkom penetracijom

Da bi se izbegao nepovoljan uticaj mehaničke i naponske poremećenosti uzoraka tla pri laboratorijskom određivanju ugla unutrašnjeg trenja, kao i pri pretpostavljanju oblika kliznih ravni ispod i oko baze šipa u raznim metodama, u novije vreme se sve češće s podacima iz statičke penetracije određuju veličine graničnog i dozvoljenog opterećenja šipova.

Analiza rezultata statičke penetracije sprovedena je za 48 betonskih šipova. Isto tako, na svim šipovima izveden je terenski opit probnog opterećenja, što omogućava da se veličine graničnih opterećenja uporede s veličinama određenim drugim metodama.

2.4.1 Metoda G. Meyerhof-a

Meyerhof (1956), na osnovu modelskih ispitivanja malih dimenzija, koristio je opšti izraz za proračun graničnog opterećenja šipa, pri čemu je za specifično trenje po omotaču šipa uveo različite koeficijente za koherentne i nekoherentne materijale.

Tako, za koherentne i nekoherentne materijale koriste se izrazi:

2.4 Field methods by static penetrations

In order to avoid the problems like mechanical disturbance of soil samples taken for the laboratory determination of the angle of internal friction, as well as the assumed shape of the slip surfaces under and around the base of pile, in recent years the static penetration tests are used to determine the ultimate load for pile.

The analysis of the results of the static penetration is made for 48 concrete piles. All piles with ratio 1: 1 were loaded until failure in soil was reached. Such procedure made it possible to compare the theoretical values of the ultimate load with the real values, registered by in situ tests.

2.4.1 Method G. Meyerhof

Meyerhof (1956) on the basis of investigation on models with small dimensions used the general expression for determining the ultimate load of pile, and introduced different coefficients for coherent and non coherent soils for specific friction of the lateral pile surface.

The following expressions were used:

$$P_f = R_p A_p + \frac{R_{pav}}{100} A_{sk} \tag{3}$$

$$P_f = R_p A_p + \frac{R_{pav}}{200} A_{sk} \tag{4}$$

Gde je:

R_p = otpornost na prodor konusa ispod baze šipa;

A_p = površina baze šipa;

R_{pav} = prosečna otpornost na prodor konusa duž omotača šipa;

A_{sk} = površina omotača šipa.

2.4.2 Metoda Mohan - a i Kumar - a

Mohan i Kumar (1963) su na osnovu podataka iz 8 probnih opterećenja instrumentalnih šipova i podataka iz literature predložili sledeći izraz za proračun graničnog i dozvoljenog opterećenja šipa:

were is:

R_p =penetration resistance under the base of a pile;

A_p = surface of a pile base;

R_{pav} = average penetration resistance of lateral surface of pile;

A_{sk} = lateral surface of pile.

2.4.2 Method Mohan and Kumar

Mohan and Kumar (1963) on the basis of the results for 8 in situ tests and data from literature used the following expression for evaluation of the ultimate and admissible loading of pile:

$$P_f = P_p + P_{sk} = P_{sk} + \frac{R_{pav}}{50} A_{sk} \tag{5}$$

gde je:

R_p = otpornost na prodor konusa ispod baze šipa;

A_p = površina poprečnog preseka baze šipa;

R_{pav} = prosečna otpornost na prodor konusa oko stabla šipa;

A_{sk} = površina omotača šipa.

Pri tome, za proračun dozvoljenog opterećenja šipa koristi se parcijalni faktor sigurnosti F_p = 2,5 za nosivost bazom i F_{sk} = 2,0 za nosivost trenjem po omotaču šipa.

where is:

R_p = penetration resistance under the base of a pile;

A_p = surface of a pile base;

R_{pav} = average penetration resistance of lateral surface of pile;

A_{sk} = lateral surface of pile;

In this case the partial factor of security for bearing of base F_p = 2.5 was used and for the bearing of the lateral surface of pile F_{sk} = 2. 0.

2.4.3 Metoda Bustamante-a i Gianeselli-a

Bustamante i Gianeselli (1982) uveli su redukcioni faktor K_p za nosivost šipa bazom i faktor Ksk za nosivost trenjem po omotaču u koherentnom tlu, pa se granično opterećenje može odrediti pomoću izraza:

2.4.3 Method Bustamante and Gianeselli

Bustamante and Gianeselli (1982) are introduced a factor K_p for the bearing of pile base and factor K sk for the bearing of lateral surface of pile in cohesive soils. The ultimate load now can be written in the following form:

$$P_f = R_p A_p K_p + \sum_i \frac{R_{pi}}{K_{ski}} D\pi h_i \tag{6}$$

Gde je:

K_p = bezdimenzioni koeficijent za slojeve tla ispod baze šipa;

R_{Ph} =prosečna penetraciona otpornost na prodor konusa u sloju debljine *h;*

K_{sk} = bezdimenzioni koeficijent za slojeve iznad baze šipa;

D = prečnik šipa;

h = debljina sloja i;

Mada se metode statičke penetracije zasnivaju na istoj vrsti terenskog ispitivanja, odnosno na merenju veličine otpornosti na prodor konusa duž stabla i ispod baze šipa, primenom pomenutih metoda dobijaju se znatne razlike u veličinama graničnog i dozvoljenog opterećenja.

where is:

K_p = dimensionless coefficient for soil layers under the pile base;

R_{Ph} = average penetration resistance in the layer of thickness h.

K_{sk} = dimensionless coefficient for soil layers above the pile base;

D = diameter of pile;

h = thickness of the layer i.

Despite the fact that all methods are based on the same kind of in situ investigation, by using the mentioned methods one obtains considerable differences in values of the ultimate and admissible loading of piles.

2.4.4 Metoda autora i upoređivanje rezultata s prikazanim metodama

U daljem tekstu prikazaće se rezultati pojedinih autora, koji se odnose na određivanje graničnog i

2.4.4 New method of Milovic and comparison of the results with the presented methods

Further are shown the results of all mentioned authors concerning the determination of the ultimate dozvoljenog opterećenja šipova, a koji su određeni s podacima iz terenskih opita statičke penetracije i opita probnog opterećenja u razmeri 1:1.

U ovom radu prikazani su rezultati analize 48 šipova za koje su određene veličine graničnog i dozvoljenog opterećenja i za koje su bili izvedeni terenski opiti probnog opterećenja.

U novoj metodi prikazan je izraz prema kome su vršeni proračuni veličine komponente sile koju prima baza šipa i komponente koju prima omotač šipa i koji je dat u sledećem obliku:

loading of piles, which are obtained by using the results of static penetration tests and the results obtained by site loading tests on the pile in the scale 1: 1.

In this paper are shown the results for 48 piles. The values of the ultimate load were obtained using the theoretical solutions and also the results of field load tests.

The expression used in the new method for the determination the values of base pile component and lateral surface component is given by:

$$P_f = P_p + P_{sk} = P_p A_p \alpha_p + \sum \frac{R_{ph}}{\alpha_{sk}} D\pi h_i \qquad (7)$$

Gde je:

R_p = otpornost na prodor konusa u zoni sloma oko baze;

R_{ph} = prosečna otpornost na prodor konusa u sloju debljine h;

A = površina poprečnog preseka baze šipa;

D = prečnik šipa;

h = debljina posmatranog sloja i;

α_P i α_{sk} = koeficijenti nosivosti bazom i trenjem po omotaču šipa.

Analizom je obuhvaćeno 48 betonskih šipova, ali će dva šipa biti detaljno obrađeni.

BETONSKI ŠIPOVI

Radi ilustracije, prikazaće se postupak analize za dva betonska šipa.

Na slici 4 prikazana je zavisnost uvedenih koeficijenata α_p od R_p.

where is:

R_p = penetration resistance under the base of a pile:

R_{ph} = average penetration resistance in the layer of thickness h;

A = surface of a pile base;

D = diameter of a pile;

h = thickness of the layer i;

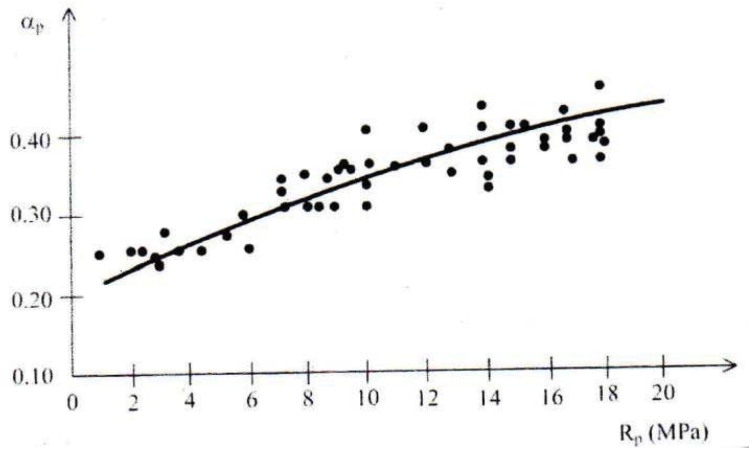
α_P and α_{sk} = dimensionless coefficients for bearing capacity.

In the analysis of 48 concrete piles are included, and 2 piles are considered in detail.

CONCRETE PILES

For illustration the procedure of the analyses of two concrete piles is shown.

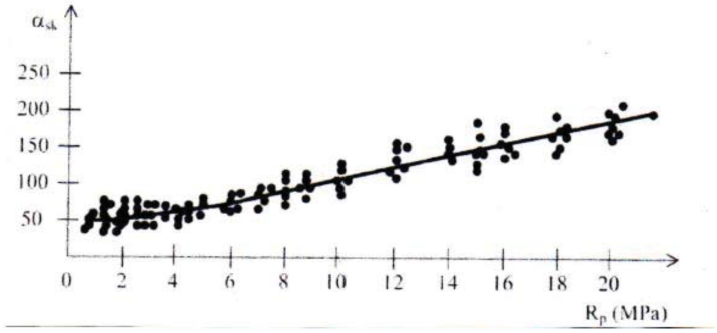
In the figure 4 the dependence of the coefficient α_p od R_p is shown.



Slika 4. Zavisnost koeficijenta α_p od R_p
Figure 4. Variation of the coefficient α_p with R_p

Na slici 5 prikazana je zavisnost uvedenih koeficijenata α_{sk} od R_{pu}.

In the figure 5 the dependence of the coefficient α_{sk} on R_{pu} is shown.



Slika 5. Zavisnost koeficijenta α_{sk} od R_{pu}

Figure 5. Variation of the coefficient α_{sk} on R_{ph}

ŠIP BR 30

Zgrada CK u Bloku 20, Novi Beograd
Dužina i prečnik šipa L = 11,6 m; D = 0,60 m;
Kota glave i baze šipa; 70,6 i 5 9,0;
Površina poprečnog preseka šipa A = 0, 352 m^2;
Površina omotača šipa A_{sk} = 21,85 m^2;
Prosečna otpornost na prodor konusa R skav - = 4,6 MPa;
Odnos modula elastičnosti E_b/E_{sk} = 10
U tabeli 1 prikazani su sastav tla i njegove penetracione otpornosti.

PILE No 30

Building CK, Block 20, New Belgrade
Length and diameter of pile L = 11. 6 m; D = 0. 60 m
Level of head and base of pile 70. 6; 59. 0
Surface of the cross section of pile A = 0. 352 m^2
Lateral surface of pile A_{sk} = 21. 85 m^2
Average resistance of cone penetration R skav = 4.6 MPa
Ratio of modules elasticity E_b/E_{sk} = 10
In Table 1 the soils profile and the penetration resistances are shown.

Tabela 1. Sastav tla i penetracione otpornosti slojeva
Table 1. Soil profile and the penetration resistance of each layer

Dubina / *Depth* z, m	Debljina / *Thickness* h, m	Vrsta tla / *Soil profile*	Otpornost *Cone resistance* R_p (MPa)
0.0 - 1.6	1.6	prašina glinovita , muljevita *muddy clay with silt*	1.5
1.6 - 6.6	5.0	prašina sa prašinastim peskom *silt with sand*	4.0
6.6 - 8.6	2.0	prašina muljevita *silt with muddy*	2.0
8.6 - 11.6	3.0	pesak sa malo šljunka *sand with gravel*	9.0
11.6 -15.0	3.4	šljunak sa sitnim peskom *gravel with silt and sand*	12.0

Pomoću svake prikazane metode, određene su veličine graničnog opterećenja, korišćenjem rezultata statičke penetracije. U datom slučaju, dobijene su sledeće vrednosti:

Mohan i dr. P_f = 4,22 + 2,01 = 6,23 MN
Meyerhof P_f = 4,22 + 0,50 = 4,72 MN
Bustamante i Gianeselli P_f = 1,48 + 1,17 = 2,65 MN
Milović P_f = 1,69 + 1,73 = 3,42 MN
Probno opterećenje P_f = 3,50 MN.

Na osnovu prikazanih rezultata, može se zaključiti da je disperzija znatna i da je veličina graničnog opterećenja po Milovićevoj metodi vrlo bliska veličini dobijenoj probnim opterećenjem.

When using all mentions methods the values of the ultimate load and the results of the penetration tests, the following results are obtained:

Mohan i dr P_f = 4.22 + 2.01 = 6.23MN
Meyerhof. P_f = 4.22 + 0.50 = 4.72 MN
Bustamante&Gianeselli P_f =1.48+1.17=2.65 MN
Milovic P_f = 1.69 + 1.73 = 3.42 MN
In situ load test P_f = 3.50MN.

On the bases of these results one may conclude that the dispersion is very high, but that the ultimate load according to Milovic method is very near to the value registered by in situ load test.

ŠIP BR 41
Betonski most u Jasenovcu

Dužina i prečnik šipa L = 16, 0 m; D = 0, 90 m;
Površina poprečnog preseka šipa A_p = 0,636 m^2;
Površina omotača šipa A_{sk} = 45 m^2;
Prosečna otpornost na prodor konusa R_{skav}= 3,2 MPa;
Odnos modula elastičnosti E_b/E_{sk} = 2.

U tabeli 2 prikazani su sastav tla i penetracione otpornosti svakog sloja.

PILE No 41
Concrete Bridge in Jasenovac

Length and diameter of pile L=16. 0 m; D= 0. 90 m
Surface of the cross section of pile A_p = 0. 636 m^2
Lateral surface of pile A_{sk} = 45 m^2
Average resistance of cone resistance R_{skav} = 3.2 MPa
Ratio of modulus elasticity E_b/E_{sk} = 2

In Table 2 is shown the soil profile and the penetration resistance of each layer.

Tabela 2. Sastav tla i penetracione otpornosti slojeva
Table 2. Soil profile and the penetration resistance of each layer

Dubina / *Depth* z, m	Debljina *Thickness* h, m	Vrsta tla / *Soil profile*	Otpornost *Cone resistance* R_p (MPa)
0.0 - 10.0	10.0	glina prašinovita, malo muljevita *clay with silt and muddy*	2.0
10.0 - 16.0	6.0	pesak sa prašinom *sand with silt*	6.0
16.0 - 22.0	6.0	Šljunak sitan sa sitnim peskom *Gravel with fine sand*	9.0

Pomoću svake prikazane metode, određene su veličine graničnog opterećenja korišćenjem rezultata statičke penetracije.

Mohan i dr. P_f = 5,72 + 2,88 = 8,60 MN
Meyerhof P_f = 5,72 + 1,44 = 7,16 MN
Bustamante i Gianeselli P_f = 2,58 + 1,22 = 3,80 MN
Milović P_f = 2,00 + 2,59 = 4,59 MN
Probno opterećenje P_f = 4,70 MN.

I u ovom slučaju zapaženo je da veličine graničnog opterećenja pokazuju neprihvatljivu razliku, dok je Milovićevom metodom postignuto smanjenje razlike s probnim opterećenjem.

U tablici 3 prikazan je za sve šipove odnos veličine graničnog opterećenja određene terenskim opitima probnog opterećenja i veličine sila koje su dobijene primenom nove Milovićeve metode. Ovaj odnos je vrlo blizak jedinici, što znači da se novom metodom može vrlo pouzdano odrediti granična nosivost šipova .

Napominje se i to da su analizirani šipovi bili izvedeni u Novom Beogradu, Novom Sadu, Zrenjaninu, Subotici, Crnji, Vrbasu, Beočinu, Jasenovcu, Belgiji, Grčkoj, Iraku, Americi i Kanadi. To znači da je tlo u kome su vršena ispitivanja bilo raznovrsno u pogledu geološkog sastava..

When using the mentioned methods for the ultimate load and the results of the penetration tests, the following values are obtained:

Mohan P_f = 5.72 + 2.88 = 8.60 MN
Meyerhof P_f = 5.72 + 1.44 = 7.16 MN
Bustamante&GIasenelli P_f =2.58+1.22=3.80 MN
Milovic P_f = 2.00 + 2.59 = 4.59 MN
In situ load test. P_f = 4.70 MN

In this case also the valu es of the ultimate load are very different and can not be accepted. However, the Milovic ' s results show very good concordance with the results from in situ load tests.

In Table 3 the ratio between the ultimate load for all piles registered by new method Milovic and by the results obtained in situ load tests are very closed tol uniti and allow to concllude thatt new method can be used with confidence to determine the ultimate load of a pile.

It is important to note that the analysed piles are carried out on several locations in New Belgrade, Novi Sad, Zrenjanin, Subotica, Crnja, Vrbas, Beocin, Jasenovac, Belgija, Greece, Iraq, USA and Canada. Thus, the various locations with various geological profile were examined.

Tabela 3. Probni opit , nova metoda
Table 3. In situ test, new method

Broj šipa *Number of Piles*	Odnos *Ratio*	Probni opit *In situ test* [MN]	Nova metoda New method [MN]
1	1.02	2.50	2.44
2	1.02	1.60	1.57
3	0.96	1.30	1.35
4	0.86	1.90	2.22
5	0.90	0.45	0.50
6	1.02	1.80	1.76

Broj šipa *Number of Piles*	Odnos *Ratio*	Probni opit *In situ test* [MN]	Nova metoda *New method [MN]*
7	1.00	0.35	0.35
8	0.91	2.50	2.75
9	1.09	3.00	2.76
10	0.91	0.30	3.3
11	1.08	2.50	2.32
12	0.88	2.00	2.26

Broj šipa *Number of Piles*	Odnos *Ratio*	Probni opit *In situ test* [MN]	Nova metoda New method [MN]
13	1.07	3.00	2.80
14	0.79	1.85	2.33
15	1.06	2.00	1.89
16	0.83	2.60	3.12
17	1.14	2.20	1.92
18	0.90	0.60	0.67
19	1.02	2.50	2.44
20	0.96	2.00	2.08
21	1.04	3.00	2.89
22	0.95	0.80	0.84
23	1.00	1.40	1.40
24	1.11	1.50	1.35
25	1.03	1.00	0.97
26	0.87	3.30	3.78
27	1.03	4.00	3.88
28	1.09	4.00	3.67
29	0.89	3.20	3.58
30	1.02	3.50	3.42

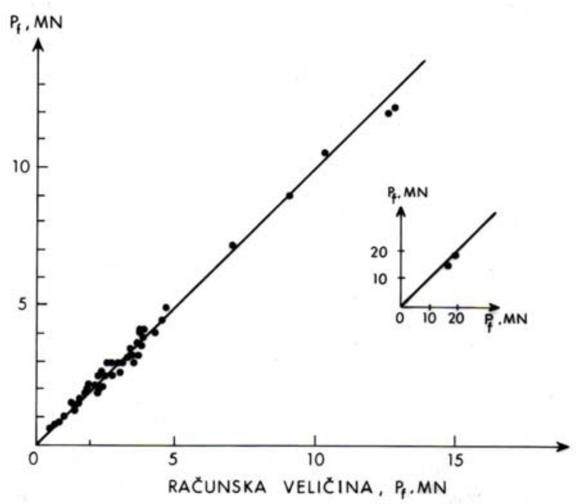
Broj šipa *Number of Piles*	Odnos *Ratio*	Probni opit *In situ test* [MN]	Nova metoda *New method [MN]*
31	0.87	2.10	2.40
32	0.94	3.50	3.73
33	1.08	4.00	3.70
34	0.93	3.20	3.44
35	0.94	4.00	4.27
36	1.04	3.80	3.66
37	1.00	4.50	4.52
38	1.05	4.00	3.81
39	0.97	3.20	3.29
40	1.01	3.00	2.97
41	1.02	4.70	4.59
42	0.93	12.00	12.88
43	0.94	12.00	12.75
44	1.02	7.20	7.05
45	1.00	9.00	8.99
46	0.91	15.00	16.54
47	0.97	10.00	10.28
48	0.95	17.50	18.44

Odnos računskih veličina graničnog opterećenja šipova po novoj metodi prema veličinama određenim terenskim opitima probnog opterećenja prikazan je i na sl. 6, iz koje se vidi da je razlika svedena na potpuno prihvatljiv nivo.

Na osnovu rezultata za 48 šipova odnos veličina graničnog opterećenja iz terenskih opita probnog opterećenja i teorijskih rezultata po novoj metodi Milovića sa odnosom 0,88 – 1,08, može se smtrati da je nova metoda znatno smanjila razliku između teorijskih veličina graničnih opterećenja i veličina određenih probnim opterećenjem.

Comparison the values between the ultimate load of pile determined by new method with the values obtained by field loading tests is shown in Fig. 6. where is clearly shown that the difference is quite acceptable .

On the basis of the results for 48 piles one may conclude that the new method Milovic with relation 0.88 - 1.08 considerably decreases the difference between the theoretical values of the ultimate load and the value obtained by in situ tests.

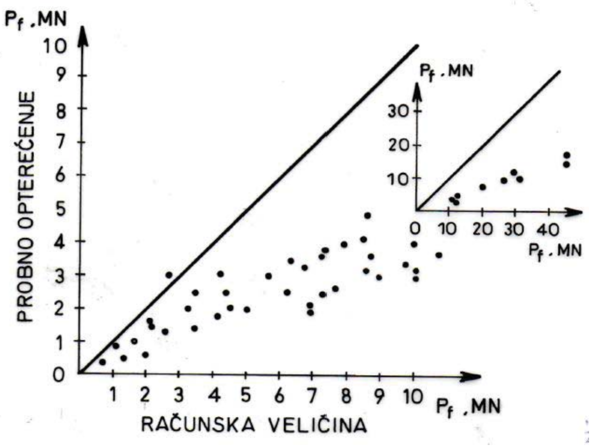


Slika 6. Upoređenje veličina graničnog opterećenja određenih novom metodom (Milović) sa veličinama određenim probnim opterećenjem

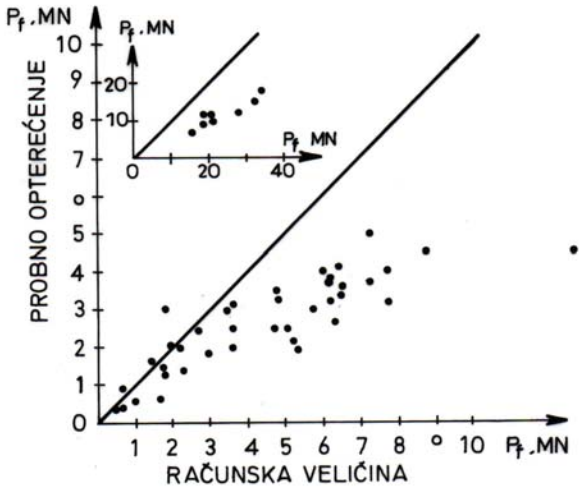
Figure 6. Comparison of the ultimate load determined by the new Milovic method with the ones obtained by field load tests

Na slikama 7, 8 i 9, prema metodama Mohan-a i dr Meyerhof-a, te Bustamante-a i Gianeselli-a, prikazane su veličine graničnog opterećenja i upoređene su s rezultatima probnog opterećenja.

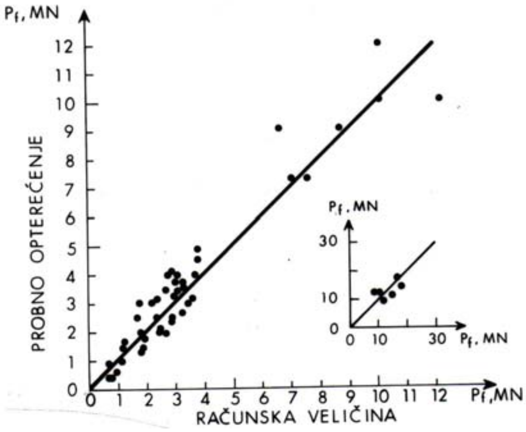
In figures 7, 8 and 9 are shown the results of the ultimate load obtained by the methods Mohan and Dr, Meyerhof, Bustamante and Gianeselli and compared with the results of in situ tests.



Slika 7. Upoređenje veličina graničnog opterećenja određenih metodom Mohan-a sa veličinama određenim probnim opterećenjem
Figure 7. Comparison of the ultimate load determined by the Mohan method with the ones obtained by field load tests



Slika 8. Upoređenje veličina graničnog opterećenja određenih metodom Meyerhof-a sa veličinama određenim probnim opterećenjem
Figure 8. Comparison of the ultimate load determined by the Meyerhof method with the ones obtained by field load tests



Slika 9. Upoređenje veličina graničnog opterećenja određenih metodom Bustamante-a sa veličinama određenim probnim opterećenjem
Figure 9. Comparison of the ultimate load determined by the Bustamante method with the ones obtained by field load tests

Rezultati Mohan-a ukazuju na to da se primenom njihove metode dobijaju veličine graničnog opterećenja, koje su znatno veće od realnih veličina određenih probnim opterećenjem. Oni su pretežno u granicama 1,12 – 4,55.

Rezultati Meyerhof-a kreću se u širokim granicama i znatno odstupaju od realnih veličina dobijenih probnim opterećenjem s granicama 0,62 – 3,22.

Rezultati Bustamante-a i Gianeselli-a pokazuju nešto uže granice, ali još uvek su veće od realnih veličina graničnog opterećenja, s granicama 0,58 – 2,43.

Rezultati dobijeni novom Milovićevom metodom kreću se u vrlo uskim granicama 0,88 – 1,08.

3 ZAKLJUČCI

Na osnovu analize rezultata dobijenih novom metodom za određivanje graničnog opterećenja šipa mogu se doneti sledeći zaključci:

Proračun veličine graničnog opterećenja šipova pomoću metoda koje se zasnivaju na korišćenju podataka iz statičkee penetracije daje veoma različite rezultate. Veličine graničnog opterećenja betonskih šipova u nekim slučajevima dostižu i četvorostruke veličine, određene terenskim opitom probnog opterećenja.

Veličine graničnih opterećenja - dobijene novom metodom - vrlo su bliske veličinama određenim probnim opterećenjem i znatno smanjuju razlike koje postoje pri korišćenju teorijskih rešenja analiziranih u ovom radu.

Odnos graničnih opterećenja određenih teorijskim metodama i određenih probnim opterećenjem betonskih šipova na terenu pokazuje nivo tačnosti analiziranih metoda:

Nova metoda Milovića	0,88-1,08
Mohan i dr.	1,12-4,55
Meyerhof	0,62-3,22
Bustamante i Gianeselli	0,55 -2,43

Razlika između teorijskih rešenja i rešenja pomoću probnih opterećenja, prema oceni Svetskog društva za mehaniku tla i fundiranje, iznosila je ± preko 30%. Na osnovu rezultata iz statičke penetracije (nova metoda), ta razlika znatno je smanjena.

The results obtained by Mohan are higher than the real values. They are between 1. 12 and 4. 55.

The results obtained by Meyerhof are also significantly different than real values obtained by in situ tests and they are situated between 0. 62 and 3. 22.

The results obtained by Bustamante and Gianeselli are showing smaller differences compared to in situ tests but are also higher than real values , between the limits 0.58 and 2.43.

The results obtained by Milovic new method are between the very narrow limits 0. 88 - 1. 08.

3 CONCLUSION

On the basis of the results obtained by new method for determination one may conclude:

The results obtained by static penetration tests show significant dispersion and in some cases values are 4 times higher than those obtained by field load tests;

The values of the ultimate load obtained .by means of new method are very close to the results obtained by field load tests and considerably decrease the difference between the obtained values.

The ratio between the ultimate loads determined by theoretical methods and by field load tests, of concrete piles, shows the level of precision of the theoretical methods:

Milovic`s new method	0.88 - 1.08
Mohan D	1.12 - 4.55
Meyerhof.	0.62 - 3.22
Bustamante and Gianesilli.	0.55 - 2.43

The difference between theoretical solutions and field load tests according to the evaluation of the World Society of Soil Mechanics and Foundations was estimated at ± more than 30%. On the bases of the obtained results from static penetration tests (new method) this difference is cosiderably decreased.

4 LITERATURA
REFERENCES

[1] Bustamante, M. and Giasenelli L. (1982) "P:ile bearing capacity by means of static penetrometar CPT ESOPT H", Amsterdam, Vol, 2, pp. 493 - 500.

[2] Bustamante, M. Frank R. et Giasenelli L. (1987) "Le dimensionement des fondation profondes". Bulletin Liaison Laboratoire des Ponts et Chaussees, 149, pp. 13-22.

[3] Meyerhof, G.G. (1956): "Penetration tests and bearing capacity of cohesionless soils."Journal of the Soil Mechanics and Foundation Engineering, ASCE, Vol. 82, No SM 1, pp. 1 -19

[4] Meyerhof, G.G. (1995): "Behaviour of pile foundations under special conditions."Canadian Geotechnical Journal, Vol. 12, pp: 204 - 222.

[5] Milovic, D. (1986): "Bearing capacity of piles determined by penetration tests. Proc. of the International Conference of Deep Foundations Peking,Vol.1,pp: 2170-2175.

[6] Milovic, D. (1993); "Predicted and observed behavior of piles. Proc of the 2 nd International International Seminar on Deep Foundations on Bored and Auger Piles, Belgium, pp: 381 - 384.

[7] Mohan, D. Jain D. S. and Kumar, V. (1963): "Load bearing capacity of piles". Geotechnique, London, Vol. 13, No 1, pp: 76 - 86.

[8] Poulos, H. G. and Davis F. H. (1980). "Pile foundation analysis and design. John Wiley, New York, pp: 1 - 397.

[9] Poulos H. G. (1989): "Pile behaviour - theory and application"Geotechnique 39, No 3,pp: 365-415. :

[10] Vesic, A, (1972); "Ehpansion of cavities in infinite soil mass."Journ of Soil Mechanics and Foundation Division, ASCE, Vol. 98,pp265 -290.

REZIME

NOSIVOST ŠIPOVA - TEORIJSKE I TERENSKE METODE

Dušan MILOVIĆ

U radu su prikazani rezultati penetracionih ispitivanja kao i terenskih opita probnog opterećenja radi proračuna graničnog opterećenja šipa. U tim ispitivanjima korišćen je kontra teret, koji je dostizao i veličinu primenjene sile čak i do 5,00 MN.

Analizom terenskih i teorijskih rezultata obuhvaćeno je 48 šipova. Primenom prikazane nove metode je postignuto znatno smanjenje razllike izmadju nove metode i i terenskih opita probnog opterecenja

Ključne reči: Nosivost šipova, statičke metode, dinamičke metode, statička penetracija, probno opterećenje šipova, nosivost bazom, nosivost bočnim trenjem.

SUMMARY

BEARING CAPACITY OF PILES - THEORY AND FIELD TESTS

Dusan MILOVIC

In the paper are presented the results of the penetration tests and the field load tests.. In these tests the piles were loaded with the concrete blocks, reaching the vertical force of up to 5.00 MN.

By the analyses of theoretical and field load tests 48 piles were included. By the application of the new method a considerable decrease between the new method and field load tests is achieved

Key words: bearing capacity of piles, static penetration tests, static methods, dynamic methods, field load test, bearing capacity of the base and of the lateral skin friction.

CREEPING (SECONDARY/TERTIARY SETTLEMENTS) OF HIGHLY COMPRESSIBLE SOILS AND SLUDGE

TEČENJA (SEKUNDARNA/TERCIJALNA SLEGANJA) VEOMA STIŠLJIVOG TLA I TALOGA

H. BRANDL

ORIGINALNI NAUČNI RAD
ORIGINAL SCIENTIFIC PAPER
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1 GENERAL

K. Terzaghi and O.K. Fröhlich's theory of (one dimensional) consolidation refers to the dissipation of excess pore water pressure during loading of saturated soil. The time taken for the clay to consolidate depends entirely on the permeability of the laterally confined clay. These assumptions correspond to the primary consolidation in an oedometer test (widely neglecting possible rearrangements of the soil structure already in the initial phase of loading).

At the 1st International Conference on Soil Mechanics and Foundation Engineering 1936 at Harvard University, Cambridge, MA., A.S. KeverlingBuisman presented a theory for creep of fine-grained soft soils. However, this (logarithmic) formula and his statement that creeping of clays never ends was severely questioned, not only by K. Terzaghi (Conference Chairman), but also internationally. Meanwhile this theory has been accepted theoretically and could be widely confirmed, especially by the following test results showing low-term creep, but also a fading out tertiary creep.

2 SETTLEMENT / CREEPING OF HIGHLY COMPRESSIBLE (ORGANIC) CLAYEY SILT

Total settlement of saturated cohesive soil comprises immediate settlements (s_o - undrained, at constant volume), primary settlements (s_1 - consolidated by pore water pressure dissipation) and long-term creeping (s_2, s_3). In the field, all phases interact during transition zones, and creeping under shear stress also occurs. This leads inevitably to soil rheology comprising also cohesionless soils and other geomaterials.

In the design phase (1971 – 1972) of a highway junction on highly compressible soils with locally organic inclusions and peaty interlayers numerous samples were taken and investigated in the laboratory. Several of them were left in the oedometers for long-term creeping tests. The maximum observation period has been from 1971 to 2013, hence 42 years. Some results are described in the following, further investigations and in-situ influences of ground improvement measures are given in the chapter after next.

Table 1 shows the relevant data of a selected sample (A). It is an extremely soft clayey silt (33% < 0,002 mm) with organic components of liquid consistency. The plasticity index of I_p = 0.34 is rather due to the decomposed peaty organics than to the mineralogical composition of the fines as can be seen from Table 2. The platy shape of the fines and its way of sedimentation created a special fabric and high compressibility.

In the natural state the permeability coefficient was about k = 10^{-7} m/s but dropped significantly during loading. At the maximum load finally a value of about k = 10^{-9} m/s was reached. These data explain, among other influence factors, the relatively quick primary consolidation and a long-lasting creeping phase.

BRANDL, H. Emeritus Professor, Vienna University of Technology

Table 1.Geotechnical parameters of organic soil (sample A) and pre-treated sewage sludge (samples B, C). In brackets the years of test start.

		Sample A (1971)	Sample B (1997)	Sample C (2003)
Natural water content	w_n (%)	130	168	131
Unit weight of soil particles	γ_s (kN/m³)	2.52	2.32	2.21
Voidratio	e (-)	4,68	4.06	3.17
Initial dry density	γ_d (kN/m³)	0.44	0.46	0.50
Liquid limit	w_l (%)	92	84	Test not possible
Plasticitylimit	w_p (%)	58	72	
Plasticityindex	I_p (%)	32	12	
Ignitionloss	(%)	25	35	27

Table 2. Mineralogical contents of the organic clayey silt (sample A)

Mica-group	33 %
Chlorite –group	16 %
Quartz	40 %
Feldspar (mainly plagioclase)	11 %

Figures 1, 2 show the void ratio – pressure diagram and the time-settlement curves of the particular load steps. The sample was kept under water to simulate in situ conditions and to prevent settlements by shrinking, Figure 2 illustrates that secondary creep occurred linearly with the logarithm of time until about one year, followed by a transition period to tertiary creep which gradually leads to a fading out of the settlement. Such a behaviour coincides with site observations showing a decreasing gradient of long-term creeping plotted on semi-logarithmic scales. This coefficient was normally considered to be constant. However, even after 42 years no final value has been reached in the oedometer test, thus indicating viscous behaviour and on-going rearrangements of the soil micro-structure, due to tabular sheet silicate in connection with the loss of adhesive water, and microscopic interactions between particles and liquid. Moreover, the compression curve partly consists of segments mutually intersecting in bifurcation points which mark occasional structural collapses. This is schematically indicated in the enlarged detail within Fig. 2.

The oedometer tests were performed with incremental loading, also comprising hydraulic conductivity tests with falling height. The sample height was h = 20 mm, the diameter varied between d = 60 to 100 mm, hence providing a d:h ratio of 3 to 5 (to assess possible skin friction).

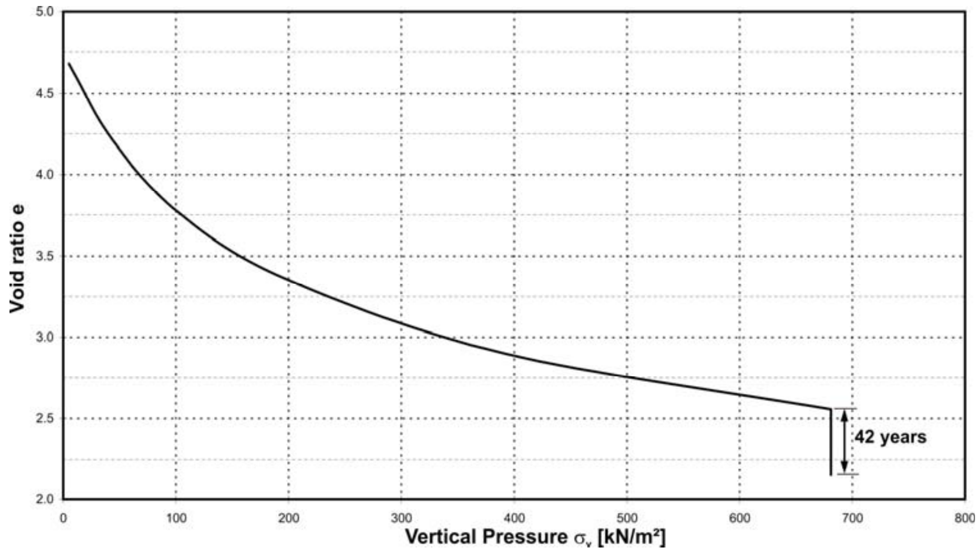

Figure 1. Void ratio – pressure diagram (oedometer test) for organic clayey silt (sample A)

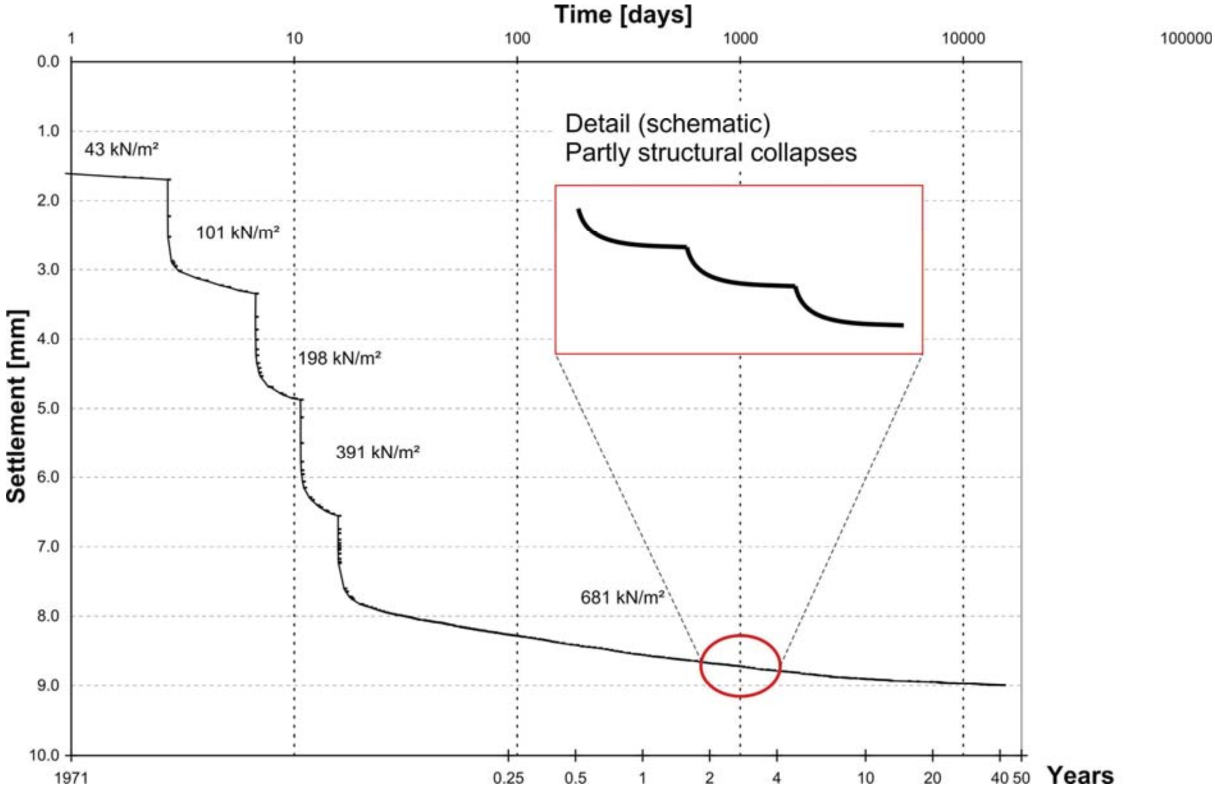

Figure 2. Settlement – log time curve for sample A. Load steps (σ_v) and maximum load during 42 years. Also indicated are occasional structural collapses.

3 LONG-TERM SETTLEMENT OF PRE-TREATED SEWAGE SLUDGE

During the past decades ponds, pit landfills or surface impoundments of liquid sewage sludge have been increasingly substituted by waste deposits of pre-treated sewage sludge, unless this is not incinerated. Such landfills may reach a height of 30 m and more, thus requiring stability analyses, settlement prognoses, assessment of long-term behaviour of the liners, etc. Consequently, these aspects have become a special field of geotechnical engineering.

Suitability tests, starting in the early 1990s disclosed that sewage sludge dewatered in a filter press and stabilized with unslaked lime can be easily deposited in all kinds of waste disposal facilities.

After comparative test series at the filter press and on the landfill with 20 to 45 % CaO, an amount of about 31 % was found to be optimal. Furthermore, 5 to 7 % $FeCl_3$ was added as aflocculant. In the case of sample B 22% CaO was added (referring to the dry mass), in the case of sample C 31 % CaO. When reacting with water, $Ca(OH)_2$ developed, thus creating a highly basic environment. Depending on the untreated sludge properties the hydraulic conductivity first decreased with the amount of added CaO but then increased. However, in the long-term decreasing k-values could be observed also for high CaO addition. This is rather similar to the stabilization of fine soils with lime.

Samples from the undisturbed filter-cake exhibited hydraulic permeability coefficients of only $k = 10^{-9}$ to 10^{-10} m/s. However, after field compaction of the broken filter cake these values increased to an in-situ permeability of about $k = 10^{-7}$ to 10^{-8} m/s, though the dry density was only $\rho_d = 0.45$ to 0.55 g/cm³ (water content usually about w = 130 %). In the long-term in-situ values down to $k = 10^{-9} – 10^{-10}$ m/s were measured, depending on the amount of added lime.

The k-value decreased with time due to mechanical, chemo-physical and biological long-term reactions. In the laboratory, values of $k = 5.10^{-7}$ to 10^{-9} m/s were measured within six months of curing, depending on vertical load and CaO additives. Figure 3 shows an example of long-term tests (running 16 years) together with the scatter of several test series with 25 to 35 % CaO within the first two years at a vertical pressure of 250 kN/m². Hence, pre-treated stabilized sludge can be thoroughly considered as secondary barrier material within the sealing system of a waste deposit. However, compaction in layers is essential.

In order to investigate the long-term behaviour of pre-treated sewage sludge and to find analogies between sludge and soil behaviour several samples were taken. The focus has been on creeping because this has the largest influence on the long-term behaviour of the surface liner of a waste deposit with regard to (differential) settlements.

In the following two examples are selected including long-term oedometer tests running from 1997 and 2003 respectively, until 2013. The samples were always under water and exposed to a constant temperature of 20°C (± 1 °C).

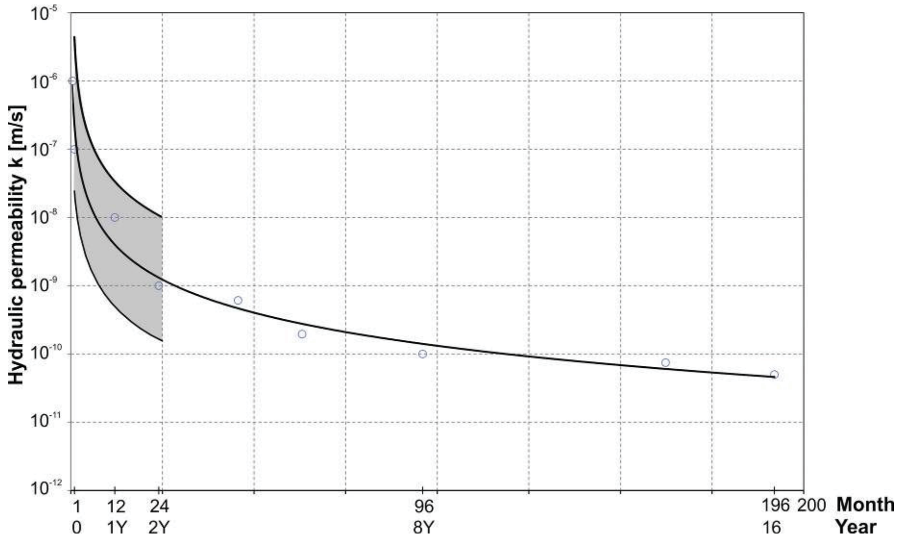

Figure 3. Decrease of hydraulic permeability of pre-treated sludge (25 - 35 % CaO) with time. Scatter of test series within first 2 years and example up to 16 years

Table 1 summarizes the most important geotechnical parameters. The particle size distribution shows "clayey sandy silt" with rather uniform mineralogical contents: Mainly calcite due to the CaO additives, further quartz and some feldspar and layer silicates. Chemical investigations found some concentration of zinc, copper and lead. The material exhibited liquid consistency and zero to low plasticity. The permeability factor was about $k = 10^{-6}$ m/s at the beginning of the compression (oedometer) test under the load step of $p = 30$ kN/m^2 and decreased to about $k = 10^{-10}$ m/s after 15 years under $p = 250$ kN/m^2. The stress-void ratio diagrams show compression curves similar to natural soils (Fig. 4) but less curved and with strong long-term compression under the maximum load.

Figures 5, 6 show the settlement - time diagrams in semi-logarithmic scale. They illustrate that within the first weeks the settlements were rather small, even under the maximum load step. Then they increased significantly, similar to very soft soils. After one year this intensive consolidation was nearly abruptly followed by creeping comprising mechanical, chemo-physical and anaerobe biological reactions. The latter might be the main reason that creeping of sample B fails to occur linearly with logarithm of time but in a slightly convex curve (Fig. 5).

It is noticeable that the hydraulic permeability decreased most in the first year – corresponding to the settlement curve. Long-term pore clogging is influenced by particle rearrangements, lime reactions and possible biological activities. The sample investigated since the year 2003 was obviously lime-saturated: Repeated hydraulic permeability tests with the oedometer caused – mainly in the first phase – some washing out of calcitic particles.

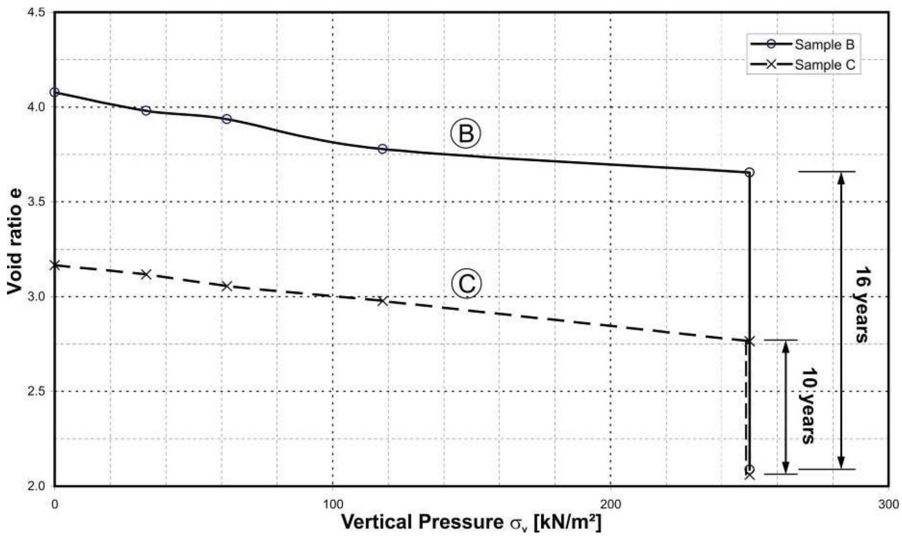

Figure 4. Void ratio – pressure diagram (oedometer tests) for pre-treated sewage sludge (samples B, C).

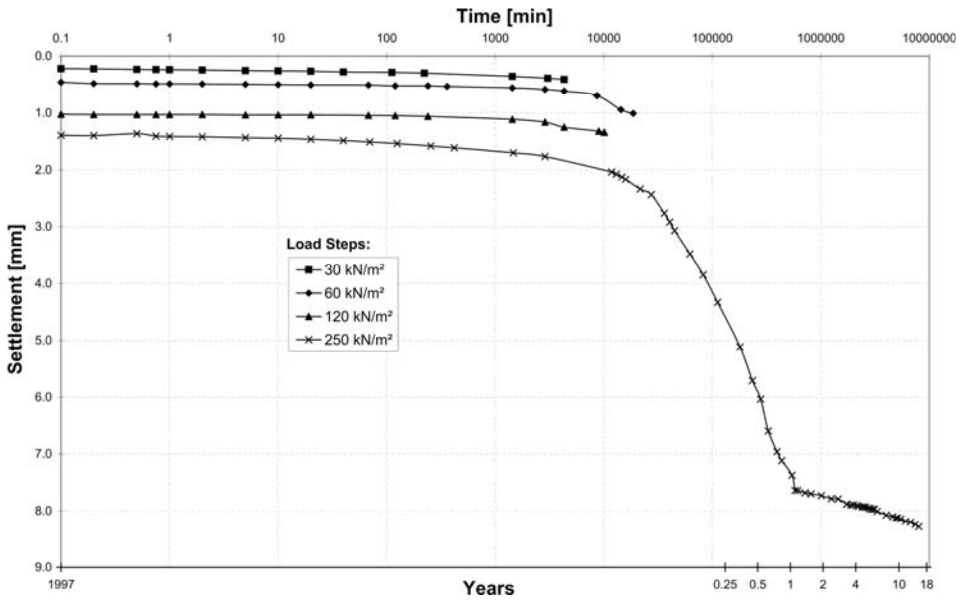

Figure 5. Settlement – log time curves for sample B and increasing load steps (max. 16 years)

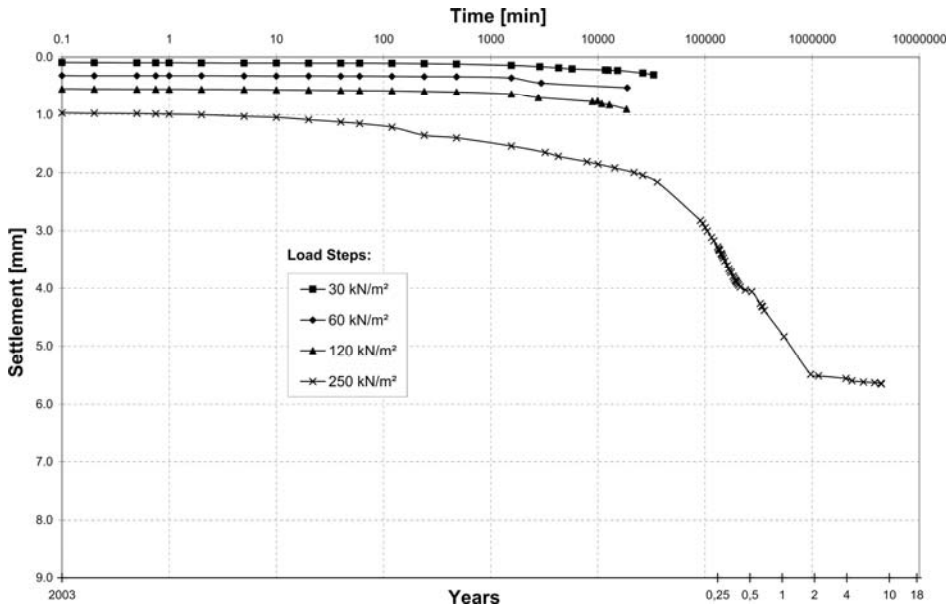

Figure 6. Settlement – log time curves for sample C and increasing load steps (max. 10 years)

Creeping continued until the end of the oedometer tests, i.e. up to 16 years without coming to the end. This clearly indicated a long-term rearrangement of the sludge structure despite the hardening effect of added lime. Similar behaviour could be found for inorganic clayey silt and silty clay stabilized with lime, when cured under water-saturated conditions. However, creeping of such soils faded out at least within ten years. In both cases (sludge and soil) the creeping value (i.e. the gradient of the settlement line) dropped with increasing amount of added lime.

Chemo-physical and anaerobe biological reactions of sludge explain a long-term creeping of sewage sludge, which sometimes differs from natural soil or peat. Though the absolute values are small, the settlement - log t correlation is unlikely a straight line but slightly curved downward – depending on organisms and chemistry (e.g. Fig. 5). Nevertheless, settlements in the oedometer cannot proceed indefinitely.

Unloading of the long-term oedometer tests showed only small swelling. This is due to the high amount of added CaO and the non-active mineralogical contents.

The hitherto field observations confirmed the results of laboratory and in-situ tests. Primary consolidation of the waste deposit occurred already during the several years lasting landfilling process, and long-term creeping is no problem for the sealing cover. It is smaller than under saturated laboratory conditions because of gradual carbonatisation of the material.

4 INFLUENCE OF GROUND IMPROVEMENT ON LONG-TERM SETTLEMENTS

Between 1972 and 1974 a large highway interchange was constructed on highly compressible heterogeneous ground (Tauernautobahn, Austria). It comprised embankments up to 8 m height, max. 2.5 m deep excavations mostly in peat and 8 bridges. The following ground improvement methods were applied (details see Brandl, 2006):

- Deep dynamic compaction/consolidation (heavy tamping),
- Vibroflotation,
- Temporary surcharge loading,
- Local combinations of the previous methods.

Deep dynamic compaction by heavy tamping has been used in Austria and Germany since the 1930s, but was first limited to granular materials, drop weights of about 10 tons and drop heights of about 10 m. Significant development started at this construction site in 1972/73 with 20 to 25 tons falling from heights up to 22.5 m to improve soft or loose soils respectively and peat to a depth of about 14 m. This required special crawler cranes and in advance fill layers as working platform.

An impact "consolidation" of more or less water saturated (organic) clayey silts and peat was considered "impossible" at that time as being completely contradictory to K. Terzaghi and O.K. Fröhlich's consolidation theory. Fortunately, the owner (Austrian Federal Ministry) could be convinced to allow an increased geotechnical risk in the frame of research and development, and to reduce costs and construction time. Intensive site observations and measurements disclosed that the excessive impacts created from heavy tamping on the soil caused particle rearrangements, local soil liquefaction and steep shear surfaces where vertical drainage up to the ground surface occurred (like artesian water). This behaviour was favoured by (micro)gas bubbles in the soft soil: 100% water saturation is hardly measured in practice, even in inorganic fine-grained soils below groundwater. This could be observed on numerous construction sites.

The thickness of the highly compressible and heterogeneous layers varied between 3 to 16 m, comprising peat, clayey to sandy silt, silty sand (locally with gravel), and finally sandy gravel. Organic interlayers were found down to 15 m below original ground. The groundwater level depended strongly on weather and season with a mean value of approximately 2 m below surface.

Laboratory tests and in-situ measurements provided compression moduli down to $E_s = 0.2$ MN/m² and a natural water content up to about w_n = 1000%. The saturation degree varied between 75 to nearly 100%, clearly increasing below groundwater table and with depth. The liquid limit lay between w_L = 20 to 600%, the plasticity index between I_p = 0 to 250%. These extremely poor ground conditions led to settlements up to about 5 meters already during the construction process. Further details can be obtained from (Brandl, H. 2006).

Oedometer tests on organic soils showed a significant tendency to creeping. According to Figure 7 a creeping coefficient for secondary settlements was derived. The transition from primary to secondary settlement is indicated in Figure 7 by an idealized line, but actually occurred within a longer period. Figure 8 shows that the creeping coefficient varied within a very wide range. Several oedometer tests ran over a period of 20 years and one test up to 42 years (from 1971 to 2013). These long-term investigations have disclosed that secondary and tertiary creep may continue extremely long if the fine-grained soil has a high void ratio and organic components. The mineralogical composition of the fines is influential as well.

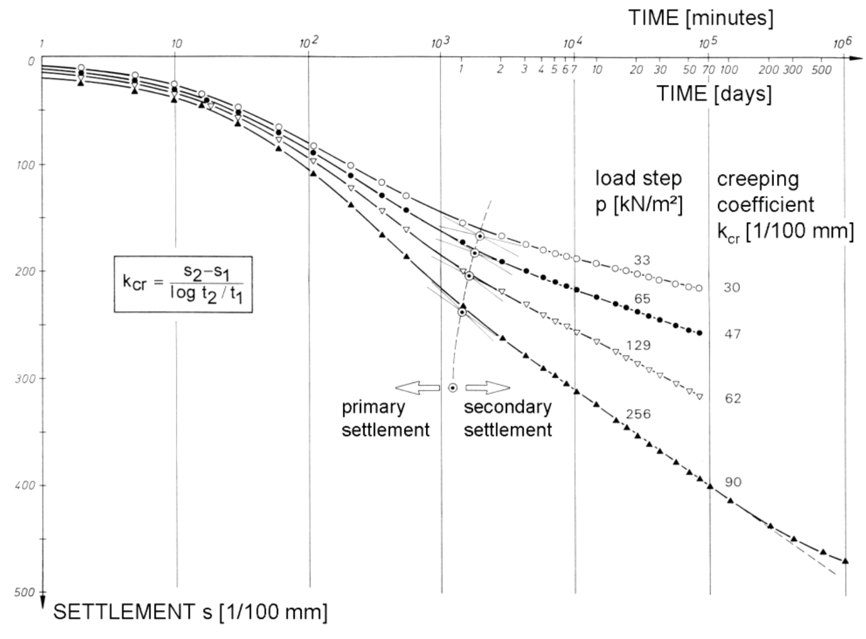

Figure 7. Time-settlement curves of decomposed peat and definition of the creeping coefficient k_{cr} (derived from oedometer tests).

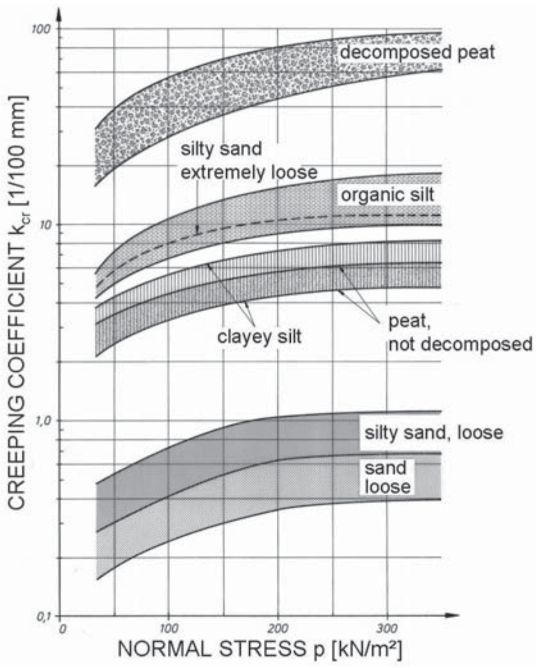

Figure 8. Creeping coefficient k_{cr} for several soils describing secondary settlement (creeping).

The scheme of Figure 9 illustrates the compaction procedure typically applied for the embankments, whereas sections below original ground surface required a partial soil exchange before heavy tamping. Due to the heterogeneous subsoil and varying embankment heights or cut depths respectively the required compaction energy varied in a wide range with a maximum of approximately E = 2500 tm/m². Deep compaction control was performed mainly by comparing pressuremeter values before and after heavy tamping. Figure 10 shows an example illustrating the influence depth of heavy tamping and the effects of the embankment weight and time. The influence depth of heavy tamping varied between 8 to 14 m depending on particular soil properties and energy input.

Figure 11 presents the settlements of an interchange section where seven series of heavy tamping and a temporary surcharge load on the embankment were applied. The final road pavement was installed approximately 16 years after opening of the highway. The secondary settlements within this period did not affect the highway traffic as they occurred rather uniformly.

Figure 12 however, shows this change of the interchange where the maximum settlement occurred after heavy tamping and embankment construction. This required periodical re-levelling despite the construction of a higher level of the pavement already before opening

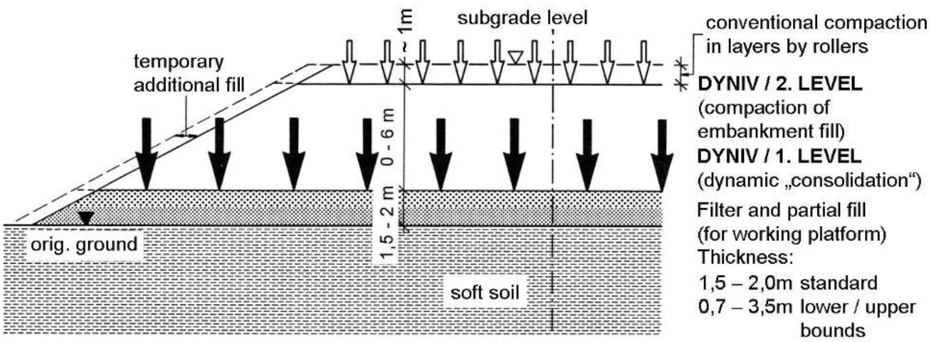

Figure 9. Standard procedure of heavy tamping at a highway interchange performed in the years 1972/1973

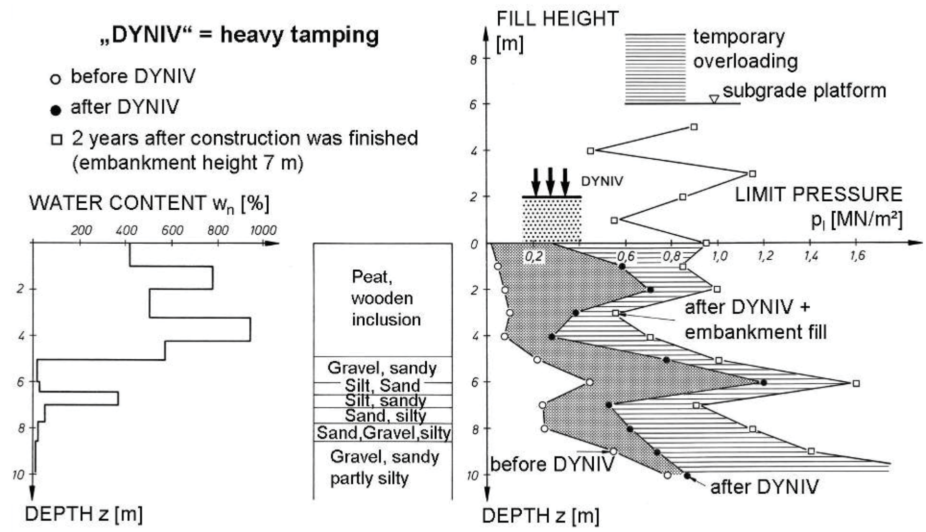

Figure 10. Example of in-situ pressuremeter tests before and after heavy tamping, and two years after the embankment had been constructed

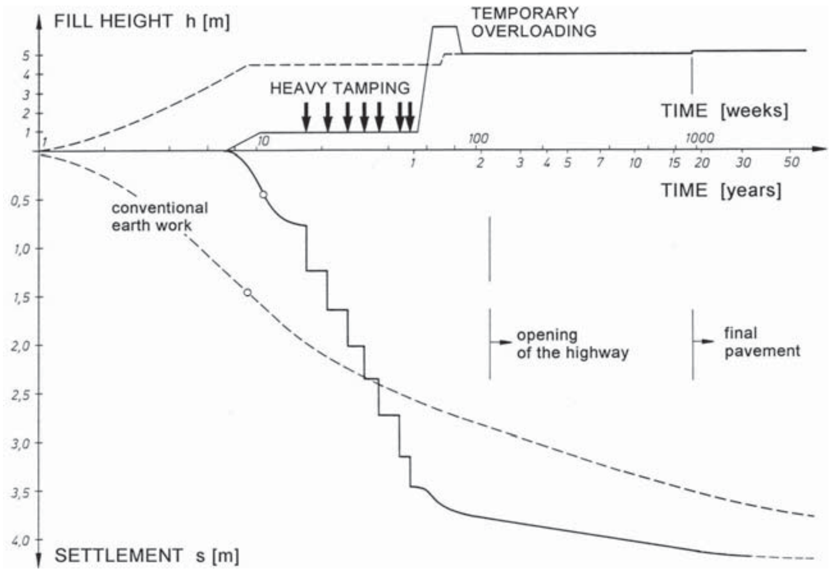

Figure 11.Time-settlement curve of an embankment section. Influence of heavy tamping and temporary surcharge load on the level of the embankment crown

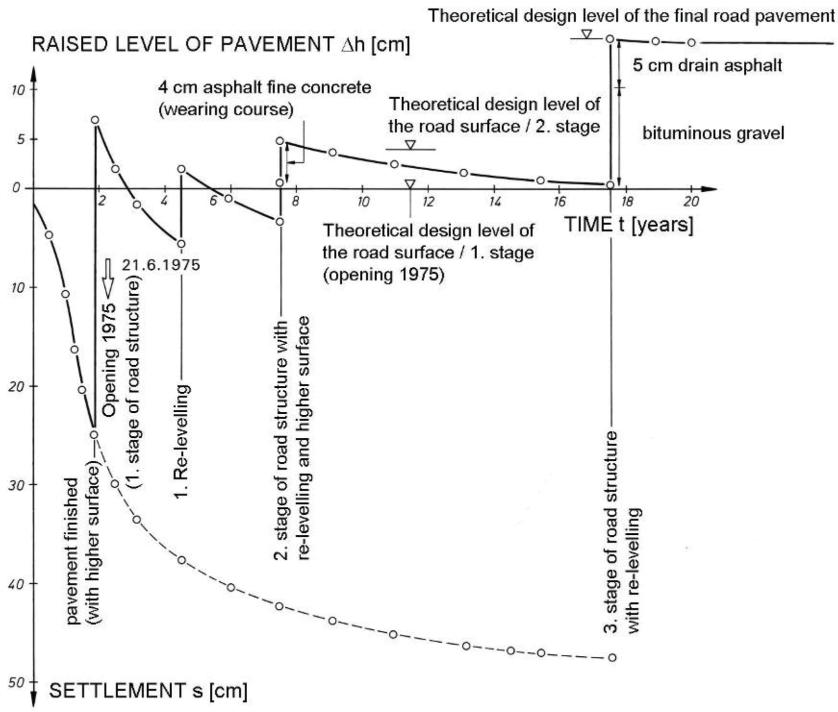

Figure 12. Time-displacement curves (related to the design level of the road surface) involving periodical re-levelling and installation of additional surfacing layers to achieve sufficient driving comfort (Section 10).

Design speed for car traffic: v = 150 km/h

for the traffic (compensation for expected long-term settlements). But only the first measure (in 1977) was an additional one; the other re-levelling procedures were performed in connection with the installation of the final layers of the road pavement according to the original design (remediation of wearing courses, placing drain asphalt etc.).

The long-term behaviour of this highway interchange may be summarized as follows:

The project was a pioneer work regarding heavy tamping and piled embankments. Previous experience with weights of 20 to 25 tons dropping from heights up to 22.5 m did not yet exist worldwide, and the fine-grained, organic ground with a water content up to 1000% was another challenge. Despite these unfavourable conditions a satisfactory long-term behaviour of the entire interchange could be achieved. The maximum total settlement (including anticipated deformations by heavy tamping and temporary surcharge loading of the embankments) was approximately 5 m which occurred mainly during the construction period. Local re-levelling of the primary (provisional) road surface on the basis of the contractor's quality guarantee was necessary only once, namely 2.5 years after opening of the highway. This measure was limited to some sections of heavy tamping only.

Long-term creeping after highway opening varied between 5 to 20 cm and occurred rather uniformly. The piled embankments resting on stone columns with compound body cover (geosynthetics, cement stabilization, crushed rock) behaved even better than the sections with heavy tamping. However, the ground properties were somewhat better there. Temporary surcharge loading of the embankment proved to be also very successful, especially in connection with previous heavy tamping.

According to Austrian highway guidelines and codes the definite surfacing of the road pavement was placed approximately 4.5 years after opening of the highway (2^{nd} stage of road structure). The final surfacing, 16 years after opening, involved the placement of a new road structure with a more traffic resistant wearing course above the old structure. This remediation was required primarily because of the long-term degradation of the road pavement (deep traffic ruttings etc.) due to heavy traffic. The influence of differential settlements was negligible. However, both road surfacing measures (4.5 and 16 years after highway opening) involved also a re-levelling.

To sum up, the long-term behaviour of this highway interchange has been very satisfactory for about 40 years now. The design speed of v = 150 km/h could be maintained during the entire period. The settlement prognoses based on laboratory and field tests, on analytical calculations, on empirical parameters and experience have been in good accordance with the measured values. Long-term creeping is still going on but negligible for traffic comfort and maintenance.

Comprehensive field observations have disclosed how method and quality of deep soil improvement influence primary consolidation and creeping of soils. Consequently, if a ground tends to strong creeping (observed in laboratory tests), soil improvement technologies have to be properly selected or adapted, resp. For instance, vertical drains accelerate only pore water dissipation during primary consolidation, but fail to improve creeping behaviour.

Prediction of primary and secondary settlements was important for constructing a temporarily higher level of sub grade and asphalt surface of the highway junction running on the embankments on highly compressible ground (“compensation fill”).

Several comparative tests showed that Atterberg limits or activity index, resp. are insufficient as a criterion for creep assessment, because soil creep depends on numerous factors: Grain size distribution, mineral optical composition, moisture content, permeability, density, fabrics structural strength, viscosity, and external factors lead to an extremely complex process.

5 CONCLUSIONS

All three-phase systems containing particles, liquids and gas exhibit creep under compressive stress. Secondary creeping of clayey soft soils mostly occurs linear with the logarithm of time. However, temporary increase may also be observed, indicating a discontinuous nature of internal deformations due to accelerated rearrangement in the fabric – mainly in soils with peaty components. A micro-mechanical explanation is ductile sliding between mineral crystals followed by repeated structural ruptures.

Long-term oedometer tests on soils and pre-treated sewage sludge have revealed several similarities between natural and artificial fine materials of high compressibility. The tests ran up to 42 years and showed a gradual transition from secondary to tertiary creep for organic clayey silts after about one year. During tertiary creep the gradient, plotted on semi-logarithmic scale, gradually decreased. This could be found also for inorganic clays under site conditions, where the gradient may eventually approach zero.

In pre-treated sewage sludge the transition from primary to secondary consolidation is more significant than in soils. No fading out tertiary creep could be observed in the semi-log diagrams of oedometer tests. This could be explained by chemo-physical and anaerobe biological long-term reactions in this material.

Several other comparative tests have confirmed that organic soils show pronounced secondary / tertiary creeping, and that creeping also depends on the mineralogical composition and the arrangement (microscopic structure) of the fines, and not only on grain size distribution, initial porosity, consistency, etc. Consequently, prognoses of creeping derived from oedometer tests (with site-specific data) are still more reliable than those from exclusively numerical modelling (with data from the literature) or uncertain correlations. Additionally, investigations based on geochemistry and electron microscope analyses might be helpful.

Finally, the long-term tests have disclosed, that K. Buisman’s creep theory is widely consistent, although tertiary creep has to be added and settlements cannot run indefinitely. The application of oedometric results for the prediction of creeping in the field has to consider lateral displacements and shearing and possible measures of ground improvement. Three-dimensional field conditions accelerate creeping in relation to one dimensional oedometer tests (with stiff side walls).

6 REFERENCES

[1] Brandl, H. (2006). "Ground improvement and earthwork innovations for transportation infrastructure".Active Geotechnical Design in Infrastructure Development. XIII Danube-European Conference on Geotechnical Eng., CIP-Ljubljana, Vol. 1, 217-232.

[2] Buisman, A.S.K. (1936). "Results of long duration settlement observations". Proceedings of the 1st International Conference of the ISSMFE (International Society for Soil Mechanics and Foundation Engineering), Cambridge, Vol. 1, 103-106.

[3] Terzaghi, K. and Fröhlich, O.K. (1936). „Theorie der Setzung von Tonschichten". Franz Deuticke, Leipzig – Wien.

[4] Havel, F. (2004). „Creep in soft soils". Doc. Thesis. Norwegian Univ. of Science and Technology, Trondheim.

SUMMARY

CREEPING (SECONDARY/TERTIARY SETTLEMENTS) OF HIGHLY COMPRESSIBLE SOILS AND SLUDGE

H. BRANDL

The paper focuses on long-term oedometer tests lasting up to 42 years and performed on silty sand, (organic) clayey silt, peat and (pre-treated) sewage sludge. Secondary consolidation (creep) could be observed in all cases, lasting over many years and occurring widely linear with the logarithm of time. This long-term phase is followed by tertiary creep with a long lasting fading out period. In addition to the laboratory tests results of comprehensive field observations are summarized, showing the influence of ground improvement on the creeping behaviour of very soft fine-grained soils (partly organic). The data were collected from a highway junction on highly compressible, heterogeneous ground (with natural water content up to 100%), constructed between 1972 and 1974, and monitored since.

Key words: long-term settlements, creeping, highly compressible soils, oedometer tests, heavy tamping, deep soil improvement, sludge

REZIME

TEČENJA (SEKUNDARNA/TERCIJALNA SLEGANJA) VEOMA STIŠLJIVOG TLA I TALOGA

H. BRANDL

Ovaj rad je usmeren na dugotrajna edometarska ispitivanja koja su trajala 42 godine i izvedena su na prašinastom pesku, (organskoj) glinovitoj prašini, tresetu, i (pre obrade) kanalizacionom talogu. Sekundarna konsolidacija (tečenje/puzanje) mogla je da bude uočena u svim slučajevima i trajala je mnogo godina i ispoljavala se, uglavnom, kao linearno zavisna od logaritma vremena. Ovu dugotrajnu fazu prati tercijarno tečenje sa dugotrajnim periodom vremena do konačnog nestajanja. Osim laboratorijskih opita, sumirani su rezultati sveobuhvatnih terenskih ispitivanja i oni pokazuju uticaj poboljšanja tla na tečenje vrlo mekog sitnozrnog tla (delimično organskog). Ovi podaci su prikupljeni na raskrsnici autoputa na veoma stišljivom heterogenom zemljištu (sa prirodnim sadržajem vode do 100%) koja je izgrađena između 1972 i 1974 i od tada je osmatrana.

Ključne reči: dugotrajna sleganja, tečenje (puzanje), jako stišljivo tlo, edometarski opiti, dinamičko zbijanje, poboljšanje dubljih slojeva tla, talog

USE OF PILOT TUNNEL METHOD TO OVERCOME DIFFICULT GROUND CONDITIONS IN KARAVANKE TUNNEL

UPOTREBA METODOLOGIJE PROBNOG TUNELA ZA PREVAZILAŽENJE TEŠKIH USLOVA GRADNJE U TUNELU KARAVANKE

Vojkan JOVIČIĆ

PREGLEDNI RAD
REVIEW PAPER
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1 INTRODUCTION

The tunnel Karavanke is some 7,9km long single tube tunnel, which is located at European corridor 10, European motorway road E61. It is the only remaining tunnel at the corridor 10 and also at the Slovenian network of motorways, which provides for the traffic in both directions in a single tube. As such, the tunnel is in breach of the directive of European Council 2004/54/ESof 2004. According to the directive, each tunnel longer than 1000m must have an escape route in the form of evacuation adit or the second tube, which can be also used for single way traffic.

The tunnel presents the most frequent traffic link between Slovenia and Austria. It is the last and the longest tunnel on the northern arm (Ljubljana – Jesenice) of the Slovenian motorway network. In historical terms the tunnel plays a significant role in connecting the Middle with Southern Europe as the link passes beneath some 2500m high Karavanke chain of mountains. Approximately half of the tunnel, that is some 3,5km, is on the Slovenian side, the rest is in Austria.

As will be explained in some detail, the construction of the first tube, which took place some 30 years ago, was met with many challenges. These challenges will remain for the construction of the second tube but will take different and sometimes more demanding forms. The main challenges for the construction of the second tube are summarised as follows: a) large convergence displacements in squeezing rock conditions, and b) the huge inflows of water during the excavation. Both of these challenges can be reasonably addressed by the use of the pilot tunnel method, which in this case provides several advantages in the comparison to the traditional NATM (New Austrian Tunnelling Method) division of the tunnel to top heading, bench and invert.

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2 PILOT TUNNEL METHOD

The pilot tunnel method is based on a construction of a small-diameter tunnel, which is driven parallel to the axis of a much larger main tunnel. Pilot tunnel can be located near the crown, bench, invert, or rarely outside the layout of the main tunnel to provide access to critical locations.

The main purposes of the pilot tunnel method are numerous and are summarised as follows: a) investigating the nature and behaviour of rock mass, b) exploring adequate excavation techniques, c) introducing new support procedures, d) treating or improving the ground prior to construction of the main tunnel, e) dewatering of the rock mass, f) enabling gradual stress relief for control of displacements and others. Pilot tunnels are used primarily in difficult ground conditions, in which this method is proved to be very useful. Kavadass (1999) reports successful use of the method for the ground pre-treatment with a Tube-a Manchette grouting from a pilot tunnel ahead of the tunnel face in Athens metro. The method, schematically presented in Figure 1, was successful in drastically reducing ground settlements and was used extensively during the construction of underground metro stations where the tunnels passed below buildings of the Old City of Athens.

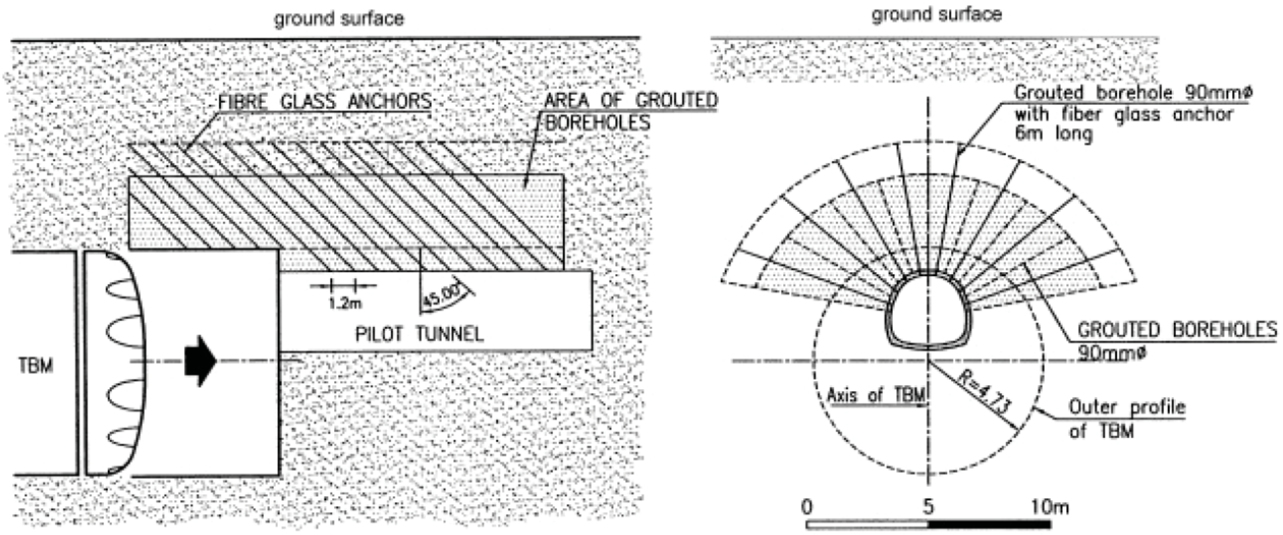


Figure 1. Ground pre-treatment from a pilot tunnel in the Athens metro (after Kavvadas, 1999)

As it will be explained in continuation, during the construction of the second tube of Karavanke tunnel there would be difficult sections in which the use of pilot tunnel would be necessary and fully justified. These are the conditions expected firstly in the zone of squeezing rock and secondly in the zones of the crossing of the aquifers, in which the large inflows of water are expected. In the first case the pilot tunnel is used to activate gradual stress relief caused by the excavation, so that the development of displacements happens in stages and is thus more controllable. In the second case the pilot tunnel is used to enable the room for the drainage measures to dewater the rock mass, which is a precondition to treat and improve the rock mass under the controllable conditions.

3 EXPECTED GEOLOGICAL CONDITIONS IN THE SECOND TUBE

Following the needs for the main design of the second tube the comprehensive site investigations were carried during the years 2015 and 2016. This information was complemented with very detailed geological mapping, which was carried out during the excavation of the first tube (Budkovič, 1999). As already indicated earlier, the geological conditions in the Karavanke tunnel were difficult and variable, in a sense that geological units are changeable at small distances. The main lithological units, which were found along the tunnel axis were Permian and Carboniferous clastic rocks with limestone lenses; Middle Permian clastic rocks with brecciated and limestone rock and Upper Permian clastic rocks within Triassic development of Carboniferous clastic rock. Main tectonic units developed in directions (E)-(W) are intersected with several, almost vertical, faults in the directions (NE)-(SW) and (NW)-(SE)(Geološki zavod Slovenije, 1988).

The prediction of the longitudinal geological section along the second tube is presented in Figure 2. The following geological units are isolated at the section: QMO – Quaternary sediments (chainages km 7.8+21 to 7.5+53), glacial moraine and weathered rock formations (sand and gravel with silt parties and larger carbonate blocks); ST – Lower Triassic Werfen formation (chainages km 7.5+53 to 6.9+54), built by oolithic limestone, marl limestone and sandstone; P – Permian layers (chainages km 6.9+54 to 6.1+56) with characteristic Bellerophon formation (dolomit) and Gröden formation (quartz conglomerate, sandstone and slate clay stone), PC – UpperCarboniferous and Lower Permian layers(chainages km 6.1+56 to 5.1+13) in the form of limestone, quartz conglomerate, sandstone and slate clay stone and T –Upper to Lower Triassic layers (chainages km 5.1+13 to 4.3+76) made of Rabelj formation (marl, marl-limestone and limestone) and Schlern formation (breccia and dolomite).

Generally, the geotechnical model of the second tube of Karavanke tunnel on the Slovenian side is divided into the five sections: Section 1 – low overburden in moraine and weathered rock material, Section 2 - – Lower Triassic Werfen formation with average overburden of 530m, in which high water inflow is expected, Section 3 – Permian and Carboniferous clastic rock with low capacity and high deformability under average overburden of 680m impying squeezing rock conditions; Section 4 – Triassic section with relatively stable conditions but with water bearing fault zone on the end and Section 5 - Triassic dolomite section in stable conditions (Budkovič, 1999).

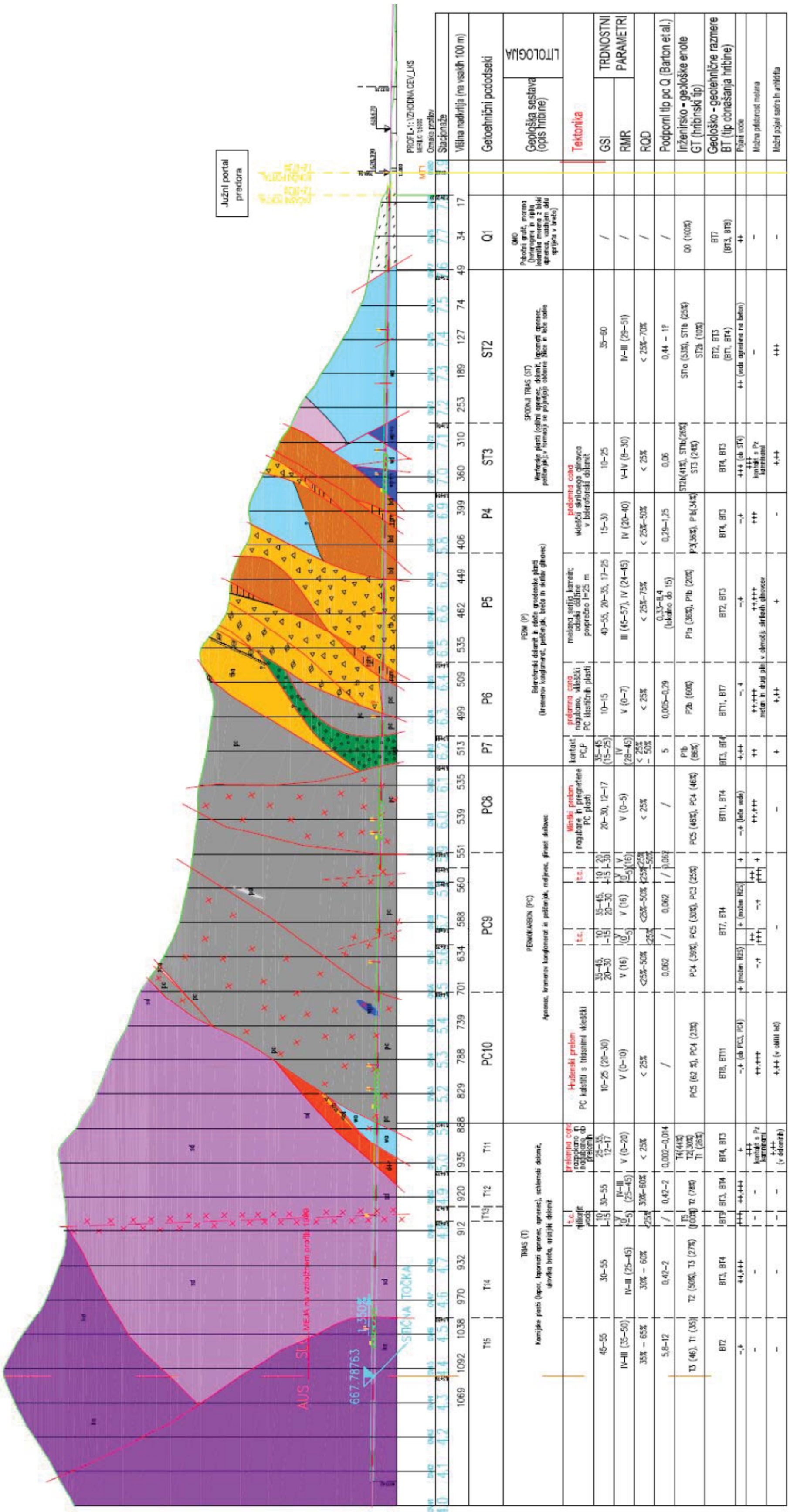


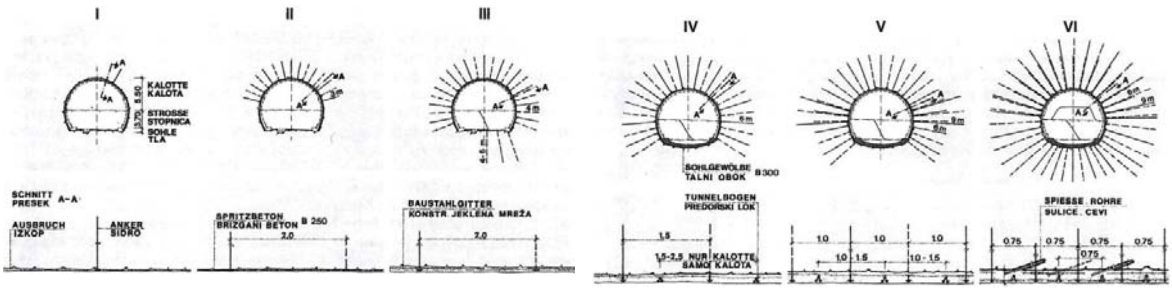
Figure 2. The longitudinal geological section along the second tube with rock mass characterisation

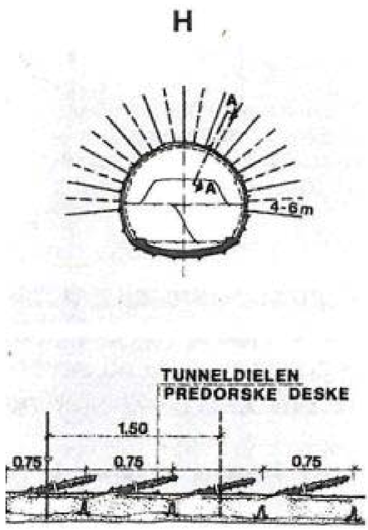
4 CONSTRUCTION OF THE FIRST TUBE

The design of the primary support of the existing tunnel was carried out according to the principles of NATM (New Austrian Tunnelling Method). The profile of the excavation was divided generally into top heading, bench and invert. The invert was not installed along the full length of the tunnel. It was an estimate at the time that the NATM is the adequate method for tunnel construction in difficult ground conditions, which were readily anticipated. On the basis of the devised longitudinal geological section the ground conditions for the Slovenian side of the tunnel were divided into the six categories and each category had its own support system, as shown in Figure 3 (Mikoš, 1991). The additional support system was developed for the loose ground, which was expected in the zone of shallow overburden, in which the moraine material dominated.

The first support category (KRH1) was envisaged for the stable rock mass condition, which actually did not occur during the excavation. The second category (KRH2), envisaged for the ˝broken rock mass ˝was used only up to 3,6% of the total length of the tunnel while the category (KRH3) envisaged for ˝ broken, spilling and folded˝ rock mass was used in 4,6% of the tunnel. Majority of the tunnel construction, some 40,2% was carried out in the category (KRH4), which was envisaged for ˝broken rock mass with rock pressure˝, while in the fifth category (KRH5) for the ˝heavily broken rock mass with heavy rock pressure˝ 25,4% of the tunnel was executed. The sixth category (KRH6) was used in the conditions of ˝heavily broken rock mass with heavy rock pressures and strong water inflows", which was undertaken along 17,6% of the tunnel length. Finally, the support category for the loose ground, which was seen mostly in the zone of shallow overburden, counted for approximately 8,6% of the tunnel excavation. In general terms, according to the comprehensively written overview of the tunnel construction presented by Mikoš (1991), particular difficulties were caused by the presence of the squeezing rock conditions, the occurrence of methane and the strong water inflows.

The difficulties started immediately during the excavation at shallow overburden in moraine materials, which was extremely heterogeneous. The roof protection was carried out using the 3,5m long spears while some top heading instabilities also occurred. The large inflows of water started immediately on the transition into the rock mass material. In the continuation the strong inflow of water of some 100 litres per second was encountered at the chainages of 732 to 746 m. The water inflows were followed by the local instabilities and the wash out of the crushed and lose stone. According to Mikoš (1991), during the further advancing through the reddish gröden layers there were no difficulties. These started again at the transition to Permian and Carboniferous clastic rocks, which occurred at the chainages of around 1450m. Here the condition of squeezing rock prevailed, which caused the failures of the tunnel lining in the diagonal direction relative to the tunnel axis. The deformations were put under control after the installation of the additional anchors and the construction of the invert. The section through Carboniferous slates was particularly demanding with higher squeezing pressures so that the 50cm deformation gaps in the tunnel lining had to be introduced to preserve the integrity of the tunnel support (Budkovič, 1993).





REGELSTÜTZMITTEL PRO LFM TUNNEL
PREDPISANA PODPORNA SREDSTVA PO TEKOČEM METRU PREDORA

GKL HK	AUSBRUCH IZKOP [m³]	SPRITZBETON B 250 BRIZG. BETON [cm]	ANKER JE LFM SIDRA PO TM [m]
I	85,3	5-10	4,5
II	85,3	5-10	11,25
III	86,5	10	38,5
IV	96,5	15	50,0
V	97,8	20 5*	108,0 6,0*
VI	100,4	25 15 SOHLE/TLA 5*	369,9 12,0*
H	99,1	25 5*	35,0

* BRUSTSICHERUNG
ZAVAROVANJE ČELNE POVRŠINE

22 Podporni ukrepi za hribino od I - H kategorije, po projektu in izvedbi.

Figure 3. The overview of the excavation categories for the first tube (Mikoš, 1991)

At the chainage 1700m the tunnel excavation was fully in Carboniferous clastic rock. Instead of the expected 35cm of total convergence movement these were accelerating in the top heading at a rate of around 17 cm per day (Mikoš, 1991). Large and fast deformations were pulling out the anchors and the anchor plates were sheared off. For this reason a new anchor head was introduced, which allowed for 20cm of axial deformation before the full capacity of the anchor was activated. Also the deformation gaps were introduced into the lining so that more of the load was transferred to the rock mass before was taken by the tunnel lining. The extreme deformations were encountered at the transition from sandstone parties within the clastic rocks into the much weaker Carboniferous slates. These movements were at some points up to 150cm so that some remedial works on the primary lining were inevitable despite all the measures that were undertaken to prevent this. The occurrence of the high concentration of methane was detected between the chainages 1560m and 2600m. This required a particular safety measures for the work under the methane regime, which additionally slowed down the progression (Mikoš, 1991). The presence of methane was detected by using the pre-drilling procedures, which were systematically used along this difficult section.

Carboniferous section ended at the chainage of 2550m. The next section was significantly easier featuring limestone and schlern dolomites. The high overburden, which was at this point some 800m, and the high inflow of water did not caused particular difficulties. By the rule, after the excavation, the inflow of water quickly ceased and the predrilling, which was continuously used also at this section, was an effective measure to instrument the drainage.

These conditions prevailed up to the chainage 3030m, in which the pre-drilling indicated that an aquifer zone lies ahead featuring extremely high water pressures. The additional boreholes were installed at the head of excavation, but these were clogged almost immediately and it was clear that the water pressure build up behind the top heading would inevitably cause an incontrollable and dangerous failure. At this point the human workforce and the machines were moved far out from the top heading and the failure was caused remotely by the controlled blast. The sudden inflow of huge amount of water flooded the tunnel. There was an estimate that 4500m^3 of the material was washed out and that the initial inflow was some 1m^3 per second (Mikoš, 1991).

Once the inflow of the water became controllable and constant the major remedial works started. Gabions were used to ensure the stability of the locally damaged tunnel lining. The water pressure relieve boreholes were further installed at the head of excavation to enable controllable drainage conditions. Finally, the bypass pilot tunnel of smaller dimensions was built along the deviation of the tunnel axis, which revealed a major fault zone that was channelling the water inflow. More pressure relief boreholes were installed from the bypass pilot tunnel towards the main axis. After the progression through the fault zone the pilot tunnel was re-directed along the tunnel axis and the works advanced within the relatively simple geotechnical conditions with no further delay. At the position of the fault zone, the head of the excavation of the main tunnel was injected and stabilised and the breakthrough of the main tunnel within the fault zone was carried out in fully controllable manner.

The continuation of the excavation up to the state border was relatively undemanding as the last 400m of the tunnel construction were carried out within the hard limestone and dolomites with occasional sections of marl and sandstone.

5 USE OF PILOT TUNNEL METHOD IN THE SECOND TUBE

Given the complex geological structure and the experience from the construction of the first tube described in the previous sections the following challenges are expected during the construction of the second tube: a) large convergence displacements in the squeezing rock conditions and b) huge inflows of water. Both of these conditions can be partly or fully addressed using the pilot tunnel method, as it will be explained in continuation.

(i) Large convergence displacements

During the construction of the first tube the large convergence displacements were first encountered in extreme form at the chainage 1450m in which there was a transition from Permian to Permian-Carboniferous rock in the form of clay slate structure. The deformations that were measured along the tunnel are shown in Figure 4. As it can be seen in the figure even more extreme deformations, up to 1,5m were experienced at the chainage of 1700m. The trend of high displacements continued along the full length of Permian-Carboniferous section with similar magnitude of deformation (Mikoš, 1991).

The philosophy of the NATM method is based on the notion that the lining needs to be flexible so that majority of the load caused by the relaxation of the initial stresses is taken by the surrounding rock mass. This is very difficult to achieve in the condition of the squeezing rock in which the ratio between the height of overburden and the uniaxial strength of the rock mass is very high. This implicitly leads to high and wide plasticisation of the rock mass around the cavity and premature installation of the tunnel lining will result in the loss of lining integrity.

The measures that are predicted to cope with large convergence displacements under the conditions of squeezing rock include the use of deformation gaps (once they close the lining start taking the load) which are integrated in the lining. They can be made to be load bearing, which can help in controlling the rate of the convergence movements and thus transfer of the force from the rock mass to the tunnel lining.

The next measure is the use of the rock anchors with ductile rods, which enables them to take the full load even after the significant level of deformation. The control of the large convergence displacements can be carried out to a certain extent by the careful sequencing of the excavation of the top heading bench and the invert. The closing of the invert should be carefully chosen once the activation of the lining is nearing to the full capacity. For this purpose the load cells are predicted to be installed in the deformation gaps so that the process can be monitored in real time and the adequate decision can be taken on time and within the required tolerances. The typical cross section, in which supporting measures are presented in the zone of squeezing rock conditions, in which the large convergence displacements are expected, is shown in Figure 5

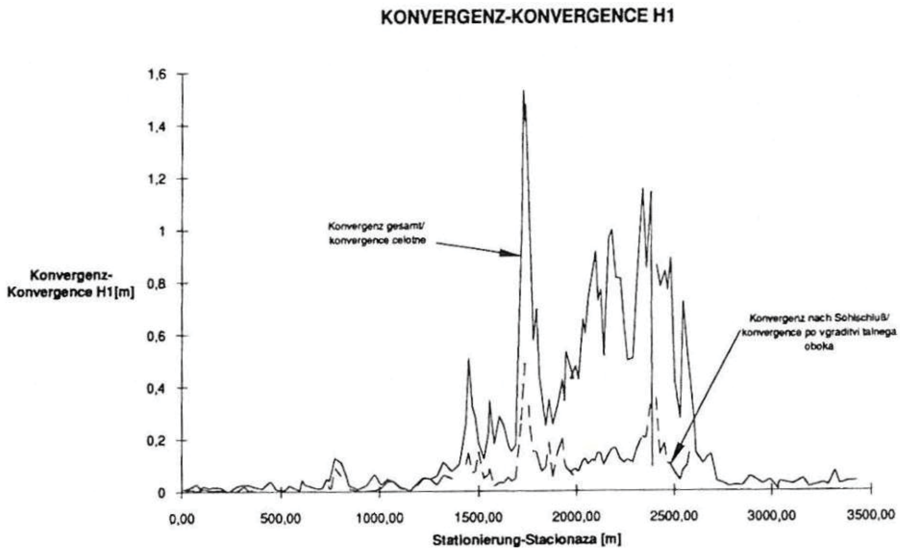


Figure 4. Themagnitude of convergence movement experienced during the construction of the first tube(after Mikoš 1999)

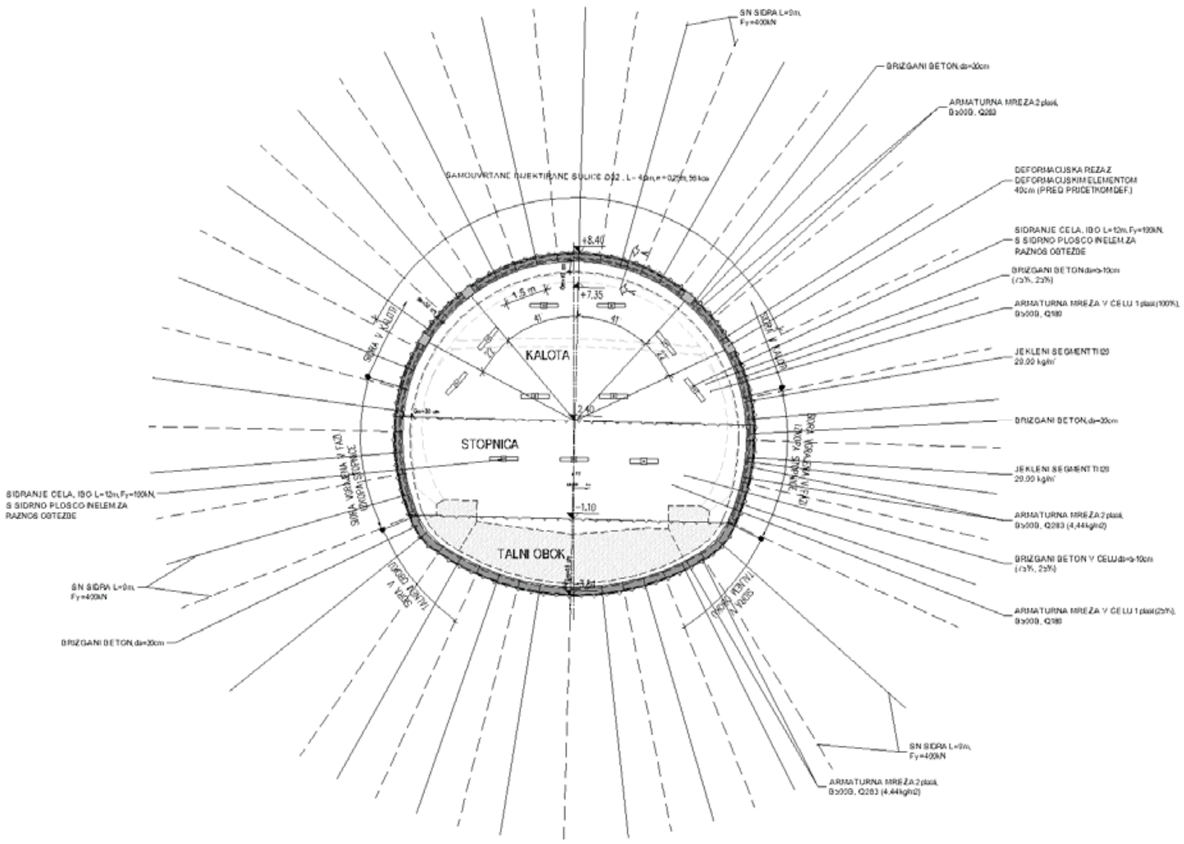


Figure 5. Support system for the second tube for the of squeezing rock conditions

Finally, if all the measures numbered above are still inadequate to control the displacements in squeezing rock conditions a pilot tunnel can be introduced with an aim to activate gradual stress relief caused by the excavation. By the use of the pilot tunnel the development of the displacements becomes more controllable as the pilot tunnel allows for partial unloading within the future cavity, which is of a smaller diameter and so easier to stabilise. Once the stable conditions are established in pilot tunnel the further treatment and improvement of the rock mass could be carried out to control the final amount of displacements.

The decision to construct a pilot tunnel will depend on the geological conditions currently met in the second tube, in particular on the conditions of efficient drive in terms of displacements caused by squeezing rock, and the risk of possible detrimental influence on the functionality of the existing tube.

(ii) Large inflows of water

As it was explained before, during the excavation of the first tube the large inflows of water were at some point almost insurmountable obstacle for the construction of the tunnel. The hydrogeological report (IRGO, 2014) based on the new site investigation and the observation of the current state of the drainage in the existing tube located seven aquifers that are relevant for the tunnel. The most water bearing aquifer, which caused the large inflow of water and stopped the excavation of the first tube, is the highly permeable and fissured Schlernian dolomite aquifer that is heavily influenced by the Goliški fault. The aquifer has a free water table and is characterised by the large differences in permeability, around 100 times higher at some locations along the certain parts of the fault zone. This situation enables the channelling of the large quantities of the water so that the water pressures of up to 75 bars can be found at the deep fault layers reaching the elevation of the tunnel.

During the excavation of the first tube the maximum recorded inflow was in the Schlemian dolomite aquifer with some 5000 litres per second (Mikoš, 1991). The other inflows were drained relatively quickly, after three to four months, during the construction of the tunnel. However, after 25 years of the drainage provided by the tunnel the Schlemian dolomite aquifer was not drained. At the moment, it provides with the inflow of some 60l per second (Brenčič, M. &Poltnig, W., 2008).It is anticipated that the hydrogeological conditions would be much more favourable during the excavation of the second tube in the comparison with the first. The difference between the axis of the tunnels is some 40 to 70 metres so that the drainage system of the tunnel represents some form of the regulated drainage of the aquifers, which can be also felt in the second tube. This is the reason to expect the significantly lower water inflows in the second tube during the excavation.

Treating and improving of the rock mass prior to construction of the main tunnel is the main reason to construct pilot tunnel at the locations of the crossing the aquifers. From the experience of the excavation of the first tube the dewatering of the rock mass can take several weeks prior to re-establishment of the normal working conditions. While the pilot tunnel is used for dewatering, the drive of the main tunnel can continue at the normal pace in front of the pilot tunnel. In this case the decision to use the pilot tunnel will depend on the volumes of the water ingresses that can have a potential

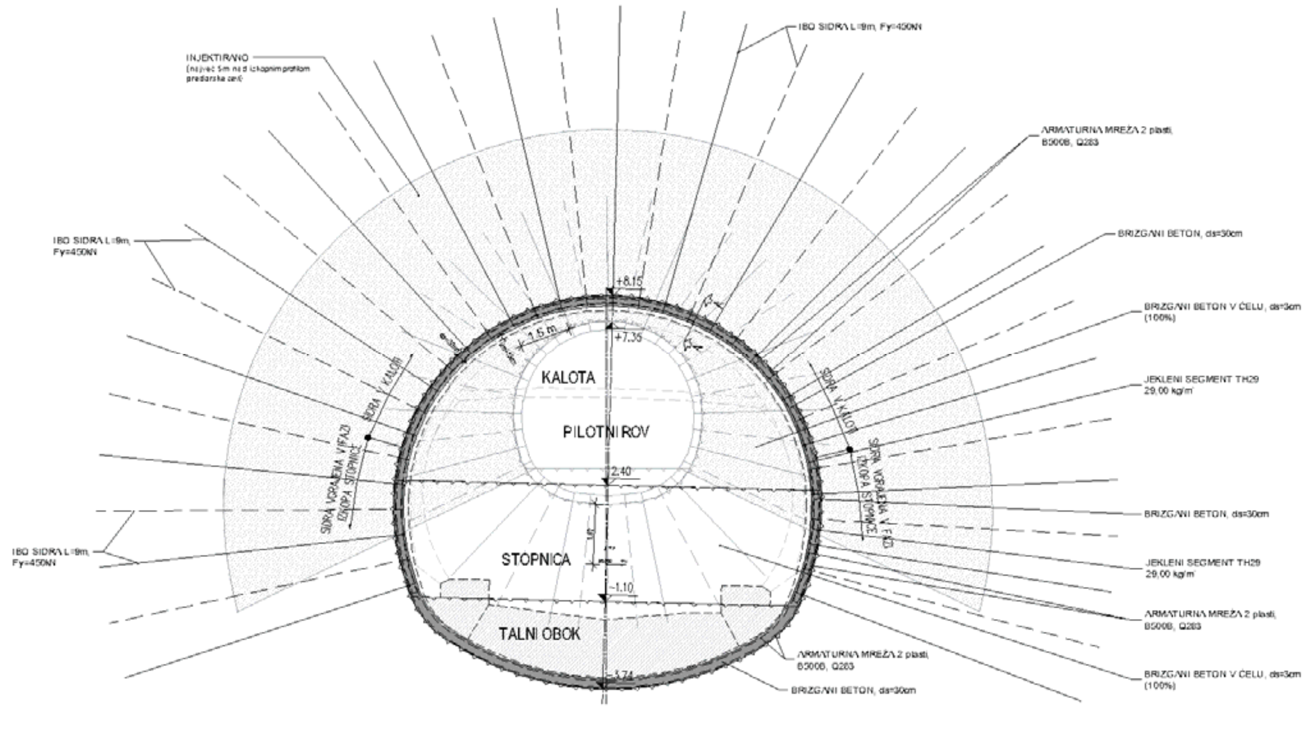


Figure 6. The cross section of the pilot tunnel relative to the main tube.

to stop the drive. At any rate, the relative cost of the other exploration methods, the cost and the time required for its construction, the value of its benefits and the available funds should all be considered prior to the final decision to use the pilot tunnel method.

Several measures were devised in design stage to control the inflow of the water during the construction of the second tube and to prevent the flooding of the tunnel that occurred during the excavation of the first tube. Pre-drilling will be used systematically along the excavation of the tunnel. The pressures will be monitored during the pre-drilling and the pressure relief boreholes will be installed if needed.

For the transition through the Schlemian dolomite aquifer, in which the largest ingresses of water are expected the use of a pilot tunnel is predicted. The cross section of the pilot tunnel is shown in Figure 6. The pilot tunnel is designed to occupy approximately one third of the excavation surface in the comparison with the main tube. The utilisation of the pilot tunnel had several purposes. The first one is to enable for the controlled drainage of the Goliški fault so that the efficient pressure-relief boreholes can be installed at the appropriate places. The second purpose is to cause partial stress relief in the area of the fault so that the tunnel lining of the full profile can take lesser load than otherwise. Finally, after the completion of the drainage measures the pilot tunnel can be used to improve the local rock mass in the fault zone and improve the stability of excavation of the main tube. The treatment comprises grouting of the rock mass as it is expected that the rock mass will be weakened by the wash out of the debris caused by the inflow. The longitudinal section of the pilot tunnel within the area of ground improvement is shown in Figure 7.

6 CONCLUSIONS

The tunnel Karavanke is some 7,9km long single tube motorway tunnel, which is located at European corridor 10 connecting Slovenia and Austria. The construction of the first tube, which took place some 30 years ago, was met with many difficulties. These difficulties are seen as new challenges for the construction of the second tube, which is about to start this year.

The main challenges for the construction of the second tube are the large convergence displacements in squeezing rock conditions and the huge inflows of water. Both items have a potential to undermine the function of the existing tube, which would be under the traffic load all the time. These issues are addressed in the design of the second tube by using the pilot tunnel method. The basic concept of the pilot tunnel method is described in the paper. This method, which is based on a construction of a small-diameter tunnel, which is driven parallel to the axis of a much larger main tunnel is occasionally used in difficult ground conditions. Among the other purposes, the pilot tunnel method can be used for treating or improving the ground prior to construction of the main tunnel.

The applicability of the pilot tunnel method for the excavation of the second tube of Karavanke tunnel is discussed in the paper. General conditions of the excavation of the second tube are presented by geological data, which were derived from the mapping during the construction of the first tube and the additional site investigations. The historical records of the construction of the existing tube are also presented in some detail. The key events during the construction of the first tube are highlighted, including the convergence displacement of up to 80cm in the sections of squeezing rock and the large ingress of water experienced during the crossing of the Schlemian dolomite aquifer.

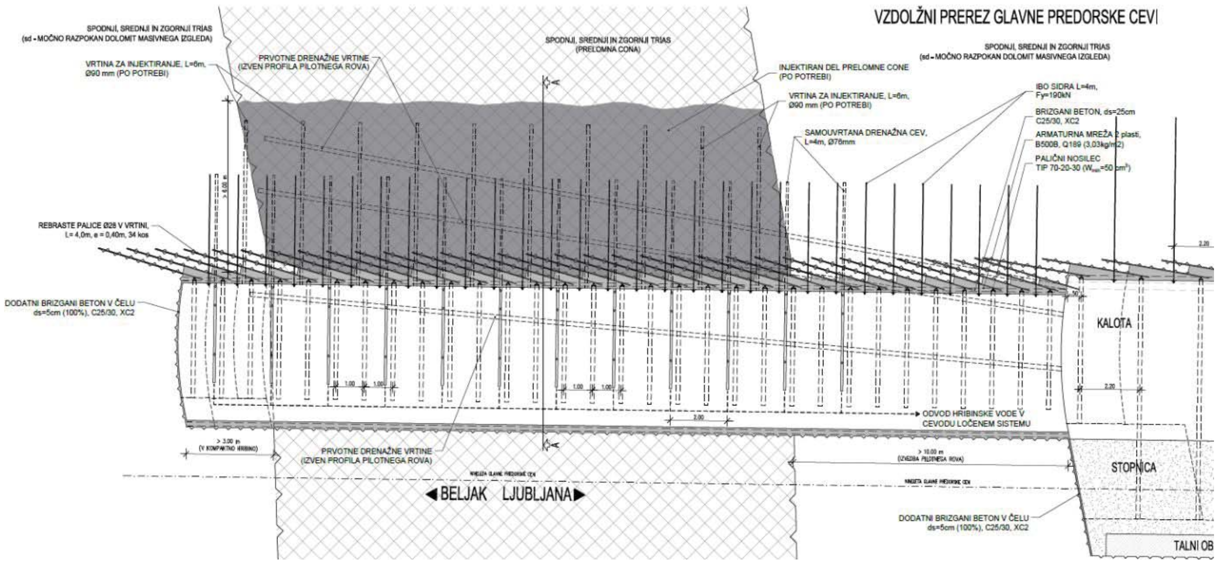


Figure 7. The longitudinal section of the pilot tunnel shown relative to the main tube: drainage and grouting measures are used to improve ground condition in the area of fault.

The possibilities of the use of the pilot tunnel method are presented on the examples of the design of the second tube of Karavanke tunnel. To control the large convergence displacements in squeezing rock conditions a pilot tunnel can be introduced with an aim to activate gradual stress relief caused by the excavation. By the using this method the development of the displacements becomes more controllable as the pilot tunnel allows for partial unloading within the space of future cavity of the main tunnel. The second and more developed example is related to treating and improving of the rock mass prior to construction of the main tunnel. This would be needed at the locations of the crossing the water bearing aquifers such as Schlemian dolomite aquifer, which effectively stopped the drive of existing tube for several months. While the pilot tunnel would be used for dewatering, the drive of the main tunnel could continue at the normal pace in front of the pilot tunnel. Once the drainage and the improvement of the rock mass concludes the construction of the main tube can be carried out within the controllable conditions without delay.

7 LITERATURE

[1] MIKOŠ, B. 1991, Cestni predor Karavanke. Republiška uprava za ceste, Ljubljana in Tauernautobahn AG, Salzburg, Frohnweller Druck GesmbH, april 1991, 65 p.

[2] MIKOŠ, B. 1991, Predor Karavanke, Geologija in geotehnika, Cestni inženiring p.o., Herausgeber, 1991, 72 p.

[3] Kavvadas M. 1999, Exeriences from the construction of the Athens metro project, Proc. 12th European Conference of Soil Mechanincs and Geotechnical Engineering, Amsterdam, June 1999, Invited lecture, Vol 3, pp 1665-1676

[4] IRGO, 2014, Hidrogeološko poročilo za predor Karavanke, Dograditev AC Predora Karavanke – Predor, Idejni projekt, oktober 2014, 56 p.

[5] Direktiva Evropskega Parlamenta in Sveta 2004/ 54/ES, Uradni list Evropskeunije, april 2004, 21p.

[6] Brenčič, M. & Poltnig, W. 2008. Podzemne vode Karavank / Grundwasser der Karawanken. Geološki zavod Slovenije &J oanneum Research Forschungsgesellschaft, 144 p.

[7] Budkovič, T. 1993. Geologija Karavanške gacestnega predora. Magistrskanaloga. Ljubljana, Univerza v Ljubljani, Fakulteta za naravoslovje in tehnologijo: 62 p.

[8] Budkovič, T. 1999. Geology of the Slovene Part of the Karavanke Road Tunnel. Gabhandlungen der Geologischen Bundesanstalt, 56/2; p. 35-48.

[9] Jovičić, V., Boštjan, V.: Design of the second tube of Karavanke Tunnel, Proc. of the 7th International Conference "Geotechnics in Civil Engineering", Ed. R. Folić, Šabac, 14-17. Nevember 2017. pp. 37-52.

[10] Geološko poročilo, (Geološki zavod Slovenije, 1988), arh.št. 194.

SUMMARY

USE OF PILOT TUNNEL METHOD TO OVERCOME DIFFICULT GROUND CONDITIONS IN KARAVANKE TUNNEL

Vojkan JOVICIC

The pilot tunnel method was used in design of Karavanke tunnel to address the two main challenges, which are expected during the construction of the second tube. The first challenge is large convergence displacements in squeezing conditions of Permian and Carboniferous clastic rock of low capacity and under high overburden. The second challenge is the large ingresses of water, which are expected at the fault zones when crossing the aquifers, which are numerous and abundant with water. The paper describes the rationale behind the use of pilot tunnel method and gives an overview of the purposes of the installation, including the background information on the applicability of the method in Karavanke tunnel. The conditions for the construction of the Karavanke tunnel are described firstly through geological conditions for the excavation of the second tube and secondly on the basis of historical records obtained during the construction of the first tube, which took place some 30 years ago. Finally, two examples of the design application of pilot tunnel were given for particular sections of the tunnel explaining the usability of the method for the given conditions.

Key words: pilot tunnel method, squeezing rock, large ingress of water, tunnel construction

REZIME

UPOTREBA METODOLOGIJE PROBNOG TUNELA ZA PREVAZILEŽANJE TEŠKIH USLOVA GRADNJE U TUNELU KARAVANKE

Vojkan JOVIČIĆ

Metoda probnog tunela je upotrebljena za projektovanje tunela Karavanke sa ciljem rešavanja dva ključna izazova koja se očekuju tokom predstojeće gradnje druge cevi. Prvi izazov predstavlja iskop tunela u uslovima iztiskivanja stenske mase u permo-karbonskoj klastičnoj stenskoj masi niske nosivosti koja se nalazi pod velikim nadslojem. Drugi izazov predstavljaju veliki dotoci vode, koji se očekuju u prelomnim zonama na mestima prolaza kroz vodonosnike, koji su brojni i izuzetno bogati sa vodom. U članku su objašnjeni opšti koncepti za primenu metode probnog tunela kao i upotrebljivost metode za uslove gradnje tunela Karavanke. Očekivani uslovi iskopa tunela su predstavljani pomoću geoloških uslova koji se očekuju tokom gradnje druge cevi a zatim i pomoću uslova iskopa zabeleženih tokom gradnje prve cevi pre 30 godina. Konačno, dva primera projekta primene probnog tunela su prikazana za dva odseka tunela sa ciljem da se predstavi upotrebljivost metodologije za date uslove gradnje.

Ključne reči: metodologija probnog tunela, iztiskivanje stenske mase, veliki dotoci vode, gradnja tunela

STATIC AND DYNAMIC EVALUATION OF ELASTIC PROPERTIES OF SOFIA SAND AND TOYOURA SAND BY SOPHISTICATED TRIAXIAL TESTS

STATIČKO I DINAMIČKO VREDNOVANJE ELASTIČNIH SVOJSTAVA PESKA IZ SOFIJE I TOJOURA SOFISTICIRANIM TRIAKSIJALNIM OPITOM

Nikolay MILEV
Junichi KOSEKI

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1 INTRODUCTION

A well know fact is that the ground deformation in every day working condition is usually less than 0.1% strain. In soil mechanics a normal assumption is that the ground consists of a continuum and that its behaviour is linear and recoverable within very small strain range i.e. less than 10^{-3}%. Therefore „elastic“ deformation properties of soil such as Young's modulus and maximum shear modulus play important role in civil engineering design. In order to obtain these paramaters through in-situ tests it is common to use corss-hole logging, down hole and suspension sonde methods while resonant column, torsional shear and triaxial tests as well as bender elements are commonly used as laboratory tests to evaluate these properties.

In this study Toyoura sand and Sofia sand having various dry densities have been subjected to cyclic triaxial tests. Relatively very small unloading-reloading cycles have been applied at several stress states and strains have been measured locally by means of local deformation transducers (LDTs), [6], at the side surface of the specimen. This method is called „static“ herein. For the „dynamic“ measurement two types of wave propagation teqniques have been adopted. One is using bender elements and the other is composed of trigger-elements which transmit shear wave and two ceramic accelerometers which receive the shear wave. Based on these „static“ and „dynamic“ measurements elastic moduli of soil are compared with each other focusing on the following topics: 1) the difference between the two types of dynamic measurements and 2) the relations between dynamic and static measurement results.

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2 TESTED MATERIAL, EQUIPMENT AND TEST PROCEDURES

2.1 Specimen preparation and apparatus

All laboratory tests have been performed at the Geotechnical Laboratory of the University of Tokyo (Institute of Industrial Science – Komaba Campus) – [10]. Basic physical and mechanical properties are obtained by convetional tests. More sophisticated to determine parameters of soil (elastic moduli) have been evaluated by means of custom eqiupped triaxial apparatus (Fig. 1). Table 1 and Table 2 summurize the performed tests. Fifteen cyclic triaxial tests with shear wave velocity measurment in total have been performed at various confining stress and relative density.

Table 1. Test list for Sofia sand

Test List			
Test №	*Material*	σ'_c [kPa]	D_r [%]
Test 103	Sofia Sand	100	70
Test 104		50	70
Test 105		100	70
Test 106		100	90
Test 107		50	90
Test 109		100	90
Test 110		100	90
Test 111		100	90
Test 112		100	90

Table 2. Test list for Toyoura sand

Test List			
Test №	*Material*	σ'_c [kPa]	D_r [%]
Test 100	Japanese Toyoura Sand	80	50
Test 101		100	50
Test 102		50	50
Test 114		100	50
Test 115		100	50
Test 116		100	50

The main purpose of the tests is to evaluate the different methods for obtaining the Young's modulus and maximum shear modulus of soil and make a comparison between them.

Two types of material have been tested: one is typical Bulgarian sand from Sofia plateau (called “Sofia sand” herein) and the other is well studied over the years soil (reference material in many papers) - Japanese sand from Yamaguchi prefecture (called “Toyoura sand” herein).

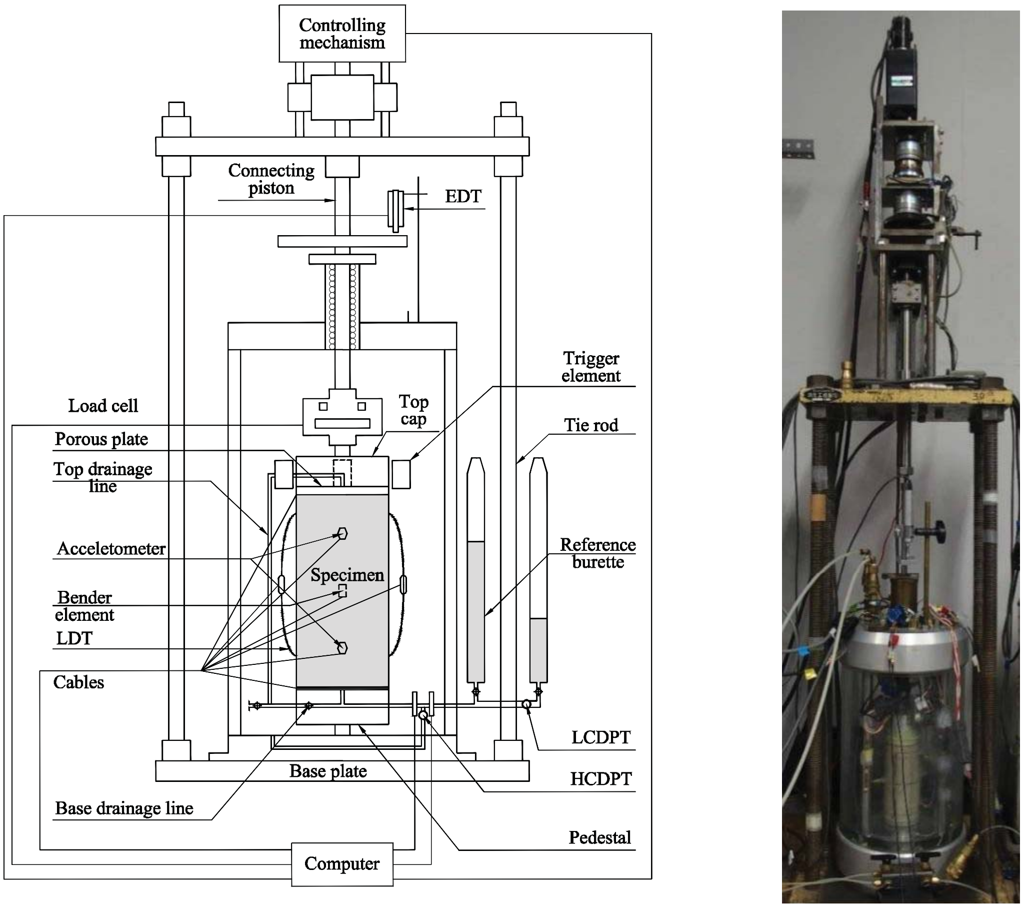


Fig. 1 Sophisticated triaxial apparatus (Geotechnical Laboratory of “Komaba” Campus of the University of Tokyo – Institute of Industrial Science)

Fig. 2. Photograph of Sofia sand

Sofia sand is beige yellowish soil from Lozenetz region which dominant minerals are: amphibole, epidote minerals, titanite, zircon, tourmaline and rutile (Fig. 2) – [1]. Its physical and mechanical properties are shown on Table 3 and its grain size distribution is shown on Fig. 4.

Toyoura sand is obtained from the Toyoura beach in Yamaguchi prefecture (Japan) and consists mostly of quartz (over 85÷90%) and limestone, mica and other materials (Fig. 3). This material is uniformly graded (with almost no particles with diameter less than 75 μm) and with round particles. Toyoura sand is a widespread material for testing especially in Japanese laboratories. It has been well studied during the last few decades and has become a reference (standard) material. The physical and mechanical properties of this kind of sand are shown in Table 4 and its size distribution is presented in Fig. 4.

Table 3. Physical and mechanical properties of Sofia sand

Specific density	Dry density	Void ratio	Maximum void ratio	Minimum void ratio	Relative density	Mean particle diameter	Fines content	Coefficient of uniformity	Angle of shearing resistance
ρ_s	ρ_d	e	e_{max}	e_{min}	D_r	D_{50}	F_C	C_U	
[g/cm^3]	[g/cm^3]	[-]	[-]	[-]	[%]	[mm]	[%]	[-]	[°]
2.68	1.40	0.918	1.390	0.866	90	0.22	4.24	2.19	38.46

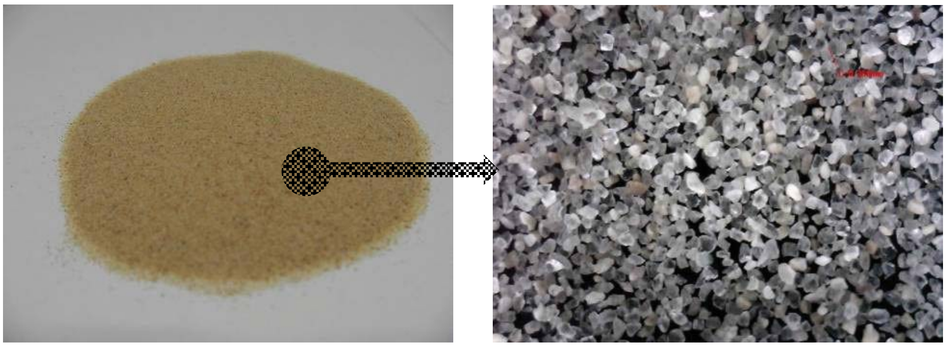


Fig. 3. Photograph of Toyoura sand

Table 4. Physical and mechanical properties of Toyoura sand

Specific density	Dry density	Void ratio	Maximum void ratio	Minimum void ratio	Relative density	Mean particle diameter	Fines content	Coefficient of uniformity	Angle of shearing resistance
ρ_s	ρ_d	e	e_{max}	e_{min}	D_r	D_{50}	F_C	C_U	
[g/cm^3]	[g/cm^3]	[-]	[-]	[-]	[%]	[mm]	[%]	[-]	[°]
2.65	1.47	0.801	0.989	0.613	50	0.21	0.19	1.20	36.87

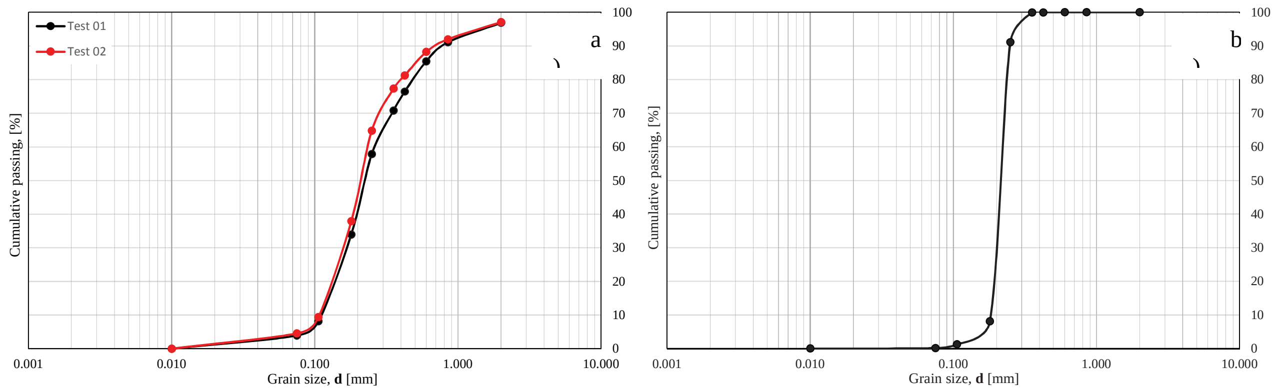


Fig. 4. Grain size distribution curves of: a) Sofia sand; b) Toyoura sand

The standards JGS 0541-2009, JGS 0542-2009 and ASTM-D3999-11 have been adopted for the performance of the cyclic loading triaxial tests and the interpretation of their results. The soil specimens have been prepared in accordance with JGS 0520-2009 and the below described sequence has been followed:

1) A latex membrane with 0.3 mm thickness is slipped on the pedestal (Fig. 5a) which is equipped with a porous plate. The membrane is marked with a pen in order to set the spots on which the transducers would be set on a later stage of the test and then the membrane is attached to the pedestal by silicone and rubber bands (Fig. 5b);

2) The pedestal and the membrane are enclosed in a steel mold made of two parts in order to ensure the cylindrical shape of the specimen. The two parts of the mold are screwed together by means of a metal bracket and the connection between them is isolated through special grease;

3) The top end of the membrane is folded over the mold (Fig. 5c);

4) Negative pressure of -30 kPa is applied so that the membrane is vacuumed to the mold.

5) Since the used material is sandy soil (cohesionless) the “air-pluviation” technique [9] has been adopted for the specimen preparation. It is possible to create a very uniform specimen of dry poorly graded coarse-grained soils through slow pluviation. In the "air-pluviation” method the material is placed in a container in this case a mold of 75 mm in diameter and 150 mm in height at a specific vertical distance (depending on the relative density which is aimed) above the specimen surface. The feed door is opened and the material is allowed to rain down in a slow constant stream. The hopper is continuously traversed across the specimen depositing a thin layer of material with each pass. The process is continued until the specimen mold is overfilled by about 1 cm. The top surface is formed with a straight edge;

6) The top cap is dropped down until it touches the top surface of the soil specimen and after that it is locked in order to avoid damaging the sample;

7) The top end of the membrane is slipped over and attached to the top cap through silicone and rubber bands;

8) The negative pressure of -30 kPa is transmitted to the soil specimen through the pedestal and the top cap and the metal mold is removed (Fig. 6a);

9) The top cap is supplied with counterbalance system and after that it is unlocked. The counterbalance ensures the absence of tension and compression in the specimen which is measured by means of a load cell – [13]. The top cap is locked once again and the counterbalance is removed;

10) Transducers for small strain measurement, bender-elements and accelerometers are attached to the specimen (Fig. 6b) – [7];

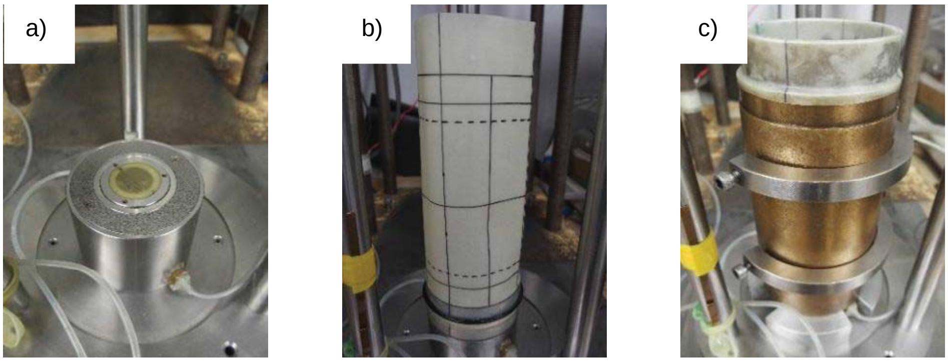


Fig. 5a) Preparation of the pedestal; b) Attaching of the membrane to the pedestal; c) Folding of the membrane over the metal mold used for preparation of the soil specimen

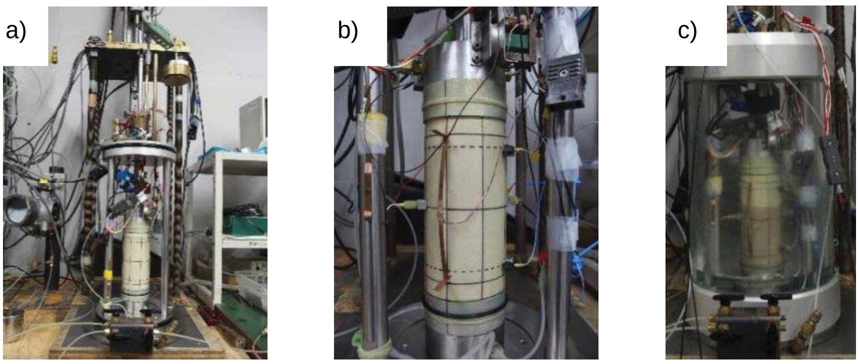


Fig. 6a) Soil specimen with negative pressure applied; b) Attaching of transducers to the soil specimen; c) Saturating of the soil specimen

11) The cell is set to the apparatus by means of three bolts. Three liters of water are poured into the cell. The counterbalance system is attached once again to the top cap in order to avoid tension and compression in the specimen;

12) A pressure of +30 kPa is reached in the cell on 5 kPa consequent steps and the initial negative pressure in the soil specimen of -30 kPa is reduced by 5 kPa on each step. After the last step the pressure in the specimen shall be 0 kPa. The absence of tension and compression in the sample is monitored during the whole operation (the "balance" is ensured by adding and removing of weight in the counterbalance system);

13) The specimen is fully saturated by means of "double vacuum" method (for Sofia sand), [4], or "CO_2" (for Toyoura sand) method depending on the type of material tested (Fig. 6c);

14) High capacity differential pressure transducer (HCDPT) and low capacity differential pressure transducer (LCDPT) are set by flushing water through them until no bubbles in the water are observed. Thereafter HCDPT and LCDPT are connected to the triaxial apparatus;

15) In consequent steps of 10 kPa (drained condition) the cell pressure and the back pressure , P_{BP} (pressure in the specimen), are increased in parallel until reaching 230 kPa and 200 kPa respectively (effective confining stress, σ'_c, of 30 kPa). The absence of tension and compression in the sample is monitored during the whole operation (the "balance" is ensured by adding and removing the weight in the counterbalance system);

16) The saturation of the soil specimen is evaluated by measuring Skempton's *B*-value (the value should be larger than 0.96) – [11] and [14];

17) The top cap is locked and the counterbalance system is removed. The apparatus is shifted below the controlling system (AC servo-motor) and the top cap is attached to it;

18) An external disk transducer for strain measurement is set to the apparatus. The transducer measures the displacement of a steel plate which is attached to the top cap;

The computer is set for automatic performance of the test;

For the triaxial apparatus employed in this study an AC servo-motor has been used in the loading system so that very small unloading-reloading cycles (cycling loading) under stress control could be applied accurately to the specimen in vertical direction. In order to measure the vertical stress, •$_1$, a load cell is located just above the top cap inside the triaxial cell in order to eliminate the effects of piston friction. The vertical strain, •$_1$, has been measured not only with external displacement transducer (EDT) but also with a pair of vertical local deformation transducers (LDTs) located on opposite sides of the specimen. The horizontal stress, •$_3$, has been applied through the air in the cell which has been measured with high capacity differential pressure transducer (HCDPT).

The total stress in the specimen during the tests and the corresponding strain are given as follows (Fig. 7):

$\sigma_3 = \sigma_c$ – radial (confining) stress (minimal principal stress), (1)

$\sigma_1 = \sigma_a = \sigma_c + (F_a / A_{specimen})$ – axial (vertical) stress – (maximum principal stress), (2)

$\varepsilon_3 = \varepsilon_c$ – radial (horizontal) strain; (3)

$\varepsilon_1 = \varepsilon_a$ – axial (vertical) strain; (4)

where:

F_a – axial (vertical) force,

$A_{specimen}$ – area of the cross section of the specimen

$\sigma_{dev} = q = \sigma_1 - \sigma_3$ – stress deviator, (5)

The corresponding effective stress which consider pore pressure are determined as follows:

$\sigma_3' = \sigma_c' = \sigma_c - u$ – effective radial stress, (6)

$\sigma_1' = \sigma_a' = \sigma_a - u$ – effective axial stress, (7)

where:

u – pore pressure,

$p' = \dfrac{\sigma_1' + \sigma_2' + \sigma_2'}{3} = \dfrac{\sigma_1' + 2\sigma_3'}{3} = \dfrac{\sigma_a' + 2\sigma_c'}{3}$ – mean effective stress, (8)

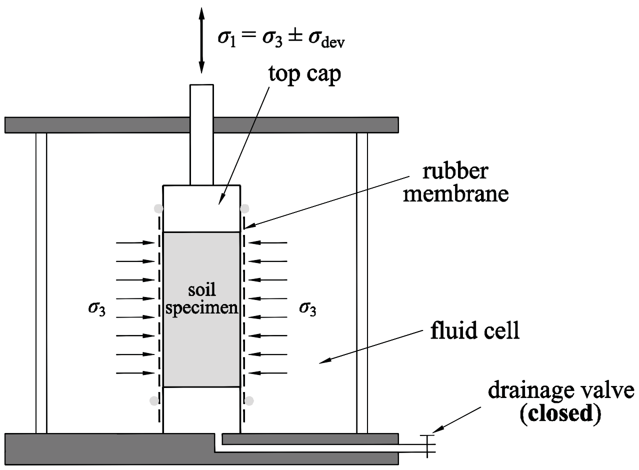


Fig. 7. Schematic overview of triaxial cyclic test of soil specimen

For the sake of reaching σ'_c = 100kPa of isotropic consolidation the stress has been increased in three consequential steps (50 kPa, 80 kPa and 100 kPa). The stress has been kept constant for 30 minutes in each step so that the deformations could cease. During this stage of the test the shear wave velocity, V_s, has been obtained for various values of σ'_c as well. When the final isotropic consolidation phase is reached at σ'_c = 100 kPa the stress has been kept constant until the vertical (axial) strains due to volume change cease. In the final stage cyclic loading in undrained conditions consisting of 10 cycles has been applied. The amplitude of the applied deviator stress, σ_{dev}, generates axial strain, ε_a, of about 10^{-6} which is in the elastic range of the soil behaviour. The whole procedure of the cyclic triaxial tests which have been performed are schematically shown in Fig. 8.

2.2 Dynamic measurements using trigger elements-accelerometers method

In order to generate shear waves a special type of source called „trigger elements“ has been employed (Fig. 7). The trigger elements are composed of multi-layered piezoelectric actuator made of ceramics (dimensions 10 mm x 10 mm x 20 mm, mass of 35 g and natural frequency of 69 kHz) and U-shaped thick steel bar to provide reaction force. Trigger elements have been used in pairs in order to apply large excitation equally. In the sake of receivng dynamic waves piezelectric accelrometers (cylindrical in shape with diameter of 3.6 mm, height of 3 mm, mass of 0.16 g and natural frequency of 60 kHz) as shown in Fig. 9 have been used (glued on the side surface of the specimen at two different heights).

2.3 Dynamic measurements using bender elements method

Bender elements are small piezo-electrical transducers which either bend as an applied voltage is changed or generate a voltage as they are bent. For the case of this study two bender elements have been glued on each side of the specimen so that shear waves could be transmitted and received in the cross section of the sample. There have been two ways for inducing shear waves in the cross section as it could be seen in Fig. 10. In the first the wave could be propagated perpendicularly through the cross section and the second parallel through the cross section.

A schematic figure of how all the equipment has been set on the specimen is shown on Fig. 10.

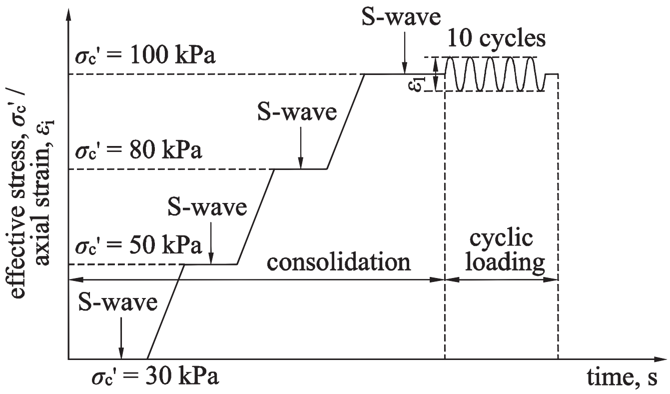


Fig. 8. Test loading sequence for elastic moduli determination of soil (σ'_c = 100 kPa)

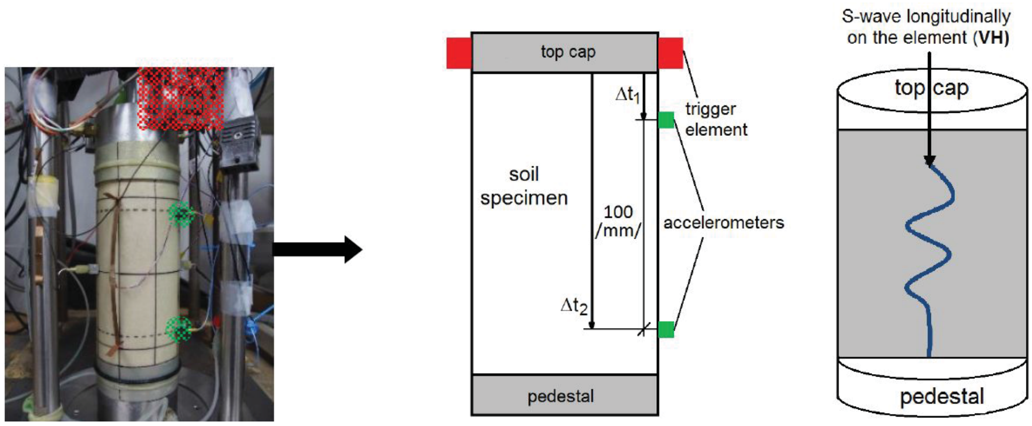


Fig. 9. Measurement of shear waves by means of trigger-elements/accelerometers method

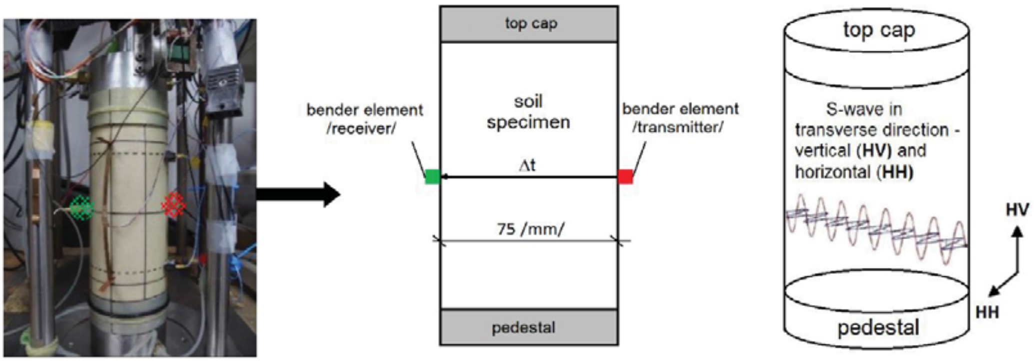


Fig. 10. Measurement of shear waves by means of bender elements method

2.4 Recording techniques of dynamic waves

A digital oscilloscope has been employed for recording of electrical outputs from accelerometers and bender elements with an interval of 10^{-6} sec (Fig. 11). To obtain clear signals a stacking (averaging) technique which has been originally installed in the oscilloscope and introduced instead of using filtering methods. The number of stacking which has been adopted is 256 with the bender elements and 128 with the accelerometers.

2.5 Testing procedures

A flow chart of the procedures for each measurement is shown in Fig. 12. Each specimen has been kept under saturated condition and subjected to isotropic consolidation. After the effective stress in the specimen, σ'_c, has reached 30 kPa, 50 kPa, 80 kPa and 100 kPa „dynamic“ measurements have been conducted. „Static“ measurements have been conducted only at the final stage of consolidation.

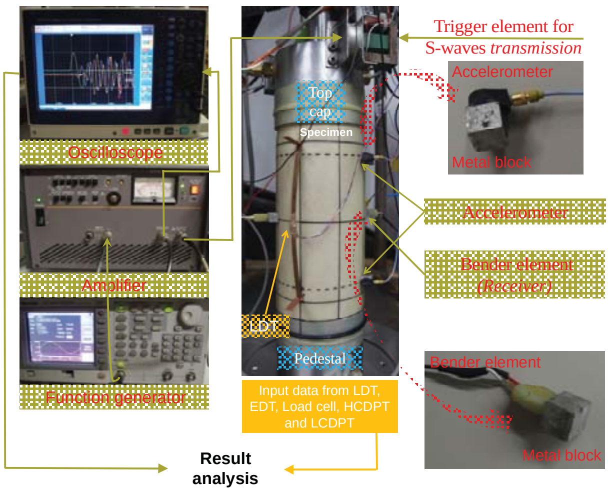


Fig. 11 Schematic overview of a soil specimen and location of the used equipment

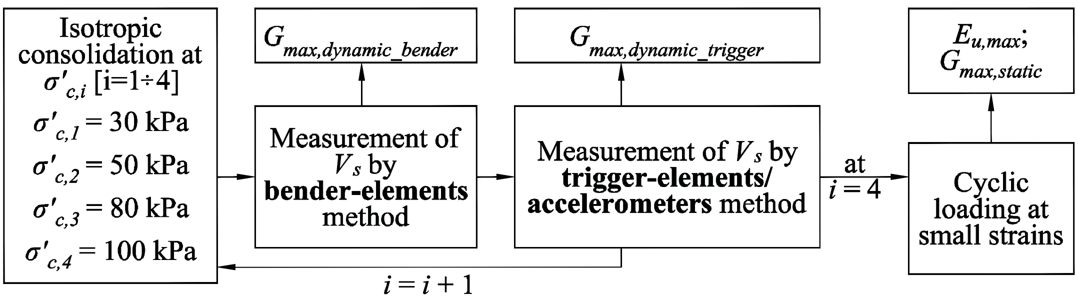


Fig. 12. Flow chart for determination of elastic moduli of soil by static and dynamic measurements

3 EVALUATION PROCEDURES OF STATIC AND DYNAMIC MODULI

3.1 Evaluating elastic modulus

Typical stress-strain relation during relatively small vertical unloading-reloading cycle is shown in Fig. 13. At each stress state the stress-strain relation has been fitted by a linear function and the small-strain Young's modulus has been evaluated on the basis of its inclination

The "static" Young's modulus obtained from undrained cyclic loading tests for cycle i, $E_{u,cyclic,i}$, is defined as follows:

$$E_{u,cyclic,i} = \frac{2\sigma_{dev,i}}{\varepsilon_{a,i}} = \frac{\sigma_{dev,i,\max} + \sigma_{dev,i,\min}}{\varepsilon_{a,i,\max} + \varepsilon_{a,i,\min}}, \qquad (9)$$

where:

$\sigma_{dev,i,\max}$ – maximum deviator stress for cycle i,
$\sigma_{dev,i,\min}$ – minimum deviator stress for cycle i,
$\varepsilon_{a,i,\max}$ – maximum axial strain for cycle i,
$\varepsilon_{a,i,\min}$ – minimum axial strain for cycle i.

In order to set the final value of the "static" Young's modulus the mean value of $E_{u,cyclic,5}$ and $E_{u,cyclic,10}$ is considered:

$$E_{u,cyclic} = \frac{E_{u,cyclic,5} + E_{u,cyclic,10}}{2}, \qquad (10)$$

As the Young's modulus is already evaluated and the Poisson's ratio of soil, ν, in undrained condition of 0.5 is adopted the shear modulus could be determined as follows:

$$G_u = G' = \frac{E_{u,cyclic}}{2(1+\nu)} = \frac{E_{u,cyclic}}{3}, \qquad (11)$$

Typical results of a triaxial cycling loading test (10 cycles) are presented on Fig. 14.

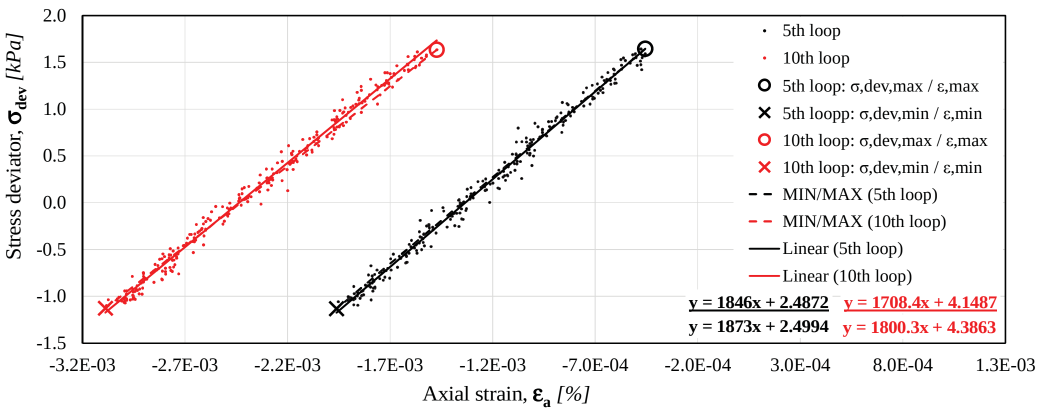
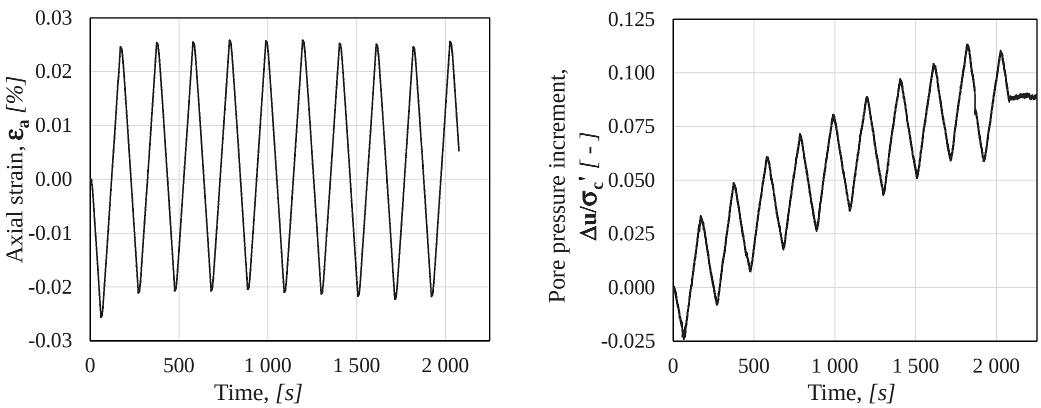


Fig. 13. 5th and 10th cycle of a cyclic triaxial test at small strain



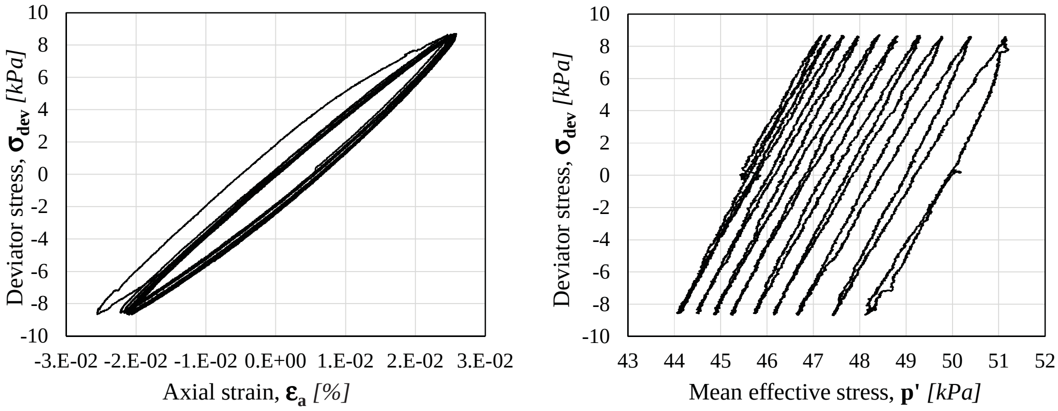


Fig. 14. Typical results from cyclic loading triaxial test

3.2 Travel time definitions

The propagation of shear waves through the soil specimen has been used to study the elastic properties of soils. All the methods involve measuring arrival time of propagated wave from the source to the receiver transducer, and as the distance between transducers is known, wave velocity can be determined.

In some cases shear waves are difficult to be identified due to near field effect, reflection and refraction of waves. These three factors make difficult to detect the accurate arrival point. There are a lot of methods to estimate the arrival time of waves, such as the cross-correlation method, time domain analysis, frequency domain approach, multiple reflections, wavelet analysis and variable path method.

Two different techniques have been adopted for this study – both related to the time domain analysis – [3], [5] and [15]. One technique detects arrival time by visual pick and the other uses mathematical procedure (cross correlation) to match the first rise points of the signals. Both methods will be explained below.

Time domain techniques are direct extraction of travel time based on the plots of the electrical signals versus time. The most commonly employed technique for detecting arrival time is a visual inspection of the received signal. Fig. 15 shows typical shear waveform in time domain series obtained on Toyoura sand.

In Fig. 15 main points have been selected for analysis:

- A: First deflection – where the output signal starts. This zone is part of the disturbance generated by the primary waves;
- B: Trough point – lowest peak before the starting of arrival of S-waves;
- C: First point on zero base line – the inflection point of the part of the wave where shear wave starts (also called "rise point");
- D: First major peak – first peak of the shear wave.

According to the reference points to consider in determining the arrival time the "first major peak to peak" approach has been adopted in the bender element method.

The time lapse between major peaks in input and output signals is considered as the travel time. Point 1 on Fig. 16 is the first major peak of the input signal and Point 2' or Point 2" (depending on the polarity of the bender elements) on the same figure is the first major peak of the received signal.

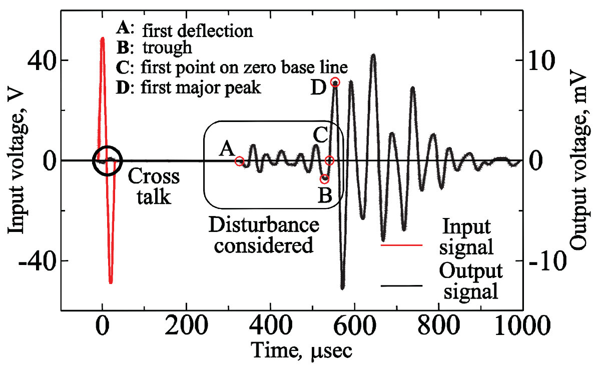


Fig. 15. Typical input and output signal of bender elements

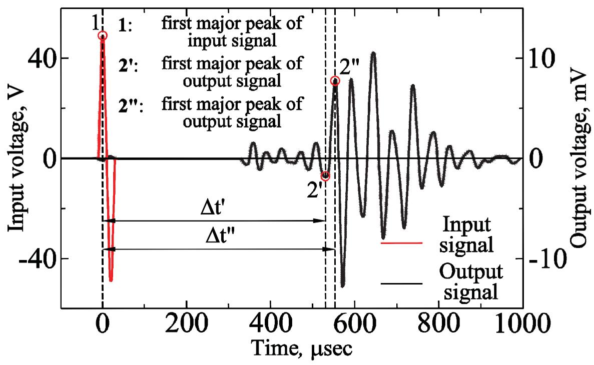


Fig. 16. Evaluation of time travel of shear waves by “first major peak to peak” approach – bender elements method

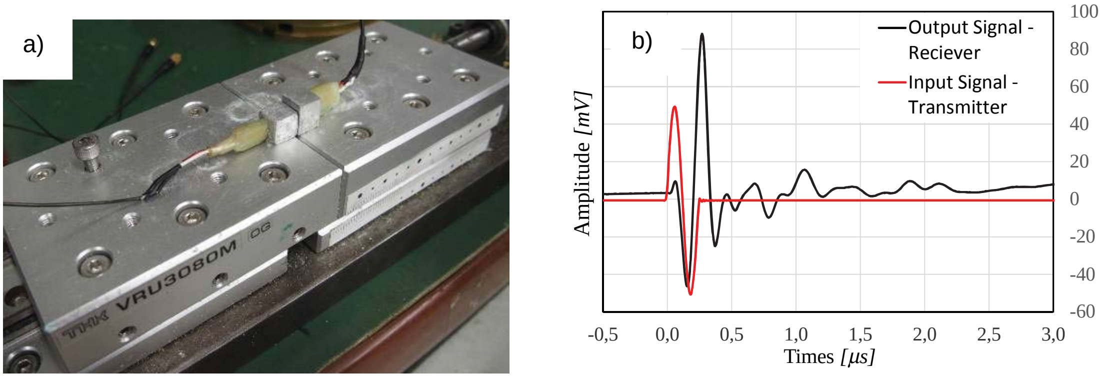


Fig. 17. Polarity check of bender elements: a) setup; b) recorded signals

When the bender elements method is adopted the question “which peak in the output signal should be chosen in order to evaluate the shear velocity – the first positive or negative major peak?” rises. For the sake of answering this question a polarity check of the bender elements is required. This is done through generating a signal and direct touch of the bender-transmitter to the bender-receiver (Fig. 17a). This means that the transmitted and received oscillations coincide almost completely in the time-domain (the difference occurs due to the distance between them – the thickness of the metal blocks which are attached to the bender elements) and in such way the two peaks could be distinguished in the analysis.

In the particular case Fig. 17b shows that the first major peak of the input signal corresponds to the first negative major peak (point 2’ on Fig. 16).

The “first major rise to rise” approach has been adopted for shear wave velocity evaluation in the trigger elements-accelerometers method – [2] and [12]. It is the most common approach used for detecting the arrival point in time domain. The time lapse between the first major deflections of the two output signals from the accelerometers is considered as the travel time (Fig. 18). In order to mathematically obtain the inflection point (rise) a cross-correlation has been adopted – [16].

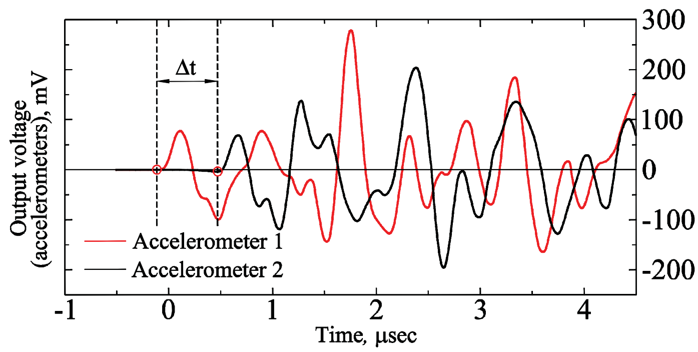


Fig. 18. Evaluation of time travel of shear waves by “first major rise to rise” approach – trigger elements/accelerometers method

3.3 Void ratio function – *f(e)*

Due to the difference in the relative density of the soil for each test the use of "void ratio function", *f(e)*, is obligatory in order to eliminate the various void quantity effect. There is a number of suggested equations in the literature for *f(e)* which allows the direct comparison of the results from tests performed at several values of the relative density of the soil. The experience of many researchers shows that the best results for tests with cohesionless soil specimens are obtained through the "void ratio funtion", *f(e)*, suggested in [8]:

$$f(\mathrm{e}) = \frac{(2.17 - e)^2}{(1 + e)}, \tag{12}$$

where:

e – void ratio.

The monitoring of the isotropic consolidation for each test allows measurement of the volume change in the specimen during the increase of the effective stress, σ'_c, until a stage where stabilization of the vertical axial strain, ε_a, accompanied by void ratio, *e*, stabilization is observed. During the stage of isotropic consolidation in the soil specimen the relation between the volume change, ε_{vol}, and the vertical axial strain, ε_a, should be theoretically 3 (ε_{vol} / ε_a ≈ 3) – Fig. 19.

The change of the void ratio, *e*, with the increase of the effective stress, σ'_c, during the isotropic consolidation is presented in Fig. 20 and Fig. 21 for all tests which have been performed.

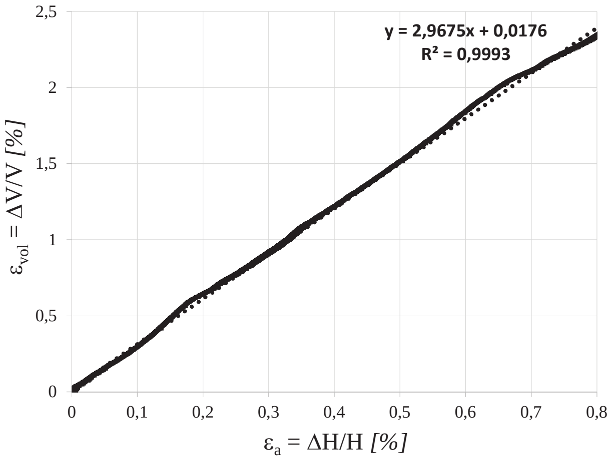


Fig. 19. Evolution of volume change versus axial strain during isotropic consolidation (Test 105)

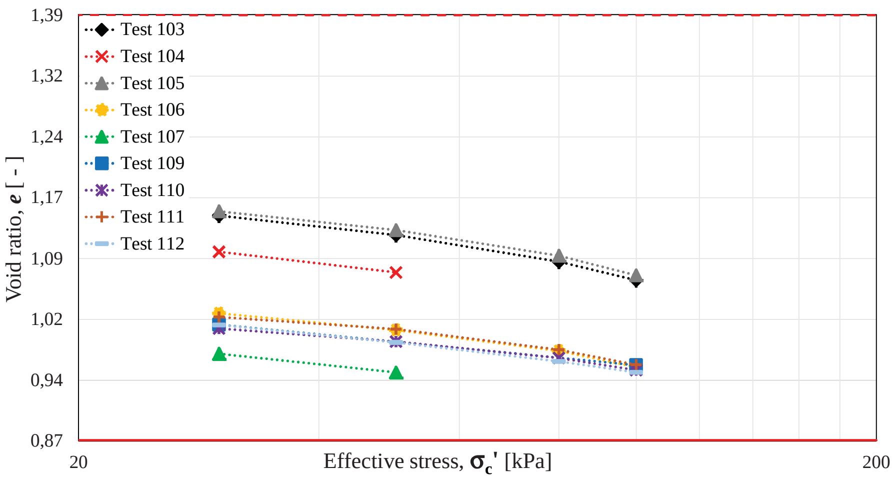


Fig. 20. Sofia sand: change of the void ratio with increase of the effective stress during isotropic consolidation

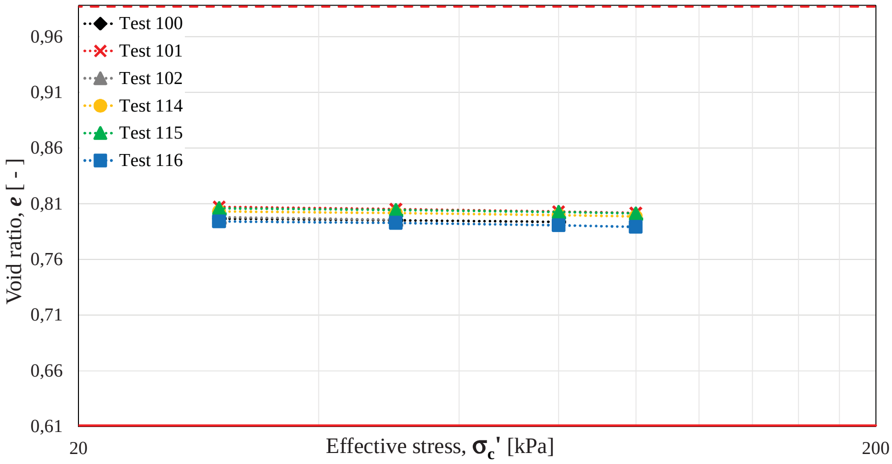


Fig. 21. Toyoura sand: change of the void ratio with increase of the effective stress during isotropic consolidation

4 TEST RESULTS

Fig. 22 ÷ Fig. 24 show the results from nine tests which have been performed with Sofia sand specimens. Both "static" and "dynamic" measurements are presented. Elastic moduli of soil have been normalized by a void ratio function in order to make a correction of void ratio's changes (changes of density) – [16].

Analogically the results from six tests which have been performed with Toyoura sand specimens are shown on Fig. 25 ÷ Fig. 27.

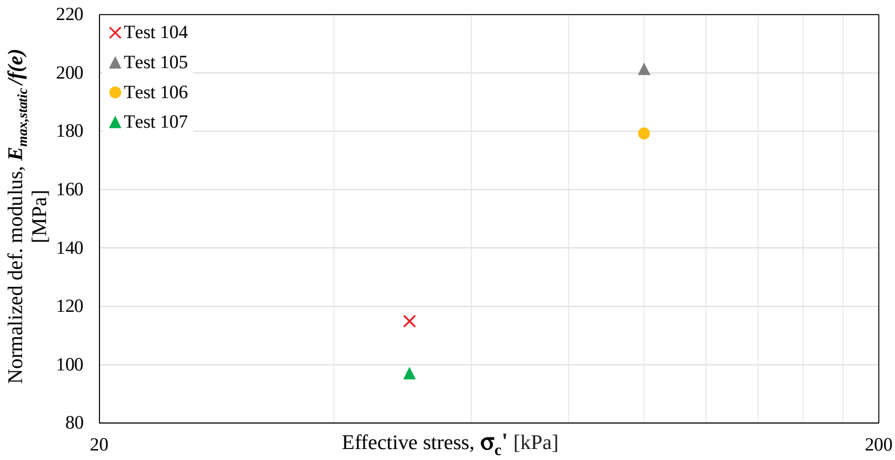


Fig. 22. Sofia sand: normalized Young's modulus determined by "static" method

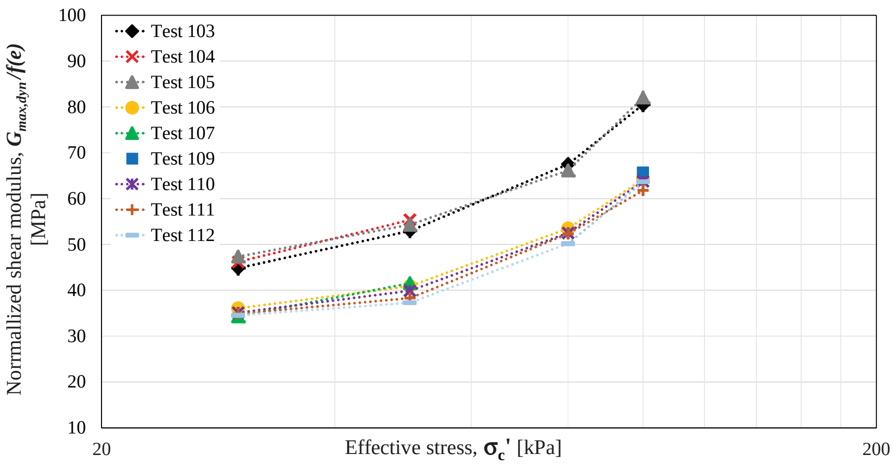


Fig. 23. Sofia sand: normalized maximum shear modulus determined by "dynamic" trigger-elements/accelerometers method

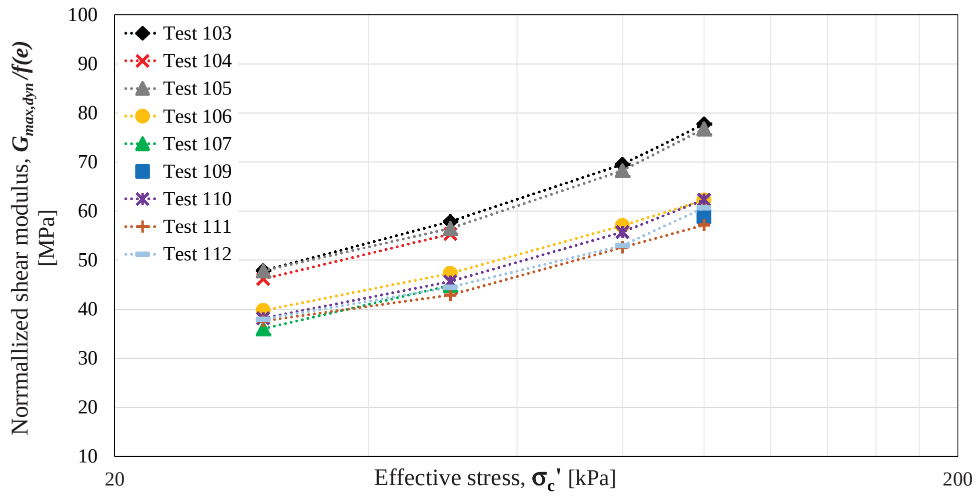


Fig. 24. Sofia sand: normalized maximum shear modulus determined by "dynamic" bender elements method

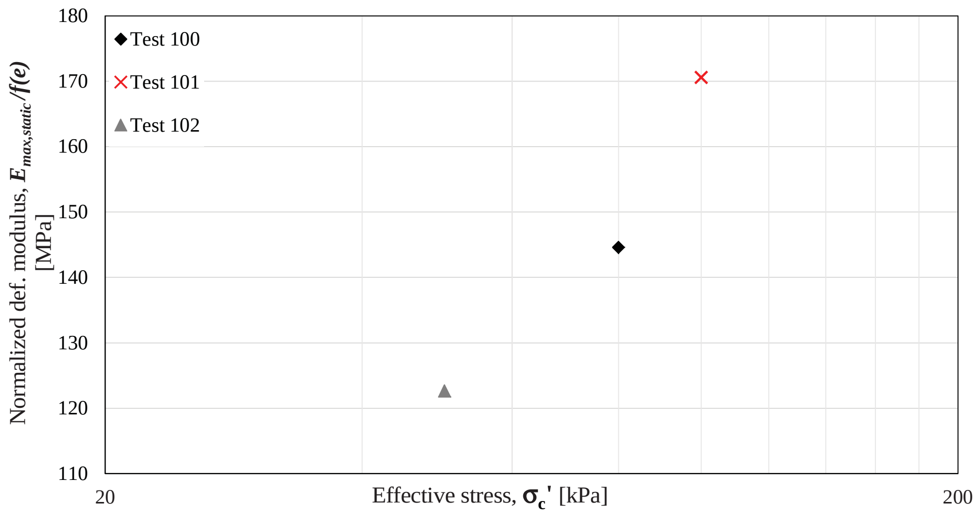


Fig. 25. Toyoura sand: normalized Young's modulus determined by "static" method

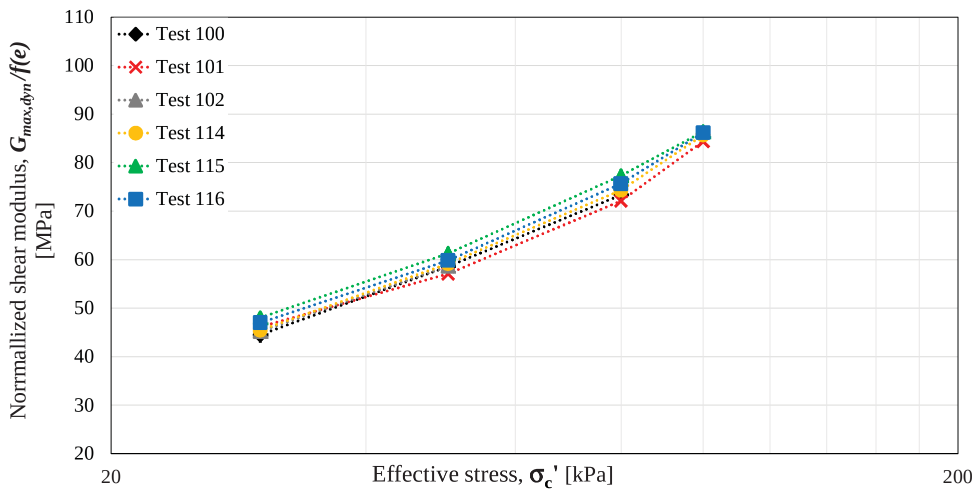


Fig. 26. Toyoura sand: normalized maximum shear modulus determined by "dynamic" trigger-elements/accelerometers method

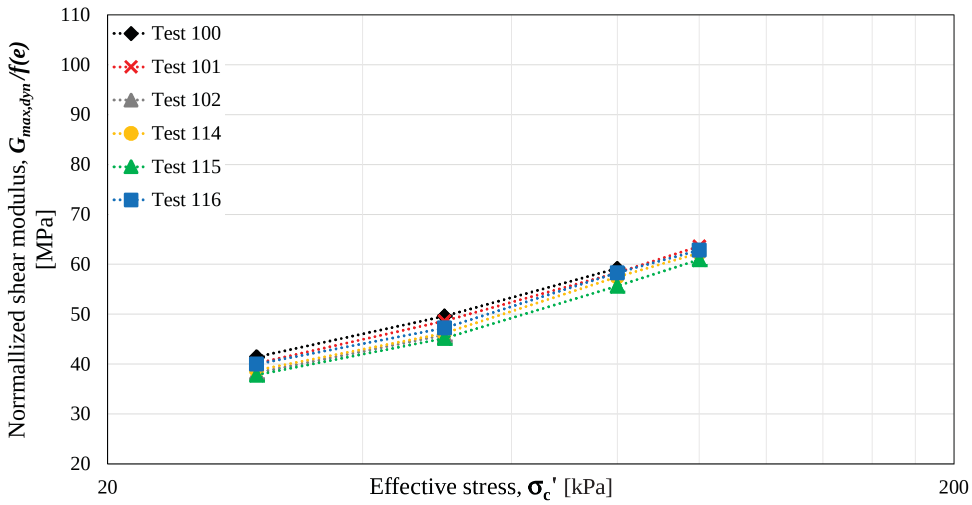


Fig. 27. Toyoura sand: normalized maximum shear modulus determined by "dynamic" bender elements method

5 CONCLUSION

The following conclusions could be drawn from the results presented in this study.

1. Dynamic measurement results in terms of elastic moduli based on shear wave velocity using two independent methods have shown good agreement to each other;

2. Dynamic Young's moduli based on shear wave velocity are larger than those by static measurement;

ACKNOWLEDGMENT

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6 REFERENCES

[1] Angelova, D., M. Yaneva, 1998. New data for the lithograph of neogene in the Sofia plateau. *Magazine of Bulgarian Geotechnical Society,* 59(2), pp. 37-40

[2] AnhDan, L., J. Koseki, T. Sato, 2002. Comparison of Young's Moduli of Dence Sand and Gravel Measured by Dynamic and Static Methods. *Geotechnical Testing Journal,* 25(4), pp. 349-368.

[3] Arulnathan, R., R. Boulanger, F. Riemer, 1998. Analysis of Bender Element Tests. *Geotechnical Testing Journal,* 21(2), pp. 120-131.

[4] Chiaro, G., 2010. *Deformation properties of sand with initial static shear in undrained cyclic torsional shear tests and their modeling*. Tokyo: Department of Civil Engineering, University of Tokyo.

[5] Clayton, C. R. I., M. Theron, I. Best, 2004. The Measurement of vertical shear wave velocity using side-mounted bender elements in the triaxial apparatus. *Geotechnique,* 54(7), pp. 495-498.

[6] Goto, S., F. Tatsuoka, S. Shibuya, Y. Kim, T. Sato, 1991. A simple gauge for local small strain measurements in the laboratory. *Soils and Foundations*, 31(1), pp. 169-180.

[7] Hayano, K., M. Matsumoto, F. Tatsuoka, J. Koseki, 2001. Evaluation of time-dependent deformation properties of sedimentary soft rock and their constitutive modeling. *Soils and Foundations*, 31(1), pp. 169-180.

[8] Hardin, B. O., F. E. Richart, 1963. Elastic wave velocities in granular soils. *Soil Mechanics and Foundations,* 89(1), pp. 33-65.

[9] Hong Nam, N., 2004. *Locally measured deformation properties of Toyoura sand in cyclic triaxial and torsional loadings and their modeling, PhD Thesis.* Tokyo: Department of Civil Engineering, The University of Tokyo.

[10] Milev, N., 2016. *Soil structure interaction – PhD Thesis (in Bulgarian)*. Sofia: UACEG.

[11] Skempton, A. W., 1954. The pore pressure coefficients A and B. *Geotechnique*, 4(4), pp. 143-147.

[12] Tanaka Y., K. Kudo, K. Nishi, T. Okamoto, T. Kataoka, T. Ueshima, 2000. Small strain characteristics of soils in Hualien, Taiwan. *Soils and Foundations,* 40(3), p. 111–125.

[13] Tani, Y., Y. Hatamura, T. Nagao, 1983. Development of Small Three Components Dynamometer for Cutting Force Measurement. *Bulletin of the Japanese Society of Mechanical Engineering*, 26(214), pp. 650-658.

[14] Towhata, I., 2008. *Geotechnical Earthquake Engineering.* Berlin: Springer.

[15] Viggiani, G., J. Atkinson, 1995. Interpretation of Bender Element Tests. *Geotechnique,* 45(1), pp. 149-154.

[16] Villalta, G., 2015. *Change of Shear wave velocities induced by repeated liquefaction.* Tokyo: Department of Civil Engineering, University of Tokyo.

SUMMARY

STATIC AND DYNAMIC EVALUATION OF ELASTIC PROPERTIES OF SOFIA SAND AND TOYOURA SAND BY SOPHISTICATED TRIAXIAL TESTS

Nikolay MILEV
Junichi KOSEKI

The main purpose of the presented paper is to show the advantages and disadvantages of evaluating the small strain stiffness of cohesionless soils by means of different types of laboratory equipment. A series of consolidated undrained cyclic triaxial testes have been performed on saturated specimens made of Toyoura sand and Sofia sand having various dry densities. Relatively small unloading-reloading cycles have been applied on the specimens in order to obtain the “static” Young’s modulus. Furthermore two types of wave propagation techniques have been adopted for the sake of a “dynamic” Young’s modulus determination: one is using bender elements in the cross section of the specimen and the other is using trigger elements in the longitudinal section of the specimen to excite shear waves and two accelerometers which capture the waves’ arrival in two points. On one hand the difference between the two types of dynamic measurements and static measurements is discussed and on the other hand some relationships between the abovementioned approaches are given.

Key words: triaxial test, small strain cyclic loading, shear wave velocity, accelerometer, bender element, shear modulus

REZIME

STATIČKO I DINAMIČKO VREDNOVANJE ELASTIČNIH SVOJSTAVA PESKA IZ SOFIJE I TOJOURA SOFISTICIRANIM TRIAKSIJALNIM OPITOM

Nikolay MILEV
Junichi KOSEKI

Osnovni cilj ovog rada jeste da pokaže prednosti i nedostatke vrednujući krutosti pri malim deformacijama nekoherentnog tla pomoću različitih tipova laboratorijske opreme. Serija konsolidacionih nedreniranih cikličkih triaksijalnih opita na uzorcima je peska iz Sofije i Tojoura, koji su imali različite gustine u suvom stanju. Relativno mali ciklusi rasterećenja i ponovnog opterećenja urađeni/izvedeni su na uzorcima s ciljem određivanja „statičkog Jungovog modula“. Osim toga, korišćena su dva postupka prostiranja talasa radi određivanja „dinamičkog Jungovog modula“: jedan je korišćenje „bender“ link elemenata u poprečnom preseku uzorka, a drugi je korišćenje „trigger“ elemenata u podužnom preseku uzorka da bi se izazvali smičući talasi i dva akcelerograma koji hvataju/registruju dolazeće talase u dve tačke. U radu su analizirane razlike između ova dva tipa dinamičkog merenja i statičkog merenja, kao i zavisnosti između prethodno pomenutih pristupa.

Ključne reči: triaksijalni test, male dilatacije pod cikličnim opterećenjem, brzina smičućih talasa, akcelerometar, „bender“ element, smičući modul

KOMPARATIVNA NELINEARNA ANALIZA INTERAKCIJE ŠIP-TLO AB 2D RAMA*

COMPARATIVE NONLINEAR ANALYSIS OF A RC 2D FRAME SOIL-PILE INTERACTION*

Boris FOLIĆ
Radomir FOLIĆ

ORIGINALNI NAUČNI RAD
ORIGINAL SCIENTIFIC PAPER
UDK: 624.154.072.332
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1 UVOD

Tokom seizmičkih analiza obično se pretpostavlja uklještenje u osnovi, a zanemaruje se fleksibilnost tla i temelja. Ipak, za tačnije seizmičke analize potrebno je uvesti u proračun pored konstrukcije zgrade temelje i tlo, što uslovljava unošenje celokupnog sistema kao ulaznih podataka. Pri tome se posebne teškoće javljaju pri unošenju podataka o karakteristikama tla. U nekim radovima koriste se specijalne histerezisne zavisnosti i nelinearni odgovor sistema sa jednim stepenom slobode (SDOF) kao reprezent konstrukcije zgrade, pri čemu je lakša analiza uz uvođenje fleksibilnosti temelj-tlo i njihovog uticaja na odgovor konstrukcije. Uglavnom, smatra se da uvođenje interakcije redukuje odgovor konstrukcije, a time i oštećenja. Međutim, u pojedinim slučajevima može doći i do negativnih efekata, što je razmatrano u radu [7]. Neka istraživanja [7] su pokazala da se u seizmičkoj analizi mogu uvesti pojednostavljeni modeli tla i znatno olakšati proračun sistema, naročito regularnih zgrada [4] i [9]. Upoređenjem rezultata driftova, na 2D i 3D ramu, dobijenih pušover analizom sa rezultatima dobijenih primenom analize vremenske istorije u [8] je pokazano da se uvođenjem faktora modifikacije mogu dobiti konzervativni rezultati primenljivi u projektantskoj praksi. I u radu [22] je prikazan približni proračun krutih ramova uz uvođenje interakcije tlo-temelj-konstrukcija, primenljiv u praksi projektovanja. Analitička rešenja su znatno ređa, iako je bilo pokušaja [20].

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1 INTRODUCTION

During seismic analysis of a structure, it is assumed that the building is clamped at the base and the soil and foundation flexibility is ignored. Yet, for more accurate seismic analyses, in addition to the building structure it is necessary to introduce foundations and soil, which requires entering of the entire system as input data. In the process, special difficulties arise when entering data of the soil characteristics. In some papers, special hysteresis dependencies and non linear response of the system with one degree of freedom (SDOF system) are used for representing of the building structure, whereby an analysis which introduces the foundation-soil flexibility and their impact on the structural response is easier to perform. It is generally considered that introduction of interaction reduces the structural response, and thus damage. However, in some cases, negative effects may occur, which was discussed in the paper [7]. Some research, e.g. [7], showed that in this seismic analysis, simplified soil models could be introduced thus making system design considerably easier, especially design of regular buildings, see [4] and [9]. By comparing results of drifts of 2D and 3D frames, obtained by a pushover analysis, with results obtained using the time history analysis, see [8], it is showed that by introducing the modification factors, one can obtain conservative results applicable in designing practice. In the paper [22], an approximate design of rigid frames, applicable in designing practice, with interaction of soil-

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* This paper is dedicated, with respect, to academician Dusan Milovic

Evropska regulativa za seizmičko projektovanje EN 1998, Part 1 i Part 5, ne razmatra detaljno problem uvođenja interakcije tlo-temelj-konstrukcija (SFI) u numeričkim seizmičkim analizama. U EN 1998-5 to se zahteva gde P-Δ efekti imaju veliku ulogu; konstrukcije sa masivnim i dubokim temeljima, i konstrukcije na veoma mekom tlu u kojima je prosečna brzina smičućih talasa manja od 100 m/s [5] i [10]. Razlike seizmičkog ponašanja objekata plitko fundiranih i na šipovima detaljno je opisana u [4] i [5]. U njima je detaljno opisan način analize kinematičke i inercijalne interakcije pri fundiranju na šipovima. Kinematička interakcija potiče od razlike pokreta tla i temelja ili šipova, tokom zemljotresa, pri čemu se masa zanemaruje. Kod inercijalne interakcije, pručava se uticaj inercijalnih sila od konstrukcije na temelje.

U ovom radu je sprovedena komparativna nelinearna statička (NSA), često nazvana pushover analiza, i dinamička analiza NDA, detaljno opisane u [3], na 2D rama AB skeletne zgrade fundirane na šipovima. U modelu je uključena i linearno-nelinearna interakcije šip-tlo korišćenjem link elemenata. Tlo je modelovano sa više(linijskim) plastičnim veznim elementima, kao anvelopama u obliku *p-y* krivih, sa obe strane šipa. P-y krive prenose (primaju) samo pritisak. Krive *p-y* su modelovane prema Koksu, Risu i Matloku [4] i [12] i [19] za potopljen pesak, i šipove prečnika 60 cm. Analizirane su seizmičke performanse sistema konstrukcija-temelj-tlo jednog 2D rama fundiranog na šipovima. Prikazano je delimično linearno i nešto detaljnije nelinearno ponašanje krovne grede, dok se ostale grede rama ponašaju nelinearno. Linearna krovna greda je ona kod koje nisu uvedeni plastični zglobovi u čvorovima na preseku sa unutrašnjim stubovima, ali ni u polju krovne grede.

2 OPIS KONSTRUKCIJE, TEMELJA I METODA ANALIZE

2.1 Analizirana konstrukcija

Analiziran je fasadni ram koji ima četiri stuba, kao i unutrašnji ram. Na fasadnom ramu su ugaoni stub i ivični stubovi. Ugaoni stubovi fasadnog rama su fundirani na grupi od 3 šipa, a unutrašnji na grupi od po četiri šipa. Fasadni ram je "kondenzovan", tako što su svi elementi šipova ubačeni putem projekcije upravno na srednju ravan rama. Na taj način se model rama može prikazati u samo jednoj ravni. Grupa od 3 kružna šipa ima deo koji se sastoji od jednog šipa (1D60), i deo koji se sastoji od dva kondenzovana šipa (2D60) slika 1. Dakle u „kondenzovanom" modelu od tri šipa unose se samo dva šipa, 1 je samostalan šip, a drugi je dvostruki šip (tj. u ravanskom modelu je unet jedan šip kod koga su poprečni preseci FRAME elementa, u programu SAP2000, u delu Set Modifiers, krutost i masa pomnoženi sa 2). Shodno tome p-y krive „dvostrukog" šipa imaju dvostruko veću krutost. Objekat ima dve relativno vitke donje etaže. Visina prve dve etaže je po 5 metara, ali su zato poprečni preseci stubova na ove dve

foundation-structure, was presented. Analytical solutions are rare, even though there were attempts, [20].

European regulation for seismic design, EN 1998, Part 1 and Part 5, does not consider in detail the problem of introducing the soil-foundation-structure interaction (SFI) in the numerical seismic analyses. In EN 1998-5 it is required where P-Δ effects have an important role: the structures with massive and deep foundations, and structures in a very soft soil where the average velocity of shear waves is less than 100 m/s, see [5] and [10]. The difference of seismic behaviour of the structures founded on shallow foundations and on the piles was described in detail in [4] and [5]. In them, the method of analysis of kinematic and inertial interaction of pile foundations was described in detail.

A comparative non-linear static analysis (NSA), often called a pushover analysis, as well as the dynamic non-linear analysis (NDA), described in detail in [3], and applied on 2D frames of RC skeletal buildings founded on piles are presented in this paper. The model involves a linear-non-linear pile-soil interaction, using link elements. The soil is modelled using multiple (linear) plastic link elements, as envelopes in the form of the p-y curves, on both sides of the pile. P-y curves are transferring only compression and are modelled according to Cox, Reese and Matlock [4], [12] and [19] for submerged sand, and piles with a diameter 60 cm. The seismic performances of the structure-foundation-soil system of a 2D frame founded on piles are analyzed. Detailed analysis of hierarchy of formation of plastic hinges in the frame and piles is presented. A partial linear and a more detailed non-linear behaviour of a roof beam are presented, while the remaining beams of the frame behave non-linearly. Linear roof beam is the beam without plastic hinges at intersections with inner columns or along the beam. Kinematic interaction arises from different motions of the soil and foundation, or piles, during earthquake, while the mass is neglected. In inertial interaction, the effect of inertial forces from the structure upon the foundation is considered.

2 DESCRIPTION OF THE STRUCTURE, FOUNDATIONS AND METHODS OF ANALYSIS

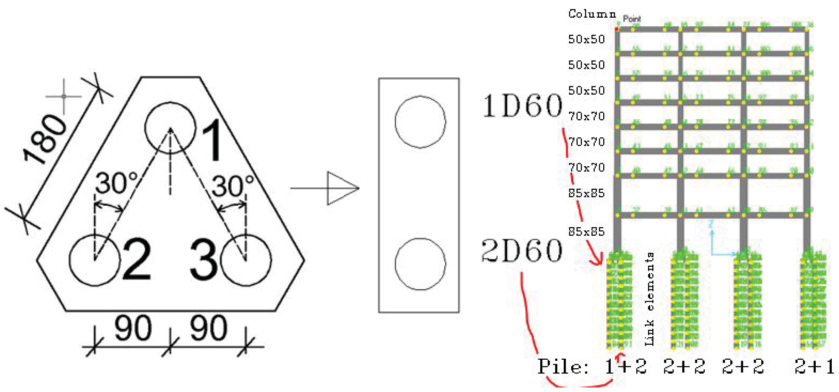
2.1 Analyzed structure

A façade frame with four columns and an inner frame are analyzed. On the façade frame, there are corner columns and peripheral columns. The corner columns of the facade frame are founded using a group of 3 piles, whereas the inner columns are founded on a group of four piles. The façade frame is "condensed" by inserting all pile elements via projection along the direction perpendicular to the frame middle plane. In this way, it is possible to draw the frame model using only two dimensions. The group of 3 circular piles consists of a part made of one pile (1D60), and another part made of two condensed piles (2D60), figure 1. Hence, in this "condensed" model, only two out of three piles are introduced, one of which is an individual pile, whereas the other is a double pile (i.e. a single pile was introduced to the model, whose Frame element cross-section, stiffness and mass were multiplied by 2), in SAP 2000 software, within the Set Modifiers module. In accordance with this, the p-y curves of the "double" pile etaže 85/85cm.

U seizmičkoj analizi su primenjene nelinearna statička (NSA), često nazvana pushover, i nelinearna dinamička analiza (NDA), u vremenskom domenu (Time History). S obzirom da se tokom NSA i NDA [3] praktično ne pojavljuju plastični zglobovi u šipovima kod krućih vrsta tla (osim u specijalnim slučajevima), analizirani su i uticaji u tlu, preko link elemenata po dubini, pod seizmičkim dejstvom (TH El Centro 0,30g). Oni na takav način nisu dovoljno obrađeni u dostupnoj literaturi. Analizirana su ekstremna pomeranja, sile, ukupan rad, "trenutni rad", i raspodela istih po dubini link elemenata šipa. Analiziran je samo jedan šip iz grupe i to samostalni šip (1D60) iz grupe od tri šipa (1+2, na slici 1). To je krajnji šip na obe strane simetričnog rama (vidi sliku 1 desno). Konkretno je, u ovom radu, istraživan samo levi krajnji šip. Takođe je analizirana promena ukupne smičuće sile u osnovi sa porastom vršnog ubrzanja (PGA), i promena stanja plastičnih zglobova u konstrukciji i šipovima i u skladu s tom promenom konstruktivnog sistema, promena prvog i drugog svojstvenog tona nakon završetka seizmičkog dejstva (Time History).

also have the double value of stiffness. The building has two relatively slender lower stories. The height of the first two storeys is 5 meters each, but the cross-sections of columns in these two floors are 85/85 cm.

Seismic analysis is performed using the non-linear static analysis (NSA), often called the pushover analysis, and the non-linear dynamic analysis (NDA) in the time domain (time history). Regarding that during NSA and NDA [3] there is practically no occurrence of plastic hinges in the piles for stiffer types of soil (except in special cases) the effects in the soil are analyzed too, via link elements along the depth, under a seismic action (TH El Centro 0.30g). They are not sufficiently discussed in the available literature in this way. Extreme displacements, forces, total work, "instantaneous work" in link elements and their distribution along depth of the pile are analyzed. Only one pile from the group is analyzed, namely, a single pile (1D60) from the group of three piles (1+2, in figure1). These are the end piles on both sides of the symmetrical frame (see figure 1 right). In this paper specifically, only the left endmost pile is studied. Also, the variation of the total seismic base-shear force, with the increase of peak acceleration (PGA) is analyzed, and also the variation of condition of plastic hinges in the structure and the piles, and variation of the first and second natural tones after seismic excitation (Time History)



Slika 1. Princip „kondenzacije" grupe od 3 šipa u grupu od 2 šipa u ravni. 1D60 samostalni šip, a 2D60 dvostruki
Figure 1. „Condensation" principle of a group of 3 piles (1D60 – individual pile, 2D60 – double pile).

Prostorni (3D) ram je dimenzionisan na zemljotresno dejstvo u programu SAP 2000 v14. sa uvođenjem upravnog pravca i torzije (sa 5% ekscentriciteta), za faktor ponašanja 5.85. Nakon toga je iz tako dimenzionisanog modela izdvojen, napred opisani, fasadni 2D ram sa pripadajućim opterećenjem. Raspon ramova je 8m, a to je i osno rastojanje stubova, u oba pravca. Objekat je dvoosno simetričan. Visina prve dve etaže je po 5m, a ostalih 6 etaža je 3.1 m. Model je sličan modelima datom u [2] i [4]. Razlika je u p-y krivama koje su u navedenom radu [4] date za šipove prečnika 1,2m. Takođe, u navedenom radu je dato više različitih modela, sa i bez interakcije šip-tlo. Izgled ovde usvojenog, samo jednog modela rama, je dat u nastavku rada, kod analize stanja plastičnih zglobova. Kod [2], za izveden objekat, stubovi su svi preseka 60/60cm, a opterećenje je nešto manje.

Izdvajanje 2D rama iz trodimenzionalnog (3D) rama prati specifična problematika [4] i [8]. Prvi parametar je

The spatial (3D) frame is dimensioned with reference to earthquake action, using SAP 2000 v14 software, including the effects in the perpendicular direction and torsion (with 5% eccentricity), for a behaviour factor of 5.85. The previously described façade 2D frame with its corresponding loads is then taken out of a 3D model dimensioned in this way. The span between frames is 8m, which is also the distance between the pile axes, in both directions, since the structure in question is symmetrical along two orthogonal axes. The height of the first two stories is 5m, while for the remaining 6 storeys, it is 3.1 m. The model is similar to models shown in [2] and [4]. However, the difference is in p-y curves, which are, in [4], given for piles of diameter 1.2m. Also, more different models with and without pile-soil interaction are given in [4]. The geometry of the single frame considered here is presented later in the section where the state of plastic hinges is analysed. Paper [2] is related to built structure, where all columns pripadajuće opterećenje, drugi je geometrija poprečnih preseka (pre svega) greda, a za dinamičku analizu, neophodno je proveriti i razliku svojstvenih perioda izdvojenog 2D rama u odnosu na 3D model. Poželjno je, proveriti i deformaciju („ugib“) vrha 3D objekta u odnosu na vrh 2D rama. Realnije ponašanje ravanskih modela (kod kojih je izdvojen samo jedan 2D ram), u odnosu na prostorni ram, daju modeli kod kojih je link elementima ili prostim štapovima spregnuto više ramova u 2D modelu [11]. Kao npr. spregnuti fasadni i unutrašnji 2D ram ili 2 unutrašnja i 2 fasadna, uz uslov da se spregnuti ramovi u 3D modelu moraju pružati u istom pravcu.

Kod izdvajanja 2D rama, neophodno je proveriti prihvatljivo poklapanje vrednosti normalnih sila u stubovima, i reakcija oslonaca za oba modela. Takođe, kod čvorova gde se sučeljavanju grede iz drugog pravca, može se primeniti komanda *Set Modifiers*, za uvećanje krutosti poprečnih preseka napregnutih na savijanje, zbog učešća torzione krutosti drugog pravca. Kod (malih) poprečnih preseka greda i tanjih ploča, kod kojih torziona krutost, brzo opada/degradira se tokom zemljotresa, bolje je uvesti nelinearnu rotacionu oprugu, kojom se simulira ova krutost, ali i pad iste tokom zemljotresa. Često se u seizmičkoj (dinamičkoj i kvazi statičkoj) analizi, zbog smanjenja broja nepoznatih isključuju ploče, i koriste bruto preseci greda, čak i bez pripadajućih aktivnih širina ploča, i bez komande *Modifiers*. Kako je to na strani sigurnosti, primenjeno je i u ovom radu, za fasadne grede. Uticaj spratnih ploča, na ujednačavanje horizontalnih pomeranja spratova, kod 2D i kod 3D rama, može se poboljšati komandom *Joint - Constraints (Equal)*.

Gornji čvorovi (u krovnoj ravni) unutrašnjih stubova ne zadovoljavaju uslove odnosa krutosti greda i stubova, te je u prvom delu rada, taj deo grede linearizovan, odnosno na tom delu nisu uneti plastični zglobovi. To će se sagledati, kasnije, u delu analize rezultata. (premestiti zarez)

Ukoliko se ne upotrebi preporučena opcija SAPa, automatske podele plastičnih zglobova u linijskim elementima od 0,02, lom greda se događa i na drugom spratu dosta rano, a kasnije i na drugim lokacijama, pa se produžava vreme proračuna. Ovaj primer nije prikazan, iako je i ovakav način loma moguć, ali manje verovatan, ako je konstrukcija dobro izvedena. Zato je u svim ovde prikazanim modelima, ova opcija primenjena.

Odnos ukupne seizmičke sile u osnovi i ukupne sile težine objekta, kod (regularnog, kvadratnog u osnovi) 3D modela, je: 5316 kN/71093 kN= 7,48%. Svojstveni periodi 3D modela su: T_1=T_2=2,04 sec, T_3=1,47 sec. Prva dva tona su horizontalna-lateralna, a treći je torzioni-obrtni [4].

are 60/60cm, and the loading is slightly lower.

Extraction of a 2D frame from a three-dimensional (3D) frame is accompanied by specific problems, [4] and [8]. The first parameter is the corresponding load; the second is the geometry of cross sections (primarily) of the beams, while for a dynamic analysis, it is necessary to check the difference between the natural periods of the extracted 2D frame and the 3D model. It is desirable to check and compare the deformation (horizontal deflection) of the top of a 3D object in comparison to the top of the 2D frame. A more realistic behaviour of the planar models (where only one 2D frame is extracted from 3D model), when compared with the spatial frame, is obtained by 2D models where more 2D frames are combined in one model, using link or truss elements, [11]. For instance, coupled façade and the interior 2D frame, or 2 interior and 2 façade frames, are examples of that. Of course, it is assumed that frames are aligned along the same direction.

When extracting a 2D frame, it is necessary to check the acceptable agreement of values of normal forces in the columns, and the support reactions of both models. Also, in a case of nodes where the beams from another direction are connected, one might use the command *Set Modifiers*, in order to increase the bending stiffness of sections, due to influence of the torsional stiffness in another direction. In the case of (relatively small) cross-sections of beams and thin slabs, where the torsional stiffness quickly decreases/degrades during earthquakes, it is better to introduce a non-linear rotational spring which simulates this stiffness and its decline during earthquakes. Very often, in the seismic (dynamic and quasi static) analysis, due to reduction of a number of unknowns, the slabs are excluded, and gross cross-sections of beams are used, even without the belonging active widths of the slabs, and without the *Modifiers* command. Since it is on the safety side, it is implemented in this paper, too, for the façade beams. The effects of the floor slabs to unify the horizontal storey displacements, in 2D and 3D frames, may be improved by using the command *Joint – Constraints (Equal)*.

Upper nodes (in the roof) of the inner columns do not meet the conditions of stiffness ratio of beams and columns, so in the first part of the paper this upper beam is linearized, i.e. no plastic hinges are introduced in this beam. It will be considered later, in the section of the result analysis.

Unless the recommended option of SAP, *Assign/Frame/Frame Signed Overwrights/Auto Subdivide Line Objects at Hinge* of 0,02 is used, the failure of the beams occurs on the second storey quite early, and later in other locations as well, so the time of calculation is prolonged. This example is not presented, even though such failure mode is quite possible, but unlikely, if the structure is well constructed. For this reason in all the presented models, this option is implemented.

The ratio of the total base-shear force and the total force of the building weight (of a regular, square layout) 3D model, is: 5316kN/71093kN= 7.48%. Natural periods of the 3D model are: T1=T2=2.04 sec, T3=1.47 sec. The first two tones are horizontal-lateral, and the third one is torsional-rotational [4].

2.2 Ponašanje i modeliranje tla u zemljotresu

Odgovor na pitanje šta se događa u tlu tokom zemljotresa, zavisi, pre svega, od načina modelovanja tla u sistemu konstrukcija temelj tlo, zatim koji zemljotres i kakav uticaj se konkretno istražuje. Verovatno najbolji odgovor, na ovo pitanje, pruža talasna mehanika, i njena primena na numeričke analize korišćenjem Solid elemenata tla. To je posebna problematika, jer u procesu modelovanja zahteva iznalaženje optimuma tokom zadovoljenja često suprotstavljenih uslova. Parametri koje u procesu definisanja modela treba odrediti su: dimenzije modela konstrukcije, zatim veličine modela sistema, veličine mreže KE, oblast frekvencije i vrste talasa koji se prostiru, granični problemi, itd. [17]. Kod ove metode, važno je u odnosu na veličinu konstrukcije proceniti minimalnu veličinu modela za sistem konstrukcija-temelj-tlo, kako bi vreme proračuna bilo što kraće, a da se pri tom adekvatno obuhvate svi potrebni talasni fenomeni. Takođe treba odrediti i maksimalnu veličinu konačnih elemenata tla, koja se ne sme prekoračiti kako ne bi došlo do neželjenih talasnih efekata u samim konačnim elementima tla i sl. Ovde ćemo se zadržati na korišćenju link elemenata tla (LES), kao Takedina anvelopa eksperimentalno određenih *p-y* krivih (ATPY), jer je to jednostavniji model za primenu, te će i odgovor biti zasnovan na istom.

2.3 *P-Y* krive za šipove u pesku

Tlo se u analizi dinamičke interakcije sistema konstrukcija-temelj-tlo može predstaviti upotrebom modela različitog stepena složenosti (sofisticiranosti). Uobičajene metode seizmičke analize nelinearnog ponašanja konstrukcija su kvazi-statička pušover NSA i nelinearna dinamička analiza u vremenskom domenu NDA, kao numerička integracija akcelerograma, tzv. metoda korak po korak (time history, step by step). Pri tome su za seizmička dejstva korišćeni akcelerogrami El Centro, za PGA 0,20; 0,25 i 0,30 *g*.

Tlo može biti modelovano preko različitih uslova oslanjanja, konstrukcije ili šipova, kao što je:

– linearnih opruga sa jednom čvornom tačkom (spring), koje trpe podjednako i zatezanja i pritisak,

– linearnih link elemenata

– više-linearnih plastičnih link elemenata, koje se mogu zadati tako, da prenose samo pritisak.

Tlo je modelovano preko elemenata veze, tzv. link elemenata, prema p-y modelu za pesak koji je razvio Ris i dr. Reese, Cox, Koop, 1974, i Reese , Sullivan, 1980, citirano prema [15].

Prema [13] verovatno prvi model p-y krivih uveli su McClelland and Focht (1958), preporučujući proceduru za korelaciju podataka triaksijalnog naponsko-deformacijskog opita sa krivama sila-pomeranje šipa za određene dubine, preko očekivanog modula reakcije tla, za svaki sloj tla, po dubini. Riz je prvi prikazao svoj koncept sloma tla oblika klina, koji se javlja blizu površine tla [19]. Uticaj variranja ulaznih parametara p-y krivih na odgovor šipa može se sagledati u [12].

2.2 Soil behaviour and its modelling during earthquakes

The answer to the question what occurs in the soil during earthquakes depends, primarily on the method of soil modelling in the structure-foundation-soil system, and then what earthquake and what effect are specifically investigated. The best answer to this question is probably provided by the wave mechanics, and its implementation in the numerical analyses using Solid elements of soil. It is a specific problem, because in the process of modelling, it requires finding an optimum for satisfying often confronted conditions. The parameters which need to be determined in the model definition process are: dimensions of the structural model, system model size, FE mesh size, frequency domain and types of propagating waves, boundary issues, etc. [17]. In this method, it is important to assess a minimum size of the model for the structure-foundation-soil system, in order to keep the calculation time as short as possible, but still to include all the necessary wave phenomena. Also, the maximum size of finite elements of soil must be determined, which must not be exceeded so as to avoid the undesirable wave effects in the actual finite elements of the soil, etc. We are discussing here the use of link elements in soil (LES), as Takeda envelopes of experimentally determined *p-y* curves (ATPY), because it is a simpler model for practical use, so the response will be based on it.

2.3 *P-Y* curves for piles in sand

Models of different levels of sophistication can be used for presentation of soil in an analysis of the dynamic interaction of a structure-foundation-soil system. The usual methods of seismic analysis of non-linear behaviour of structures are the quasi-static pushover NSA analysis and the non-linear dynamic NDA analysis in the time domain, as a numerical integration of accelerogram, a so-called step-by-step time history method. For the seismic action, accelerogram of El Centro, for PGA of 0.20; 0.25 and 0.30 *g* are used.

Soil can be modelled using various conditions of support, structure or piles, such as:

– Linear single-node springs, which equally resist tension and compression,

– Linear link elements

– Multi-linear plastic link elements which can be set so that only transfer compression.

Soil is modelled using the connection elements, so called link elements, according to the p-y sand model developed by Reese et al, Reese, Cox, Koop, 1974, and Reese, Sullivan, 1980, cited according to [15] .

According to [13] probably the first model of p-y curves was introduced by [10], which recommends the procedure for correlation of data of triaxial stress-strain test with the force-displacement curves of the piles for certain depths, via the expected soil reaction modulus, for every layer of soil, along the depth. Reese was the first to present his concept of wedge-like soil failure which occurs close to the soil surface [19]. The influence of variation of input parameters of p-y curves on the pile response can be observed in [12].

2.4 Pushover NSA – nelinearna statička analiza

U radu [3] detaljno su analizirane savremene metode za nelinearnu seizmičku analizu konstrukcija i način uvođenja prigušenja pri korišćenju neke od metoda. Ovde je ukratko prikazana pušover (PO) analiza u kojoj se određuju krive zavisnosti pomeranja kontrolnog čvora u_{max} (obično na vrhu rama) u odnosu na seizmičke smičuće sile u osnovi (BS-Base Shear), a za usvojen oblik raspodele opterećenja po visini objekta. Pretpostavlja se da usvojeni oblik opterećenja ostaje nepromenjen za sve stepene intenziteta, a time i deformisani oblik konstrukcije. Postepeno povećanje intenziteta opterećenja vrši se u koracima uz otvaranje plastičnih zglobova sve dok konstrukcije ne pređe u mehanizam. Kod konstrukcije pušover krivih, osim onih obaveznih po propisima, datih u EC8, poželjno je primeniti više različitih oblika raspodele opterećenja. Ovde su primenjeni sledeći oblici raspodele opterećenja po visini rama (odn. 2D modela zgrade):

- Konstantna raspodela (const).
- Linearno promenjiva (lin).
- Proporcionalno obliku prvog svojstvenog tona (1 mode) i
- Proporcionalno raspodeli (pripadajućih) masa (acc).

Takođe se mogu primeniti različiti tipovi prikaza PO krivih, a u SAP2000 su, za to, na raspolaganju:

1. Rezultantna sila u osnovi (BS) prema posmatranom pomeranju (MD),
2. ATC 40 metoda spektra kapaciteta,
3. FEMA 356 metoda koeficijenata,
4. FEMA 440 metoda ekvivalentne linearizacije, i
5. FEMA 440 metoda Modifikacije pomeranja.

2.4 Pushover NSA - non-linear static analysis

The paper [3] is analyzing the contemporary methods for non-linear seismic analysis of structures, and the ways how damping is introduced when using some of the methods. The pushover (PO) analysis is here briefly presented, which involves determination of curves which show the dependence of control node displacement u_{max} (typically at the top of the frame) with the seismic base shear (BS) force, for assumed shape of lateral load distribution along the height. It is assumed that the adopted form of load remains unchanged for all intensity levels, along with the structure's deformed shape. Gradual increase of the load intensity is performed in steps, along with the opening of plastic hinges up to a point where the structure becomes a mechanism. When constructing pushover curves, the use of several different shapes of load distributions is recommended, along with the ones prescribed by the regulations given in EC8. In this paper, the following shapes of load distributions along the frame height were applied:

- Constant distribution (const).
- Linear variable (lin).
- Proportional to the shape of the first natural mode (1 mode) and
- Proportional to the distribution of (corresponding) masses (acc).

In addition, different types of PO curve displays can be applied, and in the case of SAP 2000, the following ones are available:

1. Resulting base shear force (BS) according to the observed displacement (MD),
2. ATC 40 spectrum capacity method,
3. FEMA 356 coefficients method,
4. FEMA 440 equivalent linearization method, and
5. FEMA 440 displacement modification method.

3 REZULTATI PRORAČUNA I NJIHOVA ANALIZA

3.1 Rezultati NSA

Ovde su PO krive određene u programu SAP2000 v14, ali ne preko opcije *Display/Show Static Pushover Curve*, jer tada dijagram nije dovoljno pregledan, očitavanja vrednosti su nedovoljno precizna i ne mogu se vršiti odgovarajuće manipulacije, već je zbog toga to učinjeno preko putanje *Display/Show Plot Function*, dakle preko dijagrama funkcije *Umax/BS*. Takođe je PO kriva određena i prema proceduri FEMA356.

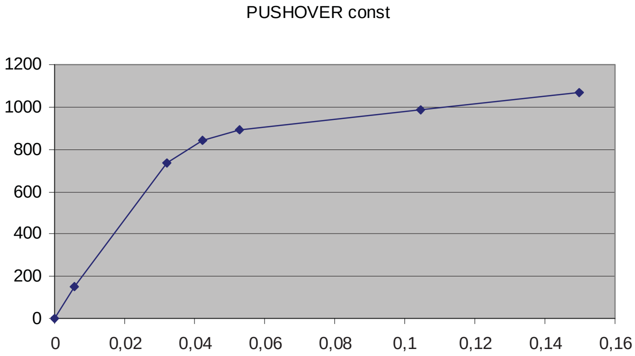
Na zbirnom dijagramu, za ovako određenje PO krive vidljiva je značajna razlika maksimalnih pomeranja kontrolnog čvora, u zavisnosti od oblika opterećenja, kao i razlike u početnoj inicijalnoj krutosti. Detaljnija analiza data je u tabeli 1.

3 CALCULATION RESULTS AND THEIR ANALYSIS

3.1 NSA results

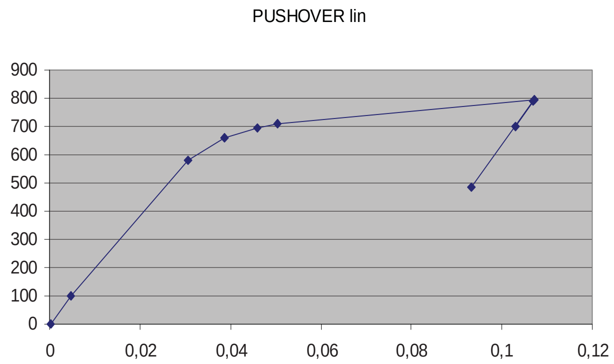
Here, the PO curves are determined using SAP 2000 v14 software, but not with the Display/Show Static Pushover Curve option, since in this case the diagram is not visible enough, reading of values from it is insufficiently accurate and appropriate manipulations cannot be performed. Thus, the above process is performed using the path Display/Show Plot Function, i.e. by using the function diagram U_{max}/BS. In addition, the PO curve is also determined according to the FEMA 356 procedure.

In the summary diagram, for PO curves compared in this way, there is a noticeable difference of maximum control node displacement, depending on the load shape, along with a difference in initial stiffness. A more detailed analysis data are given in table 1.



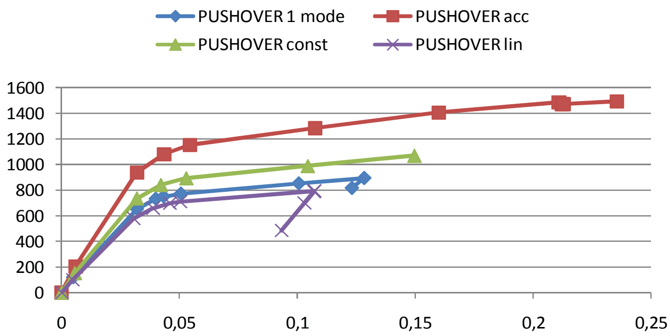
Slika 2a. Pušover kriva. Konstantna raspodela opterećenja po visini. Sila u osnovi BS=1069 kN, maksimalno pomeranje umax=14,97 cm.

Figure 2a. Pushover curve. Constant load shape along height BS=1069 kN, umax=14.97 cm.



Slika 2b. Pušover kriva. Linearna raspodela opterećenja po visini BS=793,1 kN, umax=10,73 cm.

Figure 2b. Pushover curve. Linear distributed load shape along height BS=793.1 kN, umax =10.73 cm.



Slika 3. Zbirni dijagram Pušover krivih za 4 oblika Raspodele opterećenja: linearna, 1 mode, konstantno (const) i proporcionalno masama acc.

Figure 3. Summary diagram Pushover curves, for 4 shapes load distribution: linear, 1 mode, const. and acc.

Tabela 1. Komparativni prikaz maksimalnih pomeranja čvora u vrhu i sila u osnovi u zavisnosti od oblika opterećenja. Kod vremenske analize u zavisnosti od PGA. Linearizovana krovna greda [6].

Table 1. Comparative analysis of max top node displacements and Base Shear, with respect to load shape. In TH with respect to PGA. Linear roof beam. [6].

	PGA (*g*) El Centro			Način distribucije vertikalnog opterećenja. *Distribution of vertical load*			
	0.20*g*	0.25*g*	0.30*g**	PO lin	PO const	PO acc	PO 1 mode
BS (kN)	1312	1615	1899	793.10	1068.65	1492.66	893.87
Umax (cm)	8,56	11,29	14,47	10.73	14.97	23.54	12.83
FEMA 356 C							
BS (kN)				798.67	1076.10	1504.40	900.60
Umax (cm)				27.3	26.9	24.3	28.4

* cut off at 7. 2 sec; FEMA 356 C - Site class C; Pushover= PO

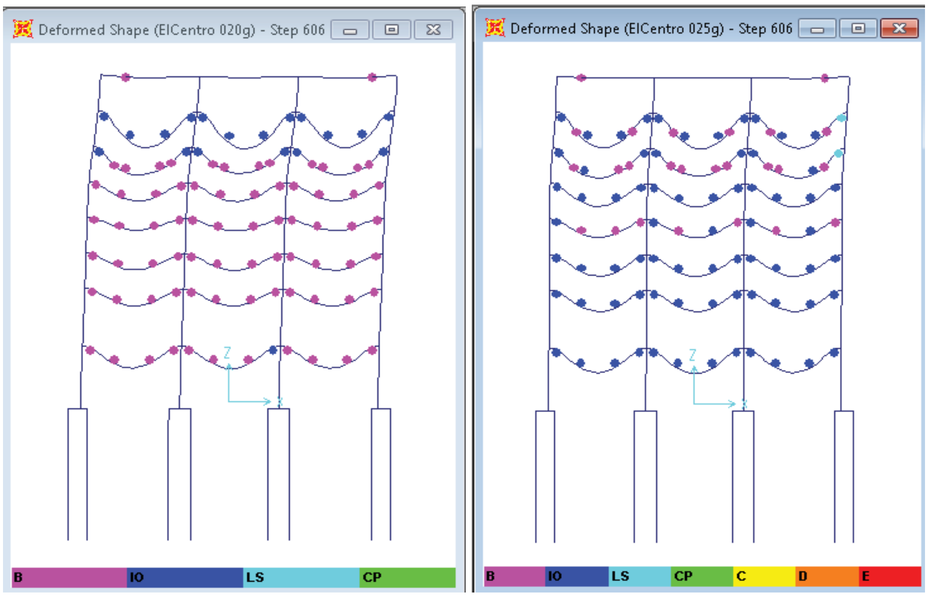
Razmatrani ram je svestrano tretiran, a s obzirom da je pre TH (NDA) analize preuzeto naponsko stanje konstrukcije od sopstvene težine, linearizovane krovne grede razmatranog rama [6] ostaju prave (slika 4). Kod nelinearnih se, nasuprot tome, jasno uočavaju ugibi što će biti prikazano u delu analiza rezultata na slikama 27, 28 i 29. Kod analize link elemenata, nelinearni su i ovi čvorovi, ali ni ovde plastični zglobovi nisu uneti na sredinama greda. Model rama sa plastičnim zglobovima i u sredinama greda, prikazan je na kraju rada, samo na

The considered frame is comprehensively treated, and since before the TH (NDA) analysis, the stress state of the structure under its self weight is taken, the linearized roof beams of the observed frame [6] remain straight (figure 4). In the non-linear ones, on the contrary, there are clearly observable deflections which will be presented in the section analysis of results in figures 27, 28 and 29. In the analysis of link elements, the nodes are non-linear, but the plastic hinges in the middle of the beams are not assumed. The model of the slici 31, uz kratak osvrt na problematiku istog.

Nelinearna dinamička analiza, urađena je za akcelerogram El Centro za vršne vrednosti PGA od 0.20, 0.25 i 0.30 *g*. Razmatrano je pomeranje čvora u vrhu i ukupne seizmicke sile u osnovi. Proveravana su stanja plastičnih zglobova (loma) na kraju svakog zemljotresa.

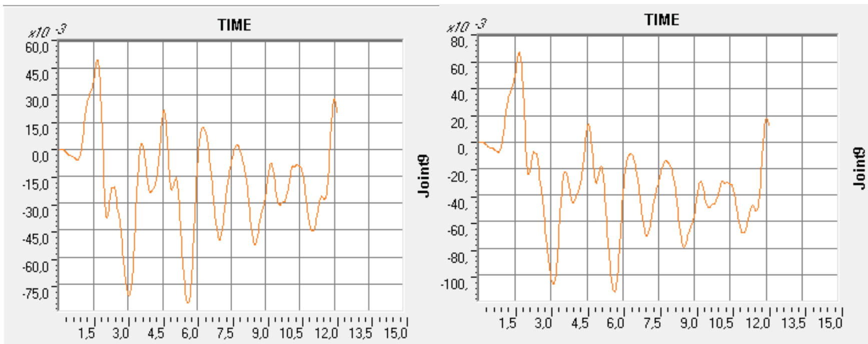
frame with plastic hinges in the middle of beams as well, is presented at the end of the paper, in figure 31 only, with a short comment on the issue.

Non-linear dynamic analysis is performed for the El Centro ground motion record, for peak PGA values 0.20, 0.25 and 0.30 *g*. Node displacements at the top and the seismic base shear are considered. The states of plastic hinges (failure) are checked at the end of each earthquake.



Slika 4. Stanje plastičnih zglobova PHS na kraju zemljotresa El Centro, levo PGA 0,20g PHS: 79Y, 19 IO, desno PGA 0,25g PHS: 71Y, 25 IO i 2 LS. Linearna krovna greda

Figure 4. State of plastic hinges (PHS) at the end earthquake ElCentro, left PGA 0.20g PHS: 79Y, 19 IO, right PGA 0.25g PHS: 71Y, 25 IO and 2 LS. Linear roof beams



Slika 5. Dijagram pomeranja čvora u vrhu zgrade tokom akcelerograma El Centro levo PGA 0,20g, Umax=8,56cm, desno PGA 0,25g, Umax =11,29 cm. Linearna krovna greda.

Figure 5. Displacement plot of the joint at the top of the building, due earthquake acc. El Centro: left PGA 0.20g, Umax =8.56cm, right PGA 0.25g, Umax =11.29 cm Linear roof beams.

Koeficijent proporcionalnosti i za sile i za pomeranja, kod prelaska sistema sa više stepeni slobode kretanja, (MDOF) na sistem sa jednim stepenom slobode (SDOF), je:

The proportionality coefficient, both for the forces and for displacements, during transition from a mullti degree of freedom system (MDOF) to a single degree of freedom system (SDOF) is:

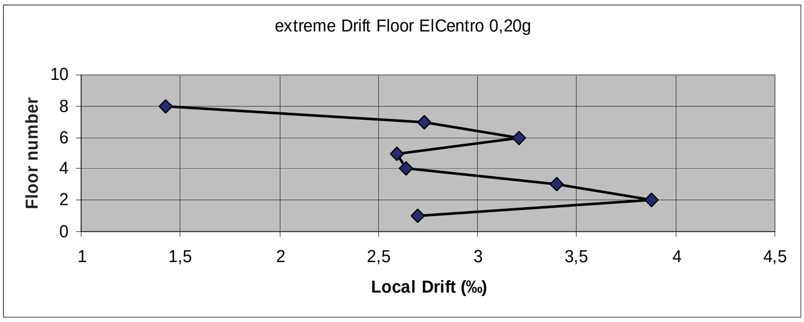
$$\Gamma = \frac{\Phi^T m 1}{\Phi^T m \Phi} = \frac{\sum m_i \Phi_i}{\sum m_i \Phi_i{}^2} = \frac{m*}{\sum m_i \Phi_i{}^2} \qquad (1)$$

3.2 Driftovi stubova za različite vrednosti PGA

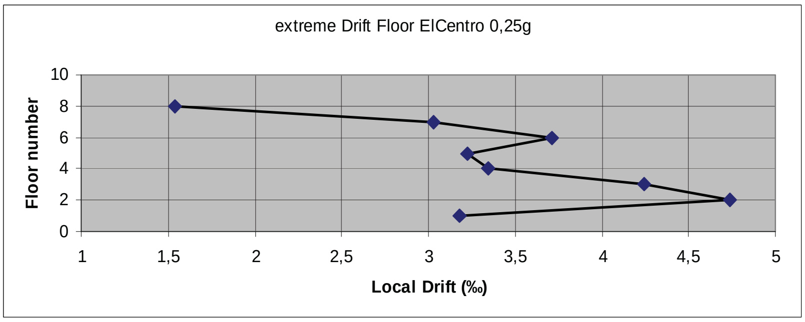
Sa nelinearnom (NL) krovnom gredom i celim NL 2D ramom, sračunate su ekstremne vrednosti drifta stubova i njihova promena tokom dejstva akcelerograma El Centro, za PGA 0,20; 0,25 i 0,30 *g*.

3.2 Column drifts for different values of PGA

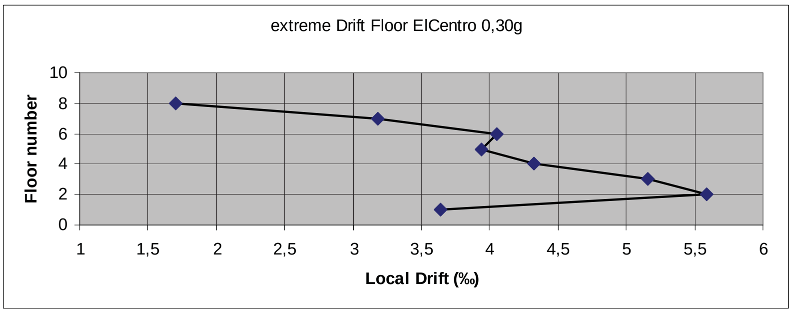
For the non-linear roof beam, and entire 2D frame, the extreme values of column drift and their variation during action of accelerogram of El Centro, for PGA of 0.20; 0.25 and 0.30 g, are calculated.



Slika 6a. Ekstremni spratni drift (stuba) El Centro 0,20g. Prekoračuje dozvoljene vrednosti
Figure 6a. El Centro 0.20g. Extreme Local Drift (column) exceeds permissible values



Slika 6b Ekstremni spratni drift (stuba) El Centro 0,25g. Prekoračuje dozvoljene vrednosti
Figure 6b El Centro 0.25g. Extreme Local Drift (column) exceeds permissible values



Slika 6c. Ekstremni spratni drift (stuba) El Centro 0,30g Prekoračuje dozvoljene vrednosti
Figure 6c. El Centro 0.30g. Extreme Local Drift (column) exceeds permissible values

Slika 6c se razlikuje po obliku u odnosu na 6a i 6b, za PGA 0,20 i 0,25*g*.

Figure 6c, is different in shape from Figs. 6a and 6b, which are given for PGA 0.20 and 0.25*g*.

3.3 *P-y* krive

P-y kriva (slika 7) se sastoji iz 4 dela, prvi linearni od koordinatnog početka do tačke *k*, drugi eksponencijalni deo od *k* do *m*, i treći deo je druga linearna funkcija od *m* do *u*, a posle tačke *u*, *p-y* kriva je konstantna prava.

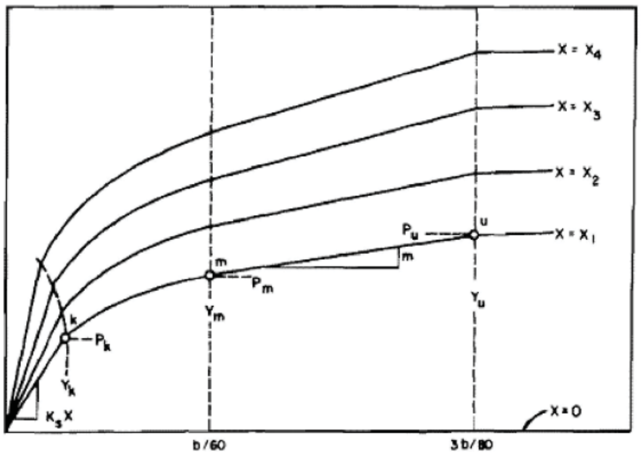
Koeficijenti redukcije (slika 8) A i B zavise od vrste opterećenja, a za dinamičku analizu koriste se krive cikličnog opterećenja. Koeficijenti A i B su dati na

3.3 *P-y* curves

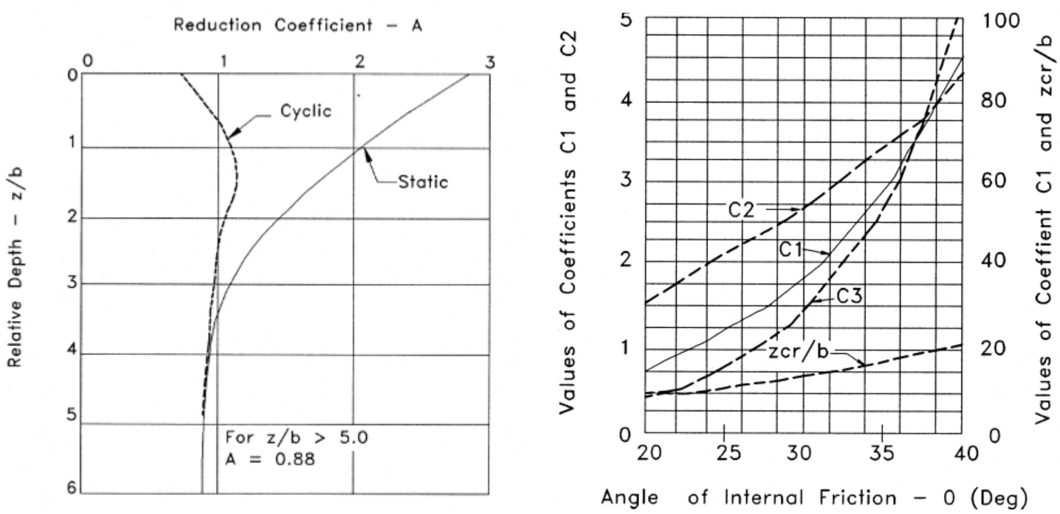
P-y curve (Fig. 7) consists of 4 parts: the first is linear from the coordinate beginning till the point u, the second is exponential part from k to m, the third is the second linear function from m to u, while after the point u, p-y line remains constant.

Reduction coefficients A and B (Fig. 8) depend on the type of the load, and for dynamic analysis, the dijagramu u intervalu od 0 do 6 z/b. Posle 5 dijametara šipa koeficijenti A i B imaju konstantnu vrednost. z_{cr} je dubina posle koje se oblik loma klinom menja u oblik loma blokom.

curves of cyclic load are used. The coefficients A and B are given in the graph in the interval 0 to z/b. After 5 pile diameters, coefficients A and B have a constant value. z_{cr} is the depth after which the wedge-like failure transforms into the block-like failure.



Slika 7. Konstrukcija karakterističnih oblika p-y krivih, Ris, Koks, Kup i dr. 1974, citirano prema [4]
Figure 7. Construction of characteristic shapes of p-y curves Reese, Cox, Coop at all. 1974, after [4]



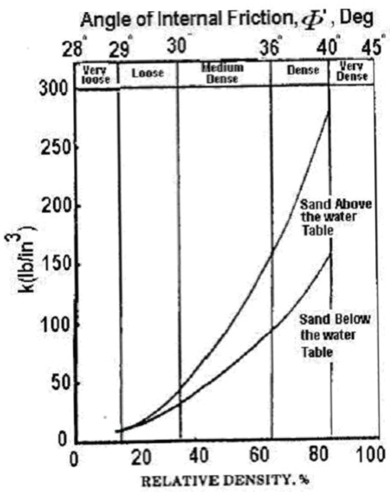
Slika 8. levo Koeficijenti redukcije A i B; Slika desno Faktori za sračunavanje granične otpornosti tla za horizontalno opterećen šip u pesku C_1,C_2,C_3 i z_{cr}, u odnosu na ugao unutrašnjeg trenja [15].

Figure 8. left Reduction coefficient A and B; right Factor for calculation of ultimate soil resistance for horizontal loaded pile in section C_1,C_2,C_3 and z_{cr}, related to the friction angle, after [15].

Tabela 2. Koeficijent horizontalne reakcije tla za pesak. Početni nagib p-y krive, u funkciji relativne krutosti i nivoa podzemne vode, potopljen i suv pesak.

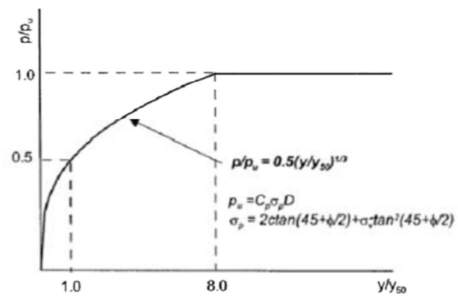
Table 2. Coefficient of horizontal reaction for sand. Initial inclination p-y curve vs. relative density and level below ground water (submerged and dry sand).

Soil modulus k	parametar k za relativni stepen zbijenosti peska		
Realtivna zbijenost: *Relative density:*	Rastresit *Loose*	Srednje zbijen *Medium dense*	Krut *Dense*
Potopljen pesak *Submerged sand*	5.430 kPa/m	16.300 kPa/m	33.900 kPa/m
Suv, iznad NPV *Dry sand, above water level*	6.790 kPa/m	24.430 kPa/m	61.000 kPa/m



Slika 9. Početni modul horizontalne reakcije tla u zavisnosti od zbijenosti i ugla unutrašnjeg trenja, API (American Petroleum Institute), prema [4]

Figure 9. Initial modul of lateral reaction of soil in function of compaction and internal angle of friction, API (American Petroleum Institute), after [4]



Slika 10. Karakteristične krive za pesak sa uticajem kohezivnog dela, Rees i dr. [4]

Slika 10. Caracteristic curve for sand with influences of cohesive part, Rees at all [4]

Marchinson and Oneill, 1984 [15], pojednostavili su gornju proceduru i tri dela prave zamenili sa jednom jednačinom, kao što sledi:

Marchinson and Oneill, 1984 [15] simplified the above process, and replaced the three part *p-y* curve with a single analytical equation, as follows:

$$\frac{p}{p_u} = n \cdot A \cdot \tanh\left(\frac{k_H}{n \cdot A \cdot p_u} y\right) \tag{2}$$

p_u – granična horizontalna otpornost na dubini H od površine tla,

k_H – krutost tla, početni modul horizontalne reakcije (prema tabeli 2, za pesak),

y – horizontalno pomeranje šipa,

n – geometrijski faktor, =1,0 za prizmatične šipove,

A = 0,9 za ciklično opterećenje, (3-0,8(z/b)) ≥ 0,90 za statičko opt., z dubina za koju se p/y kriva određuje.

p-y krive su eksperimentalno izvedene za statičko i ciklično opterećenje, tako da kada koristimo ciklične krive za dinamičko opterećenje, ipak još uvek koristimo relativno mirno opterećenje, gde se mogu uhvatiti samo efekti ponavljanja opterećenja, ali ne i u potpunosti dinamički uticaji.

Where:

p_u – ultimate lateral soil resistance at a depth H below ground surface,

k_H – soil stiffness, initial modulus of lateral reaction (according to table 2, for sand)

y – lateral displacement of pile

n – geometric factor, =1.0 for prismatic piles

A = 0.9 for cyclic load (3-0.8(*z/b*)) ≥ 0.90; for the static load, for the depth z applies the *p-y* curve.

p-y curves are experimentally derived for static and cyclic load, so when the cyclic curves are used for dynamic loading, it is still a relatively calm loading, so only the effects of loading repetition can be assessed, but not the complete dynamic effects.

Tabela 3. p-y kriva: φ= 35°; D=0,60 m; γ= 10 kN/m^3; k= 33900 kPa/m.
Table 3. p-y curve: φ= 35°; D=0.60 m; γ= 10 kN/m^3; k= 33900 kPa/m.

i	z	k_o	y_a	p_a=p_k	p_b=p_m	p_c=p_u
1	1	33900	8.15E-04	27-64	42.86	52.23
2	2	67800	7.00E-04	47.47	105.34	144.80
3	3	101700	3.53E-04	35.90	178.72	285.95
4	4	135600	5.63E-04	76.29	303.64	485.82
5	5	169500	8.18E-04	138.73	461.23	737.98
6	6	203400	1.12E-03	227.78	651.51	1042.41
7	7	237300	1.47E-03	347.96	874.45	1399.13
8	8	271200	1.86E-03	503.70	1130.07	1808.12
9	9	305100	2.29E-03	699.40	1418.37	2269.39
10	10	339000	2.77E-03	939.43	1739.34	2782.95
11	11	372900	2.88E-03	1074.71	1952.70	3124.33

Tabela 4. p-y kriva za φ= 35°; D=0,60 m; γ= 10 kN/m3; k= 33900 kPa/m
Table 4. p-y curves for φ= 35°; D=0.60 m; γ= 10 kN/m3; k= 33900 kPa/m

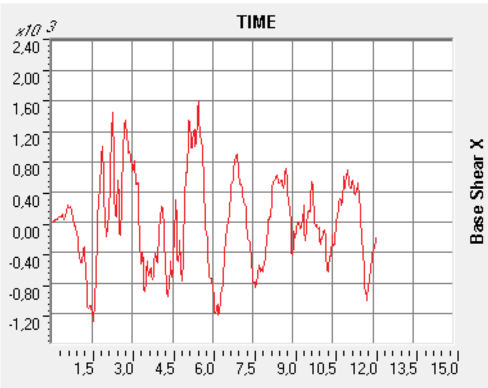
k_o=	33900	k_o=	67800	k_o=	101700	k_o=	135600
z=	**1**	z=	**2**	z=	**3**	z=	**4**
p_c=	52.23	p_c=	144.80	p_c=	285.95	p_c=	485.82
y (m)	*p* (kPa/m)	*y* (m)	*p*(kPa/m)	*y* (m)	*p*(kPa/m)	*y* (m)	*p*(kPa/m)
0.000001	0.00001	0.000001	0.00001	0.000001	0.00001	0.000001	0.00001
0	0	0	0	0	0	0	0
-0.001	-29.82	-0.001	-63.24	-0.001	-97.62	-0.001	-132.19
-0.002	-44.98	-0.002	-106.22	-0.002	-174.86	-0.002	-246.15
-0.003	-50.15	-0.003	-128.35	-0.003	-225.42	-0.003	-332.50
-0.0045	-51.93	-0.0045	-140.58	-0.0045	-263.57	-0.0045	-412.93
-0.005	-52.07	-0.005	-142.14	-0.005	-270.08	-0.005	-429.66
-0.007	-52.22	-0.007	-144.39	-0.007	-282.04	-0.007	-466.69
-0.009	-52.23	-0.009	-144.74	-0.009	-285.00	-0.009	-479.47
-0.01	-52.23	-0.01	-144.78	-0.01	-285.48	-0.01	-482.18
-0.015	-52.23	-0.015	-144.80	-0.015	-285.94	-0.015	-485.60
-0.02	-52.23	-0.02	-144.80	-0.02	-285.95	-0.02	-485.81
-0.0229	-52.23	-0.0229	-144.80	-0.0229	-285.95	-0.0229	-485.82
-0.025	-52.23	-0.025	-144.80	-0.025	-285.95	-0.025	-485.82
-0.03	-52.23	-0.03	-144.80	-0.03	-285.95	-0.03	-485.82
-0.035	-52.23	-0.035	-144.80	-0.035	-285.95	-0.035	-485.82
-0.18	-52.23	-0.18	-144.80	-0.18	-285.95	-0.18	-485.82

U pokušaju da se što bolje obuhvate i dinamički uticaji, link elementi su modelovani preko više linearnih plastičnih elemenata histerezisnog tipa, gde je ciklična p-y kriva poslužila kao svojevrsna anvelopa. Krive su sračunate kombinacijom obe metode, za prvu je korišćen program koji sračunava parametre krive za svaki metar dubine, rezultati su dati u tabeli 3, a uvedene su u SAP2000 kao što sledi u tabeli 4.

In order to include dynamic effects as much as possible, the link elements are modelled using multiple linear plastic elements of hysteretic type, where the cyclic p-y curve is used as kind of an envelope. The curves are calculated by combining both methods, and for the former the software is used which calculates the curve parameters for each meter of depth, the results being provided in table 3. Also, they are introduced in SAP2000 as given in table 4.

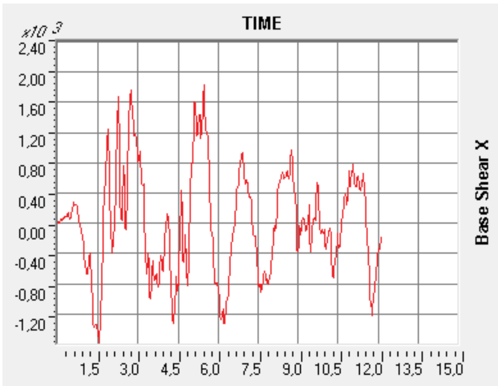
3.4 Sile u osnovi usled razmatranog seizmičkog dejstva

3.4 Base shear force due to the considered seismic action



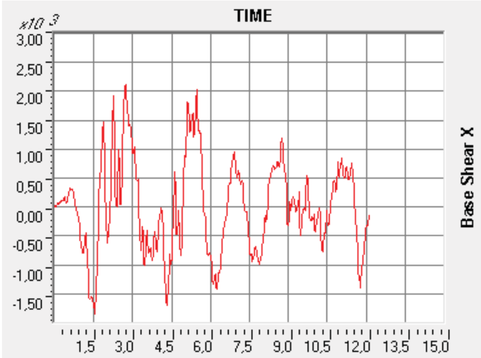
Slika 11a. Sila u osnovi X. ElCentro PGA 0,20 g. BS max 1608 kN (5,460 sec). BS min 1304 kN (1,540 sec).

Figure 11a. Base Shear X. ElCentro PGA 0.20 g. BS max 1608 kN (5.460 sec). BS min 1304 kN (1.540 sec).



Slika 11b. Sila u osnovi X. ElCentro PGA 0,25 g. BS max 1834 kN (5,460 sec). BS min 1582 kN (1,540 sec).

Figure 11b. Base Shear X. ElCentro PGA 0.25 g. BS max 1834 kN (5.460 sec). BS min 1582 kN (1.540 sec).



Slika 11c. Sila u osnovi X. ElCentro PGA 0,30 g. BS max 2114 kN (2,720 sec). BS min 1854 kN (1,540 sec)
Figure11c. Base Shear X. ElCentro PGA 0.30 g. BS max 2114 kN (2.720 sec). BS min 1854 kN (1.540 sec)

Sva tri grafika sila u osnovi su vrlo slična. Međutim primećuje se da je vršna vrednost maksimuma za 0,30g, pomerena sa 5,46 sec na 2,72 sec. Prema istraživanjima [18], kod analize uticaja akcelerograma, nije bitno samo vršno ubrzanje tla PGA, već je neophodno posmatrati i neposrednu okolinu, i uočiti na koji način je maksimum spregnut sa susednim ekstremima. To se ovde primenjuje i kod sile u osnovi.

All three graphs of the BS forces are basically very similar. However it can be observed that the peak value for 0,30g, changes from 5.46 sec to 2.72 sec. According to the research [18], during the accelerogram (i.e. time history) analysis, not only peak ground acceleration (PGA) is important, but it is necessary to observe the immediate surroundings of the peak, and find out in which way the peak is related to the adjacent peaks. It is implemented here for the Base Shear force.

Tabela 5. Zavisnost sile u osnovi i PGA(g). Trenutak max i min.
Table 5. Variation of base shear force with respect to PGA (g). Instances of max and min

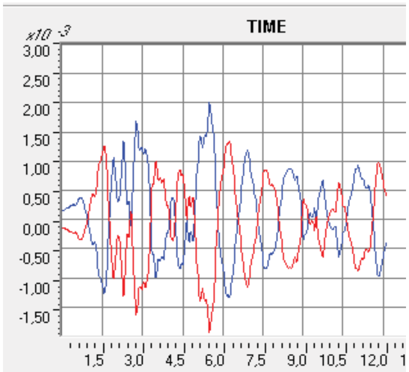
ElCentro PGA (*g*)	Base Shear max (kN)	Base Shear min (kN)	t max (sec)	t min (sec)
0.20	1608	1304	5.460	1.540
0.25	1834	1582	5.460	1.540
0.30	2114	1854	2.720	1.540

3.5 Uticaji u link elementima iz NDA (TH) za dejstvo „El Centro“ i preko rada

Kao rezultat ove analize na sl. 12 prikazani su dijagrami pomeranja i sila spregnutih parova link elemenata 1 i 2 (dubina 1 m), za PGA 0.20*g*. Ovi upareni elementi su spregnuti u istom čvoru šipa.

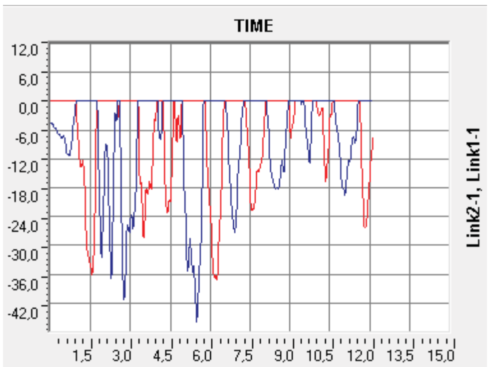
3.5 Effects in the link elements from NDA (TH) action of „El Centro“ and via the work

As a result of this analysis Fig. 12 presents displacement and force diagrams of coupled link elements 1 and 2 (depth 1 m), for PGA 0,20*g*. Coupled link elements are related to the same node.



Slika 12a. Link 1 i 2, nivo -1,0 m. PGA 0,20g El Centro NDA. Pomeranje: max 0,201cm. min 0,194 cm

Figure 12a. Link1 and 2, level -1,0 m. PGA 0,20g El Centro NDA. Displacement: max 0.201cm. Min 0.194 cm 0.201cm. min 0.194 cm.

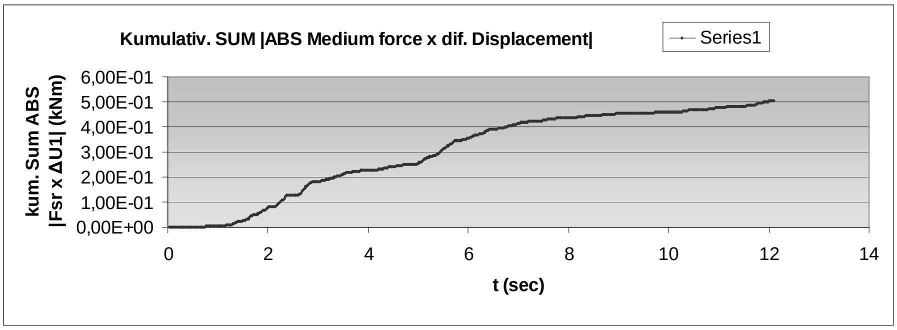


Slika 12b. Link 1 i 2, nivo -1,0 m. PGA 0,20g El Centro NDA. Sila. max 45,88 kN

Figure 12b. Link 1 and 2, level -1.0 m. PGA 0.20g El Centro NDA. Force. max 45.88 kN

Pored nelinearne analize razmatran je apsolutni rad, uparenih link elemenata, kao pozitivna vrednost dejstva sile duž puta. Dijagram kumulativnog Apsolutnog rada, uparenih link elemenata prikazan je na sl. 13. Apsolutni rad je neophodan kako ne bi došlo do poništavanja pozitivnog i negativnog rada tokom sumiranja. Negativni rad je posledica množenja sile u link elementu sa negativnim predznakom, sa pozitivnim predznakom pomeranja istog. Na slici 13 uočljivi su strmiji delovi kumulativne krive, koji predstavljaju mesta na kojima se "gomila" intenzitet akcelerograma koji utiče na rad link elemenata. Tu se zapravo radi o disipaciji seizmičke energije u tlu, na oko 1 m dubine od površine tla.

In addition to the non-linear analysis, the Absolute work of coupled link elements is determined as a positive value of force action along the path. The diagram of cumulative Absolute work of the coupled link elements is presented in Fig. 13. The absolute work is necessary to avoid cancelation of the positive and negative works during addition. The negative work is a consequence of multiplication of a force in the link element with a negative sign, with positive displacement. In Figure 13 steep sections of the cumulative curve are noticeable, which represent the locations where the intensity of the accelerogram which affects the work of link elements is "piling up". It is actually the case of dissipation of seismic energy in soil, at around 1 m bellow the surface.



Slika 13. Kumulativni Apsolutni rad link elemenata tokom dejstva El Centro. Link 1 i 2, nivo -1,0 m dubina tla. PGA 0,20g ELCentro NDA.

Figure 13. Cumulative Absolute work of link elements under action of El Centro. Link 1 and 2, level 1,0 m depth below ground surface. PGA 0.20g ELCentro NDA.

Nadalje, u tabelama su navedene karakteristične vrednosti pojedinih uticaja u link elementima.

Further, the tables 6 to 9 show the characteristic values of individual parameters of the link elements.

*Tabela 6. Link 1 i 2 Trenutni rad= Sila * Pomeranje F_i x U_{1i} (kNm) El Centro 0,20g.*
*Table 6. Link 1 and 2 " Instantenous work" = Force * Displacement F_i x U_{1i} (kNm) El Centro 0.20g.*

	Link 2	Link 1	ElCe 0.20*g*
min	4.98E-02	8.90E-02	
max	-7.04E-08	-4.79E-07	
extr	0.04982	0.089016	
Suma	2.945	4.522	7.467
%	39.44	60.56	

Ovde se kao trenutni rad ne posmatra čist rad, kao dejstvo sile duž puta, već samo trenutna vrednosti sile (F_i), kao reakcije link elementa u trenutku (t_i) pomnožene sa trenutnom (ekstremnom) vrednošću odgovarajućeg pomeranja čvora (U_{1i}) u kome se sustiču link elemenat i konačni elementi šipa, u datom trenutku dejstva akcelerograma. Zato je i napisan proizvod sile i pomeranja, tj. sila x pomeranje. Obe ove veličine su linearne funkcije, u posmatranim vremenskim intervalima, te se u opštem smislu (kao integral) množi trougao sa trapezom (ovde trapez zamenjen pravougaonikom) čija je ordinata srednja vrednost sile. Strogo uzevši trebalo bi, dakle, posmatrati srednju vrednost sile (F_{sr}) i razliku pomeranja (Δ_{U1}), u svakom pojedinačnom intervalu vremena, kao čisti rad. Čisti rad link elemenata dat je u tabeli 7, i to su vrednosti manjeg reda veličine od prethodno navedenog trenutnog rada. S

Here, instantaneous work is not considered as an effective work, as an action of the force along the path, but only as the instantaneous value of the force (F_i), as a reaction of the link element at a time (t_i) multiplied by the instantaneous (extreme) value of the corresponding node displacement (U_{1i}) at which the link element and the finite elements of the pile join together, at a given moment of ground motion action. That is why it is written as a product of the force and displacement, i.e. Force x Displacement. Both parameters are linear functions, at the observed time intervals, so in the general sense (as an integral) a triangle is multiplied by a trapezoid (here a trapezoid is replaced by a rectangle) whose ordinate is a mean force value. Strictly speaking, one should observe the mean force value (F_{sr}) and displacement difference (Δ_{U1}), at each individual interval of time, as effective work. Effective work of elements is provided in table 7, obzirom da su izlazni podaci, za razliku od ulaznih, dati u jednakim vremenskim intervalima od 0,02sec, iz vrednosti čistog rada se direktno može dobiti i snaga po link elementu množenjem sa 1/0.02=50 Hz (J/sec=Watt).

and those are the values of a lower order of magnitude than the previously mentioned instantaneous work. Regarding that the output data, as opposed to the input data, are given at equal time intervals of 0.02sec, from the value of the effective work one can directly calculate the power by a link element, by multiplying with 1/0,02=50 Hz (J/sec=Wat).

Tabela 7. Link 1 i 2. Apsolutni rad, ABS |$A = F_{sr} \times \Delta_{U1}$| (kNm); El Centro 0,20g
Table 7. Link 1 i 2. Abs. work, ABS |$A = F_{sr} \times \Delta_{U1}$| (kNm); El Centro 0.20g

	Link 2	Link 1	
min	3.96E-03	6.77E-03	
max	0.00E+00	0.00E+00	
extr	0.00396	0.006773	Suma
Suma	0.197484	0.308631	0.506115
	39.02	60.98	%

Ekstremne vrednosti suma Abs rada sila u uparenim link elementima su različite. Za gornji slučaj je to odnos 39/61=0.64. To je posledica uvedenog nelinearnog ponašanja link elemenata i nesimetrije akcelerograma.

The extreme values of sums of Absolute work of the forces in the coupled link elements are different. In this case, it is the ratio 39/61=0.64. It is a consequence of the introduced non-linear behaviour of link elements and asymmetry of accelerogram.

Tabela 8. Link 1 i 2 Pomernje (m); Sila – reakcije (kN); ElCentro 0,20 g
Table 8. Link 1 and 2 Displacement (m); Force – reaction (kN); ElCentro 0.20 g.

Extr	Link 2	Link 1
Displac.(m)	0.00201	0.00194
Force (kN)	37.18	45.88

Ekstremne vrednosti sila i pomeranja u uparenim link elementima su različite. Za gornji slučaj je to odnos za sile 37/46=0,81 što nije zanemarljivo, a za pomeranja 201/194=1,036 što su bliske vrednosti.

The extreme values of forces and displacements in the coupled link elements are different. In this case, the force ratio is 37/46=0.81, which is not negligible, but obtained displacement ratio is 201/194=1.036 which are close values.

Table 9. Promena ekstremnog pomeranja (cm) i sila (kN) u link elementima sa porastom PGA El Centro.
Table 9. Variation of extreme displacement in (cm) and force in (kN) in link elements under PGA El Centro

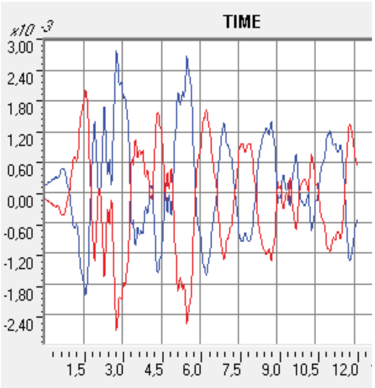
	pomeranja	pomeranja	sile	Sile
PGA (*g*)	Link 1 i 2	Link 3 i 4	Link 1 i 2	Link 3 i 4
0.20	0.201	0.100	45.88	65.26
0.25	0.231	0.114	47.65	72.71
0.30	0.281	0.145	49.71	85.81

Postoji jaka linearna zavisnost između vršnog ubrzanja PGA i pomeranja Link elementa. Približno za Link 1 i 2: U1≈0.95 · PGA(*g*); a za Link 3 i 4: U1≈0,48 · PGA(*g*). Što se tiče sila Link elementa (*y*), linearna zavisnost između PGA ($x=a_h/g$) i istih, određena je preciznije tehnikom najmanjih kvadrata, i za Link 1 i 2: y = 38.3·x + 38.172, (R^2 = 0,9981); a za Link 3 i 4: *y* = 205,5·*x* + 23.218, (R^2 = 0,9754). Kod Link elemenata 1 i 2, usled porasta PGA od 0,20g do 0,30 *g*, pomeranja rastu za oko 40%, dok kod Link 3 i 4, za istu promenu PGA ekstremno pomeranje raste za 45%.

Nadalje su, na slikama 14 do 24, prikazani dijagrami sila i pomeranja Link elemenata za El Centro PGA 0,3g.

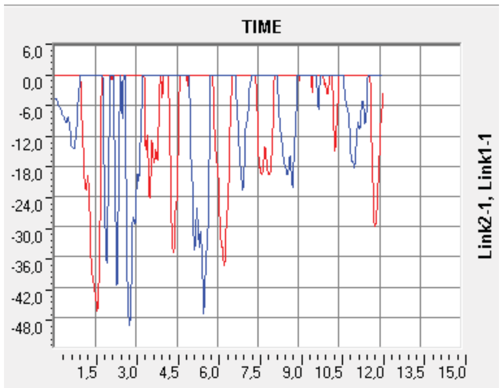
There is a strong linear dependence between the peak ground acceleration PGA and Link element displacement. Approximately, for the Links 1 and 2 it is: U1≈0.95 * PGA (*g*); and for the Links 3 and 4: U1≈0.48*PGA (*g*). As for the forces of the Link element (y), the linear dependence between PGA ($x=a_h/g$) and forces is more accurately determined using the least square technique. For the Links 1 and 2 it is: *y* = 38.3·*x* + 38.172, (R^2 = 0.9981), while for the Links 3 and 4 it is: y = 205.5·x + 23.218, (R^2 = 0.9754). In the case of Link elements 1 and 2, due to the increase of PGA from 0.20*g* to 0.30 *g*, the displacements increase for around 40%, while for the Links 3 and 4, for the same change of PGA the extreme displacement increases for 45%.

Further in the text, Figures 14 to 24 are presenting the force and displacement diagrams of the Link elements for El Centro PGA 0.30 g.



Slika 14a. Link 1 i 2, nivo -1,0 m. PGA 0,30g ElCentro NDA. Pomeranje: max 0,281cm. min 0,272 cm

Figure 14a. Link 1 and 2, level -1.0 m. PGA 0.30g El Centro NDA. Displacement. max 0.281cm. min 0.272 cm

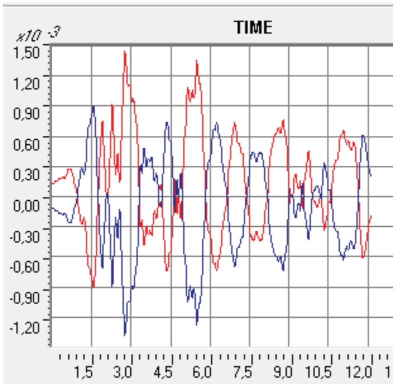


Slika 14b. Link 1 i 2, nivo -1,0 m. PGA 0,30g ElCentro NDA. Sila max 49,71 kN

Figure 14b. Link 1 and 2, level -1.0 m. PGA 0.30g El Centro NDA. Force. max 49.71 kN

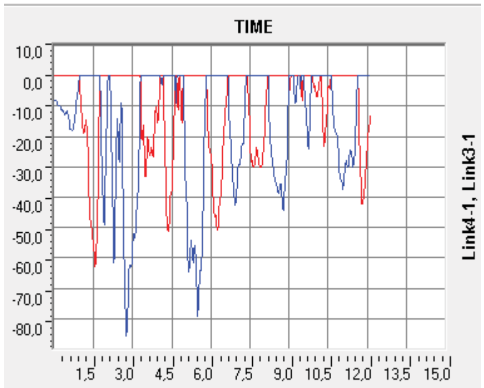
Primetne su praznine u silama reakcija Link elemenata 1 i 2. Na oko 3,9 sec, zatim 4,6sec, a na 9 sec je najveća pauza u reakcijama sila Link elementa 1 i 2. To bi mogao biti znak da je došlo do odvajanja (gap) na kontaktu šipa i tla.

There are noticeable gaps in the reaction forces of Link elements 1 and 2. They are at around 3.9 sec, then at 4.6sec, and at 9 sec there is the largest gap in the force reactions of Link elements 1 and 2. It could be a sign that there is a gap between the pile and the soil.



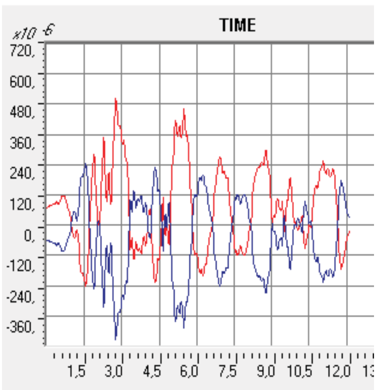
Slika 15a. Link 3 i 4, nivo -2 m. PGA 0,30g El Centro NDA. Pomeranje: max 0,281cm, min 0,272 cm.

Figure 15a. Link 3 and 4, level -2 m. PGA 0.30g EL Centro NDA. Displacement: max 0.281cm, min 0.272 cm.



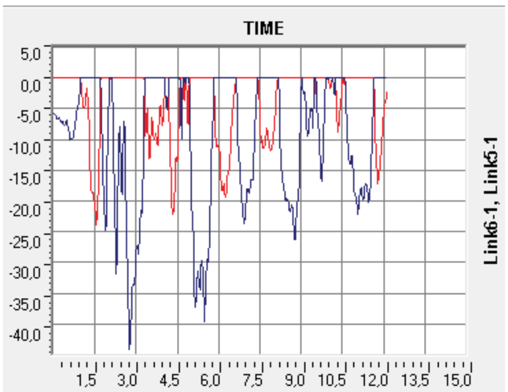
Slika 15b. Link 3 i 4, nivo -2,0 m. PGA 0,30g El Centro. NDA. Sila max 85,81 kN.

Figure 15b. Links 3 and 4, level -2.0 m. PGA 0.30g El Centro. NDA. Force max 85.81 kN



Slika 16a. Link 5 i 6, nivo -3,0 m. PGA 0,30g El Centro NDA. Pomeranje: max 0,0505 cm, min 0,0452 cm.

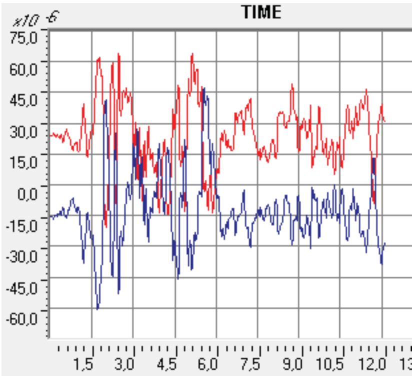
Figure 16a. Link 5 and 6, level -3.0 m. PGA 0.30g El Centro NDA. Displacem.: max 0.0505 cm, min 0.045 cm.



Slika 16b. Link 5 i 6, nivo -3,0 m. PGA 0,30g El Centro. NDA. Sila max 44,10 kN.

Figure 16b. Links 5 and 6, level -3.0 m. PGA 0.30g El Centro. NDA. Force max 44.10 kN

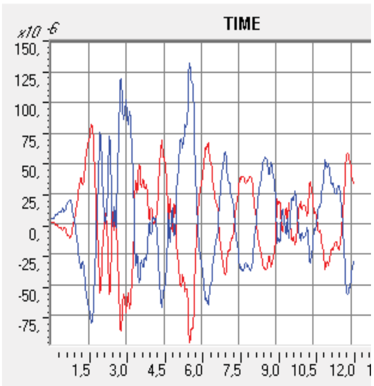
Iako je po vremenu trajanja raspodela sila gotovo ujednačena Link 5 / Link 6 ≈ 60% / 40%, Link elemenat 5 ima veće sile reakcija (skoro duplo veće od L6). Precizniji podaci su prikazani u sumarnoj tabeli Link elemenata.



*Slika 17a. Link 7 i 8, nivo -4,0 m. PGA 0,30g El Centro NDA. Pomeranje: max 6,376*10-5 m, min 6,048*10-5m.*

*Figure 17a. Link 7 and 8, level -4 m. PGA 0.30g El Centro NDA. Displace. max 6.376*10-5m, min 6.048*10-5m.*

Primetna je znatna nesimetrija sila reakcija šipa na dubini 4m od površine terena. Praktično samo link element 7 reaguje i u odnosu na link element 8, to je preko 90% reaktivne sile tokom ukupnog trajanja seizmičkog odgovora na El Centro od 0,30*g*. Pretpostavlja se da je ovakvo ponašanje u sprezi sa prazninom reakcija u link elementima 1 i 2, i asimetrijama intenziteta sila Link elemenata od 3 do 6.



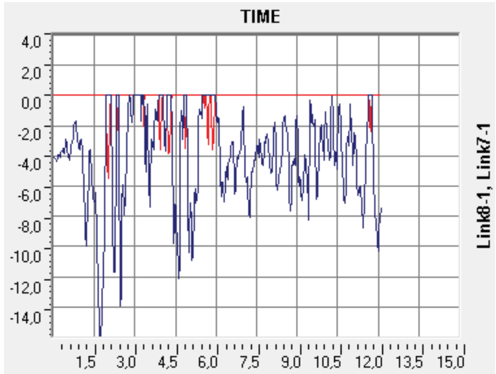
*Slika 18a. Link 9 i 10, nivo -5,0 m. PGA 0,30g El Centro NDA. Pomeranje: max 1,330*10-4m, min. 9,831*10-5m.*

*Figure 18a. Displacement Link 9 and 10, level -5.0 m. PGA 0.30g El Centro NDA. max 1.330*10-4m, min. 9.831*10-5m.*

Raspodela sila reakcija postaje ponovo ujednačena kod Linka 9 i 10 (5 m od nivoa terena). To bi moglo biti mesto uklještenja šipa, kod modela zamenjujuće konzole, (5/0,6=8,3 *D*), s tim da uklještenje može biti i elastično.

Na slici 21b primećuje se prelazni oblik dijagrama sila, u odnosu na više (i niže) nivoe tla od nivoa -7m. Još uvek se uočavaju duži intervali sila reakcija pojedinog link elementa, ali linije nisu više tako glatke kao za gornje slojeve tla, i manje podsećaju na anvelope, a više

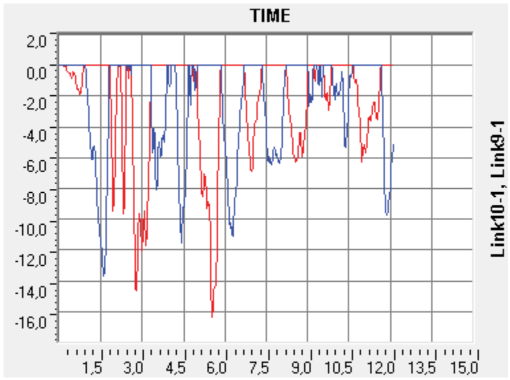
Even though in terms of time, the force distribution is almost uniform: Link 5 / Link 6 ≈ 60% / 40%, the Link element 5 exhibits higher reaction forces (almost twice as that of L6). More accurate data are presented in the summary table of the Link elements.



Slika 17b. Link 7 i 8, nivo -4,0 m. PGA 0,30g El Centro. NDA. Sila max 15,99 kN.

Figure 17b. Links 7 and 8, level -4.0 m. PGA 0.30g El Centro. NDA. Force max 15.99 kN

There is a considerable asymmetry of reaction forces of the pile at a depth of 4m below the surface. Practically, only the link element 7 is reacting, and in relation to the link element 8, it is over 90% of the reactive force during the total duration of the seismic response to El Centro of 0,30*g*. It is assumed that such behaviour is related to the absence of reaction of the link elements 1 and 2, and to force intensity asymmetry of the Link elements 3 to 6.



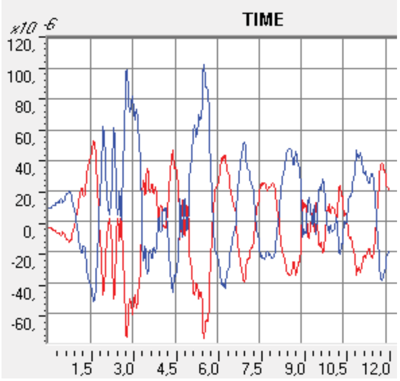
Slika 18b. Link 9 i 10, nivo -5,0 m. PGA 0,30g El Centro. NDA. Sila max 16,38 kN.

Figure 18b. Links 9 and 10, level -5.0 m. PGA 0.30g El Centro. NDA. Force max 16.38 kN

Distribution of reaction forces becomes even again in the Links 9 and 10 (5 m below the ground level). It could be location where the pile is clamped, in the substitute cantilever model (5/0.6=8.3 *D*), provided that the restraint may be elastic as well.

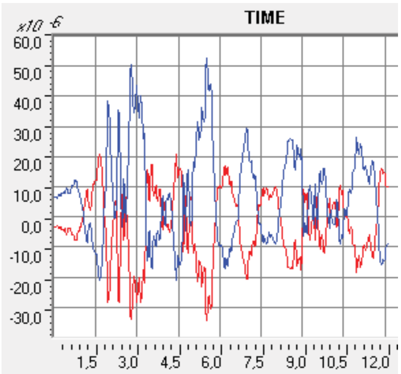
In Fig. 21 one may notice a transition form of the force diagram with respect to higher (and lower) soil levels then the level -7m. The longer intervals of reaction forces of individual link elements can still be observed, but the lines are not as smooth as for the upper layers.

na impulsne strukture (igličastog oblika).



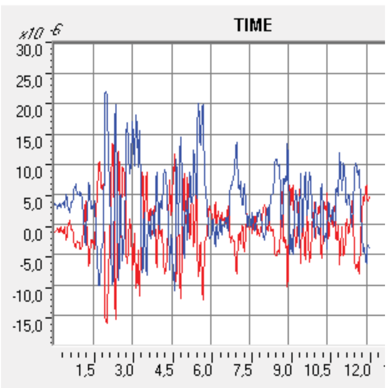
*Slika 19a. Link 11 i 12, nivo -6,0 m. PGA 0,30g El Centro NDA. Pomeranje: max 1,025*10-4 m, min. 7,628*10-5m.*

*Figure 19a. Displacement Link 11 and 12, level -6.0 m. PGA 0.30g ELCentro NDA. max 1.025*10-4m, min. 7.628*10-5m.*



*Slika 20a. Link 13 i 14, nivo -7,0 m. PGA 0,30g El Centro NDA. Pomeranje: max 3,440*10-5 m, min. 5,259*10-5m.*

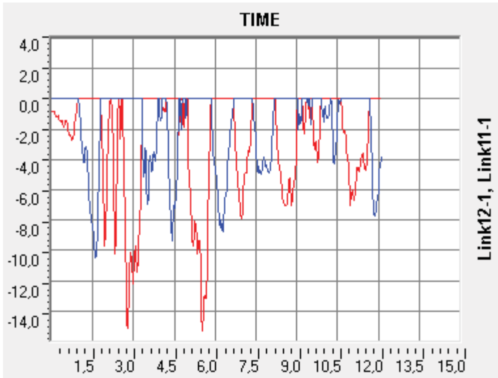
*Figure 20a. Displacement Link 13 and 14, level -7.0 m. PGA 0.30g El Centro NDA. max 3.440*10-5 m, min. 5.259*10-5m.*



Slika 21a. Link 15 i 16, nivo -8,0 m. PGA 0,30g El Centro NDA. Pomeranje: max 2,197x10-5 m, min. 1,640x10-5m.

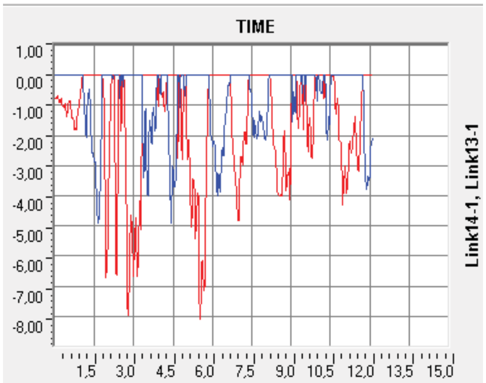
*Figure 21a. Link 15 and 16, level -8.0 m. PGA 0.30g El Centro NDA. Displacement max 2.197*10-5 m, min 1.640*10-5m.*

Also, the lines resemble envelopes less, but rather resemble the impulsive structures (having a pointed form).



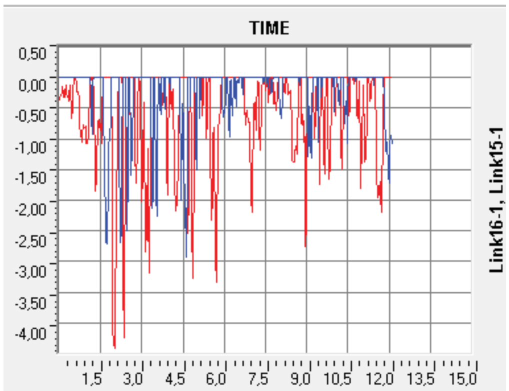
Slika 19b. Link 11 i 12, nivo -6,0 m. PGA 0,30g El Centro. NDA. Sila max 15,32 kN.

Figure 19b. Links 11 and 12, level -6.0 m. PGA 0.30g El Centro. NDA. Force max 15.32 kN



Slika 20b. Link 13 i 14, nivo -7,0 m. PGA 0,30g El Centro. NDA. Sila max 8,086 kN.

Figure 20b. Links 13 and 14, level -7.0 m. PGA 0.30g ELCentro. NDA. Force max 8.086 kN



Slika 21b. Link 15 i 16, nivo -8,0 m. PGA 0,30g El Centro. NDA. Sila max 4,413 kN.

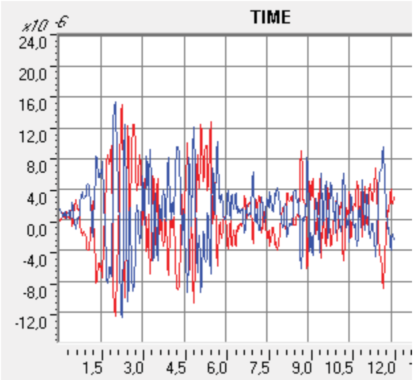
Figure 21b. Links 15 and 16, level -8.0 m. PGA 0.30g El Centro. NDA. Force max 4.413 kN

Dijagram sila reakcija postaje oštar sa naizmenično raspoređenim elementima Link 15 i 16, nivo 8.0 m od površine tla (8/0.6=13.3 *D*).

Dijagram pomeranja Link elemenata 21 i 22 se ne uočava, jer se radi o malim veličinama, ali se mogu jasno očitati numeričke vrednosti, koje su i date u opisu slike 24a. (11/0,6=18,3 *D*).

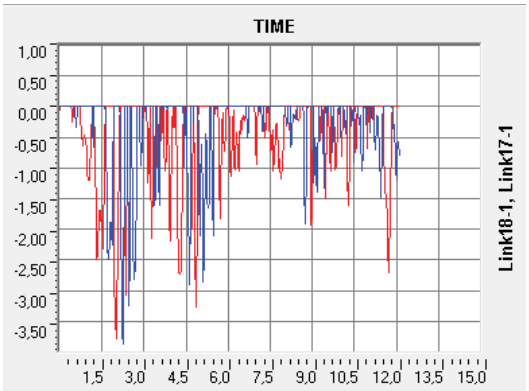
The reaction force diagram becomes pointed, with alternatively arranged elements Links 15 and 16, level 8.0 m below surface (8/0,6=13.3 *D*).

There is no noticeable displacement diagram of Link elements 21 and 22, because those are small values. However, the numerical values may be clearly seen, as they are provided in the description of Figure 24a. (11/0.6=18.3 *D*).



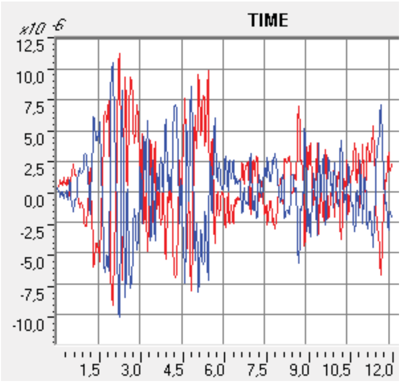
Slika 22a. Link 17 i 18, nivo -9,0 m. 0,30g El Centro. NDA. Pomeranje max 1,536x10-5 m.

Figure 22a. Link 17 and 18, level -9.0 m. 0.30g El Centro. NDA. Displacem. max 1.536x10-5 m



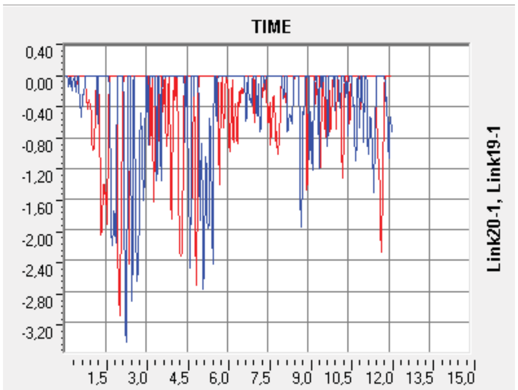
Slika 22b. Link 17 i 18, nivo -9,0 m; 0,30g El Centro. NDA. max Sila 3,881 kN.

Figure 22b. Links 17 and 18, level -9.0 m 0.30g El Centro. NDA. max Force 3.881 kN.



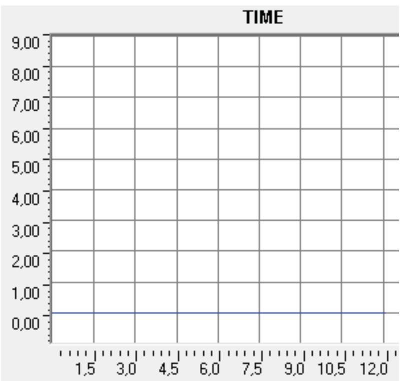
*Slika 23a. Link 19 i 20, nivo -10, m. 0,30g El Centro. NDA. Pomeranje max 1.131*10-5 m.*

*Figure 23a. Link 19 and 20, level -10. m. 0.30g El Centro. NDA. Displace. max 1.131*10-5 m.*



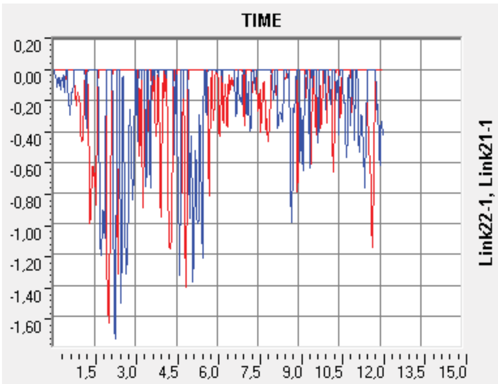
Slika. 23b. Link 19 i 20, nivo -10, m. 0,30g El Centro. NDA. max Sila 3,468 kN.

Figure 23b. Links 19 and 20, level -10.0 m 0.30g El Centro. NDA. max Force 3.468 kN.



*Slika 24a. Link 21 i 22, nivo -11,0 m. 0,30gEl Centro. NDA. Pomeranje max 4,949*10-6 m.*

*Figure 24a. Link 21 and 22, level -11.0 m. 0.30g El Centro. NDA. Displacem. max 4.949*10-6 m.*



Slika. 24b. Link 21 i 22, nivo -11,0 m. 0,30g El Centro. NDA. max Sila 1,747 kN

Figure 24b. Links 21 and 22, level -11.0 m 0.30g El Centro. NDA. max Force 1.747 kN.

Tabela 10. Link elementi po dubini, za levi krajnji šip. Ekstremno pomeranje, ekstremne sile za El Centro 0.30 g
Table 10. Link elements with depth, for left edge pile. Extreme displacement, extr. forces, for El Centro PGA 0.30 g.

Z	Link	U extr	F extr	(Fi*Ui) extr	Σ (Fi*Ui)	(Fsr*ΔUi)extr	Σ ABS(A)	A/ΣA
(m)		(m)	(kN)	(kNm)	(kNm)	(kNm)	(kNm)	%
1	1	0.00272	49.71	0.1352167	6.1454125	0.01424095	0.4188097	59.89
	2	0.00281	46.74	0.0939416	4.3699499	0.00716251	0.2804514	40.11
2	3	0.00138	85.81	0.1184187	6.5062411	0.01073076	0.4461167	66.98
	4	0.00145	62.63	0.0566888	2.8040334	0.00529569	0.2199727	33.02
3	5	0.0004518	44.10	0.0199249	1.2593537	0.00203323	0.0915624	77.69
	6	0.0005053	23.77	0.0057889	0.2804816	0.00062265	0.0262916	22.31
4	7	6.048E-05	15.99	0.0009670	0.0612844	0.00016362	0.0076096	94.26
	8	6.376E-05	5.46	0.0001129	0.0012892	2.4366E-05	0.0004630	5.74
5	9	0.000133	13.65	0.0011192	0.0590964	8.3331E-05	0.0045659	43.80
	10	9.831E-05	16.38	0.0016099	0.0642328	0.00015859	0.0058584	56.20
6	11	0.0001025	10.48	0.0005471	0.0289231	4.3299E-05	0.0025059	32.07
	12	7.628E-05	15.32	0.0011687	0.0609466	0.0001198	0.0053071	67.93
7	13	5.259E-05	4.93	0.0001033	0.0048663	1.8819E-05	0.0006463	24.67
	14	0.0000344	8.09	0.0002782	0.0153709	5.56E-05	0.0019731	75.33
8	15	2.197E-05	2.94	3204E-05	0.0007730	1.2754E-05	0.0002461	29.33
	16	0.0000164	4.41	7.238E-05	0.0021958	2.1798E-05	0.0005931	70.67
9	17	1.536E-05	3.88	4.967E-05	0.0010876	1.6543E-05	0.0003283	47.73
	18	1.496E-05	3.79	4.731E-05	0.0013961	1.0419E-05	0.0003595	52.27
10	19	1.051E-05	3.47	3.565E-05	0.0010162	1.2222E-05	0.0002711	55.46
	20	1.131E-05	3.13	2.896E-05	0.0008312	6.186E-06	0.0002177	44.54
11	21	4.76E-06	1.75	8.22E-06	2.34E-04	2.76E-06	6.98E-05	56.37
	22	4.95E-06	1.64	7.29E-06	1.75E-04	1.54E-06	5.40E-05	43.63

Maksimalna pomeranja link elemenata u prvih tri metara dubine iznose od 2,8 mm do 0,4 mm. Uprkos ovako malim pomeranjima preko 95 % seizmičke energije link elemenata ovog šipa se potroši upravo na toj dubini. To je (3m/0,60m=5*D*) dubina od pet prečnika šipa. To je u skladu sa najvećim uticajima na koeficijente A i B (za granično opterećenje kod pomeranja i sile) kod teorije *p-y* krivih za statičko i ponovljeno opterećenje.

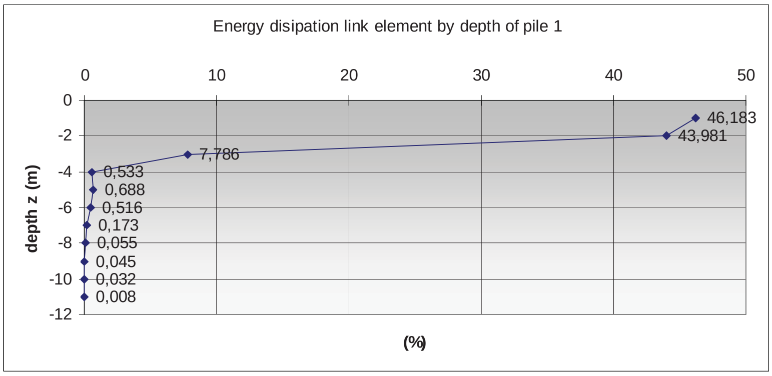
Tabela 11 odnosi se na levi krajnji stojeći šip prečnika D60cm, fundiranog na dubini od 12m, pri čemu su temeljni jastuci debljine 100cm, te je dužina šipa 11m, od donje ivice jastuka do uklještenja u bazi. Krive *p-y* urađene su za svaki metar po dubini šipa, s tim što je uticaj temeljnih jastuka zanemaren. (Napomena: kod izvođenja temelja mašina, neophodno je izvršiti dobro zbijanje tla oko temelja, jer time ovaj uticaj kontakta temelja i tla postaje značajan). Prema tabeli 11, za levi krajnji stojeći šip D60cm, 90% disipacije energije link elemenata tla se obavlja u gornja dva metra dubine (2m/0,60m=3.33), a 99% disipacije energije link elemenata tla, se obavlja u gornjih četiri-pet metara dubine (5m/0,60m=8,33*D*). Ukupan rad link elemenata ovog stojećeg šipa, tokom dejstva zemljotresa El Centro od PGA 0,30 *g*, je relativno mali i iznosi svega 1513 Nm. Iako naizgled mali, ovaj pritisak je dobro raspoređen po dubini tla i veoma značajan za seizmičku otpornost konstrukcije. Slikovito, to bi bio rad koji bi izvršio čovek koji bi čekrkom podigao teret mase 155kg, sa površine zemlje na 1 metar visine.

The maximum displacement of link elements at the first three meters of depth range between 2.8mm and 0.4 mm. In spite of such small displacements, over 95 % of seismic energy of link elements of this pile is dissipated exactly at that depth. It is (3m/0,60m=5*D*) a depth of five diameters of a pile. It is in agreement with the highest effect on coefficients A and B (for limit loads of displacements and forces) of the theory of *p-y* curves for the static and cyclic loads.

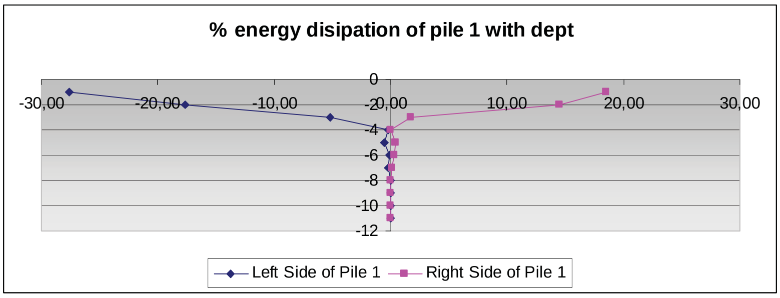
Table 11 refers to the left-end standing pile, with diameter D60cm, founded at a depth of 12m, whereby the foundation cap is 100cm thick, so the pile length is 11m, from the lower side of the cap to the base. The *p-y* curves are calculated for each meter of pile depth, while the influence of the foundation cap is ignored. (Note: when constructing foundations for machinery, it is necessary to compact the soil around the foundations well, because the effect of the foundation and soil contact becomes influential). According to table 11, in case of the left-end standing pile D60cm, 90% of energy dissipation of the link elements of the soil is performed in the top 2 meters of depth (2m/0,60m=3.33). And 99% of energy dissipation of link elements of soil is performed in the top four-five meters of depth (5m/0.60m=8,33*D*). The total work of the link elements of this standing pile, during the action of El Centro earthquake with PGA 0.30 *g*, is relatively small and amounts to mere 1513 Nm. Even though it is seemingly small, this pressure is well distributed along the soil depth, and very important for seismic resistance of the structure. In descriptive terms, it would be the work performed by a man who would lift a 155kg weight using a pulley to a height of 1 meter.

Table 11 Link elementi po dubini za levi krajnji šip. Kumulativni ABS rad uparenih link elemenata (na svaki metar dubine).
Table 11 Link elements with depth for left edge pile. Cumulative ABS work of coupled link elements.

z(m)	Link	cumulative ΣABS Ai (kNm)	%	Σ %
-1	L1+L2	0.6989213980	46.183	46.183
-2	L3+L4	0.6655929120	43.981	90.163
-3	L5+L6	0.1178280390	7.786	97.949
-4	L7+L8	0.0080650920	0.533	98.482
-5	L9+10	0.010409607	0.688	99.170
-6	L11+12	0.007806610	0.516	99.686
-7	L13+14	0.002618564	0.173	99.859
-8	L15+16	0.000839051	0.055	99.914
-9	L17+18	0.000687646	0.045	99.960
-10	L19+20	0.000488583	0.032	99.992
-11	L21+22	0.000123804	0.008	100.000
	Σ	1.513381300		



Slika 25. Procenat (%) disipacije energije za link elemente po dubini za šip 1
Figure 25. Percent (%) energy dissipation of link elements with depth from soil surface for pile 1



Slika 26. Procenat (%) disipacije seizmičke energije, za link elemente, po dubini šipa 1 (za levu i desnu stranu).
Figure 26. Percent (%) seismic energy dissipation, of link elements with depth for pile 1 (left and right sides)

Pretpostavlja se da se seizmički udar dešava u jednoj ravni, ravni 2*D* rama. Tako da se može posmatrati leva i desna strana rama. Ovo nije sasvim tačno, ali može se prihvatiti u postupku postepene analize uticaja u tlu i sistemu tlo-šip.

3.6 Razvoj plastičnih zglobova i prvi svojstveni ton (El Centro sa PGA 0,20; 0,25; 0,30g)

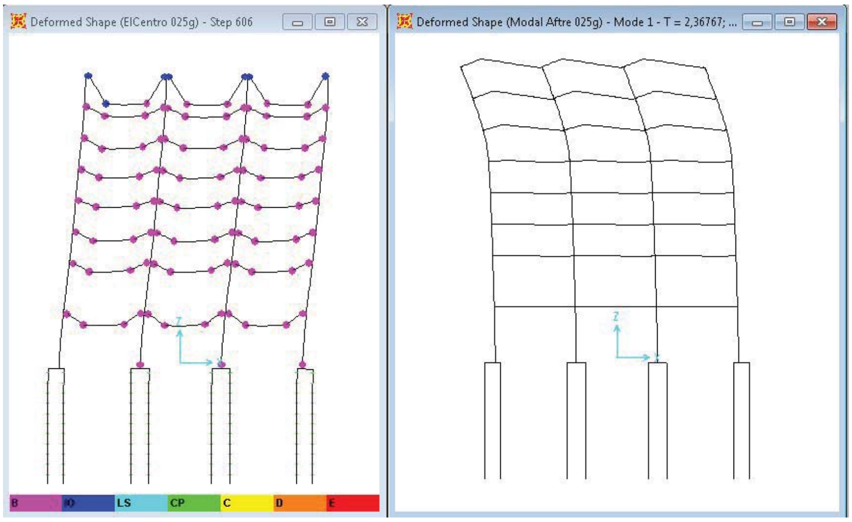
Proučena je i promena stanja plastičnih zglobova - state of plastic hinge (SPH) i usled promene statičkog sistema promenu prvog svojstvenog oblika, 2*D* rama fundiranog na šipovima, sa porastom PGA.

It is assumed that the seismic impact occurs in one plane, the 2*D* frame plane. Thus, the left and the right sides of the frame may be distinguished. This is not entirely true, but it can be accepted in the analysis procedure of the effects in the soil and the soil-pile system.

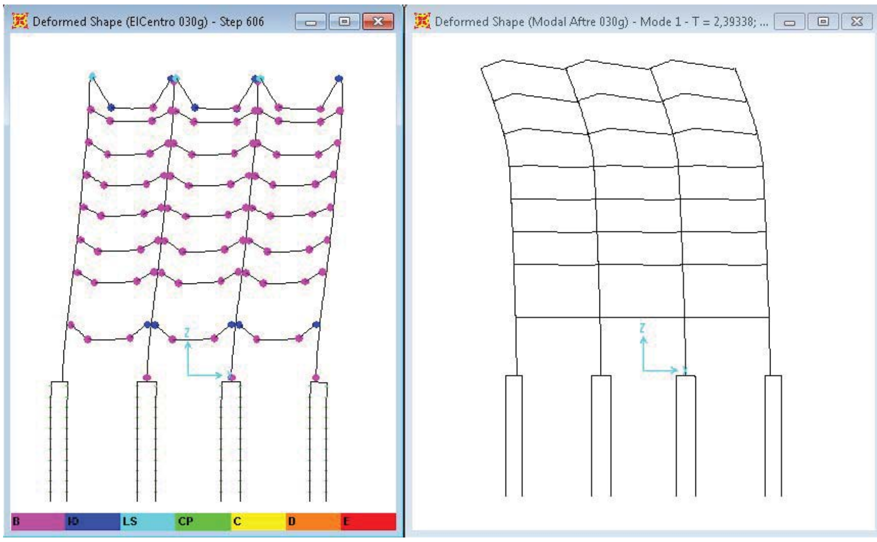
3.6 Development of plastic hinges and the first natural mode (El Centro of PGA 0.20; 0.25; 0.30*g*)

The variation and change of condition of plastic hinges and the first natural mode of a 2*D* frame founded on piles, with the increase of PGA is studied.

Slika 27. El Centor 0,20g stanje na kraju zemljotresa. Levo) plast. zglobovi: 90 Y + 6 IO; desno) oblik 1 vibracija
Figure 27. El Centro 0.20 g. State at the end of an earthqake. Left, Pl Hinge state: 90 Y + 6 IO. Right, Mode 1.



Slika 28. El Centor 0,25g stanje na kraju zemljotresa. Levo) plast. zglobovi: : 92 Y +7 IO; desno) oblik 1 vibracija
Figure 28. El Centro 0.25 g. State at the end of an earthqake. Pl Hinge state: 92 Y +7 IO. Right, Mode 1.



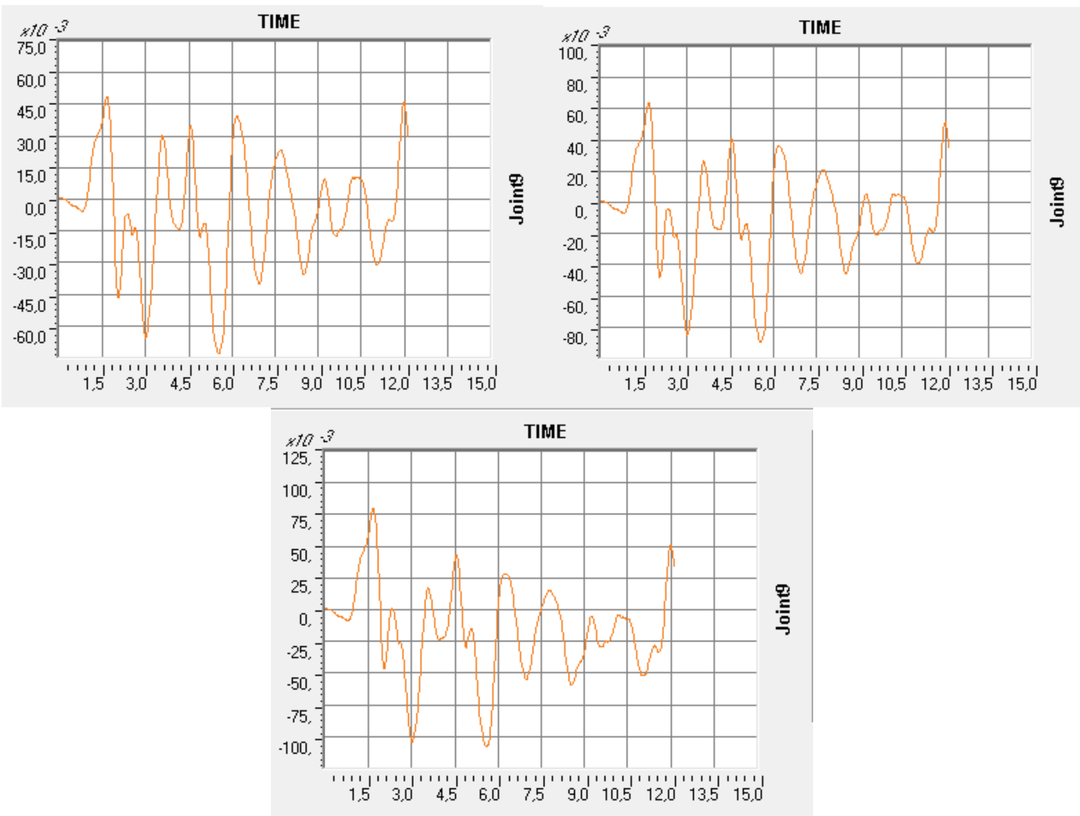
Slika 29. El Centor 0,30g stanje na kraju zemljotresa. Levo) plast. Zglobovi: 86 Y +10 IO+3 LS; desno) oblik 1 vibracija
Figure 29. El Centro 0.30 g. State at the end of an earthqake. Pl Hinge state: 86 Y +10 IO+3 LS. Right, Mode 1.

U tabeli 12 prikazana je promena prvog i drugog svojstvenog tona, nakon dejstva akcelerograma El Centro, od 0,20; 0,25 i 0,30 *g* i odgovarajuće promene statičkog sistema zbog pojave plastičnih zglobova.

Table 12 is presenting variation of the first and the second natural modes, after action of accelerogram El Centro, of 0.20; 0.25 and 0.30 g and the corresponding change of the statical system due to appearance of plastic hinges.

Tabela 12. Prva dva svojstvena perioda posle El Centra različitih PGA. 2D ram.
Table 12. The first two natural periods after El Centro with different PGA. 2D Frame.

PGA (*g*)	T_1 (sec)	T_2 (sec)	T_1 %	T_2 %
start	1.37255	0.44269	0	0
0.20	1.73011	0.86837	26.05	96.16
0.25	2.36767	1.00557	72.50	127.15
0.30	2.39338	1.03398	74.37	133.57



Slika 30. Pomeranje čvora 9, u vrhu rama, tokom dejstva zemljotresa El Centro, za PGA 0.20; 0.25 i 0.30g. Gore levo za 0,20 g, gore desno 0,25 g i dole 0,30 g

Figure 30. Displacement of node 9, at the top of the frame, during action of earthquake El Centro, for PGA 0.20; 0.25 and 0.30g. Upper left for 0.20 g, upper right 0.25 g and down 0.30 g

Table. 13. Pomeranje čvora u vrhu stuba, za različito PGA, za nelinearnu i linearnu krovnu gredu.
Table. 13. Displacement of the node at the column top, for different PGA, for nonlinear and linear roof girder.

PGA (*g*)	min *U1* Joint 9	max *U1* Joint 9	extr *U1* Joint 9	Lin. Roof Beam	L/NL RB%
0.20	-0.0731	0.0485	0.0731	0.0856	117.10
0.25	-0.0894	0.0636	0.0894	0.1129	126.29
0.30	-0.1078	0.0802	0.1078	0.1447	134.23

Ekstremno pomeranje čvora u vrhu je kod linearizovane krovne grede veće kod PGA 0,20*g* za 17%, kod PGA 0,25*g* za 26%, i kod PGA 0,30*g*, za 34%.

Od interesa je i prikaz uticaja uvođenja plastičnih

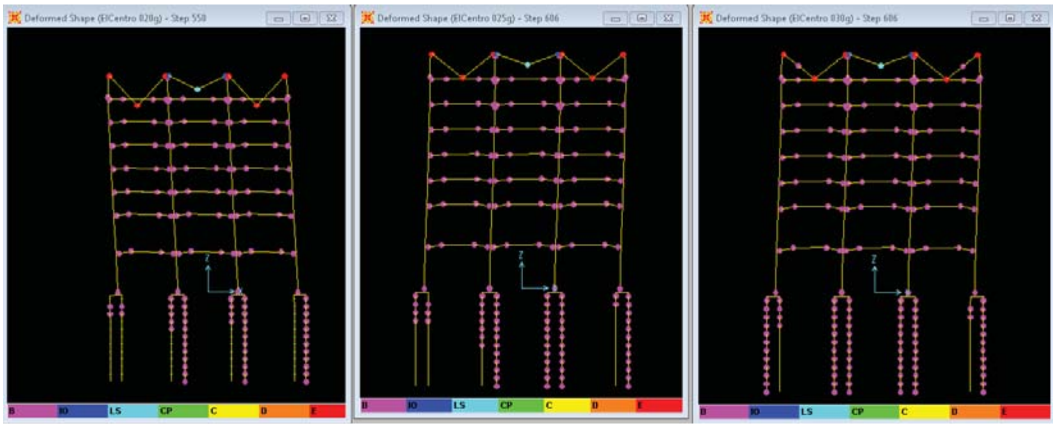
Extreme displacement of node at the top in the case of linearized roof beam is 17% higher for PGA 0,20*g*, 26% for PGA 0,25*g* and 34%for PGA 0,30*g*.

It is also of interest to present the effect of zglobova u sredinama raspona greda, za različito PGA i raspored ostalih plastičnih zglobova, uključivo i one koji se javljaju u šipovima.

Na slici 31, sistem postaje senzitivan na uvođenje plastičnih zglobova u sredinama raspona greda. Krovne grede doživljavaju kolaps, za sva 3 PGA (0.20; 0.25; 0.30*g*). Ostale grede i stubovi se povoljnije ponašaju, međutim, to je na račun šipova, jer se plastični zglobovi sele u šipove, već pri PGA 0.20*g*. Svi ostali plastični zglobovi osim krovnih ulaze u stanje B, tj. početak tečenja *Y(yield)*. Slična pojava se dešava [4] kod srednjeg rama mosta, ali to je logična pojava ispitivanja promene krutosti tla. Naime u [4], varirane su krutosti linearnih opruga tla na šipovima, kada je često uočena pojava seljenja plastičnih zglobova u šipove, tokom smanjenja krutosti tla. Ova pojava ne događa se uvek, a zavisi i od prvog (ponekad i drugog) sopstvenog tona konstrukcije, i spektra odgovora primenjenog akcelerograma. Dubinu pojave plastičnih zglobova, kod srednjeg rama mosta, za nekoliko vrsta tla, proučio je [21]. U [14] navodi se podatak, da se rezultati *p-y* krivih u nekim slučajevima mogu razlikovati i nekoliko puta.

introduction of plastic hinges at the mid-span of beams, for different PGA and the arrangement of other plastic hinges, including those occurring in the piles.

In Figure 31, the system becomes sensitive to introduction of plastic hinges at the mid-spans of the beams. The roof beams collapse, in case of all 3 PGA (0,20; 0,25; 0,30*g*). The other beams and columns behave in a more favourable way; however, this comes at the expanse of the piles, because the plastic hinges migrate to the piles, as early as at PGA 0,20*g*. All other plastic hinges, except the roof ones, acquire the B state, which is the onset of yield *Y(yield)*. A similar phenomenon took place in [4] at the middle frame of a bridge, but it is a logical consequence of testing the variation of soil rigidity. Namely in [4], the stiffness of linear springs of the soil on piles was varied, and the phenomenon of migration of plastic hinges into piles was often observed, during the reduction of soil density. This phenomenon does not occur always, and it depends on the first (and sometimes on the second) natural mode of the structure, and the response spectrum of the applied accelerogram. The depth of the onset of plastic hinges, for the middle frame of a bridge, for several types of soil was studied in [21]. In [14] it was mentioned, that the results of *p-y* curves in some cases can be different several times.



Slika 31. Raspodela plastičnih zglobova za različito PGA pri pojavi plastičnih zglobova u sredini raspona krovnih greda

Figure 31. Plasitic hinge in midle of beam span, for different PGA, and distribution of plastic hinges

Ovde kod zgrada, krutost tla nije varirana. Naime, korišćena je samo jedna vrsta tla (krut pesak, potopljen), uvek ista dužina šipa i uslovi uklještenja u bazi. Korišćena je samo jedna vrsta akcelerograma, a to je El Centro, samo horizontalna komponenta, za PGA 0.20; 0.25 i 0.30*g*. Ova pojava kod zgrada zahteva dalja istraživanja. Između ostalog, precizniju primenu modela datih u [2]. Za određene vrste tla, akcelerograme, vršna ubrzanja i karakteristike šipova, mogu se izvući rezultati, kojima se umesto p-y krivih modeluje sekantna krutost opruga tla [16].

For the buildings here, the soil stiffness is not varied. Namely, only one type of soil is used (dense sand, submerged), always the same length of piles and clamped conditions at the base. Only one type of accelerogram is used: it is El Centro, only the horizontal component, for PGA 0.20; 0.25 and 0,30*g*. This phenomenon related to buildings requires further research. Among other things, a more accurate use of models provided in [2]. For certain types of soils, accelerograms, peak accelerations and pile characteristics, results may be determined, where one could model the secant stiffness of the soil springs instead of using the *p-y* curves [16].

4 ZAKLJUČAK

Tlo ispod temelja često se u seizmičkim analizama apstrahuje, a konstrukcija smatra uklještenom u temelje. Međutim, kod visokih zgrada, mostova većih raspona i

4 CONCLUSIONS

Soil beneath the foundations is often ignored in seismic analyses, and structures are considered as clamped in the foundations. However, tall buildings, nekim inženjerskim konstrukcijama treba u seizmičkoj analizi uključiti i interakciju konstrukcija-temelj-tlo. Uvođenje analize na 3D modelima je veoma kompleksno, pa je u ovom radu pokazano da se zamenom prostorne skeletne konstrukcije zgrade 2D ramom problem znatno pojednostavljuje.

Kod određivanja seizmičkih performansi konstrukcije korišćenjem NSA (pušover analize) bitno je odrediti tačku kada konstrukcija prelazi u mehanizam. Promenu broja i stanja plastičnih zglobova sa porastom pomeranja, u koracima NSA kod određivanja PO krivih u programu SAP2000 v14, nije lako direktno utvrditi. Bolji prikaz PO krivih za zgrade ima program ETABS, mada se i kod programskog paketa SAP2000, mogu dobiti dobri prikazi naročito kod inženjerskih objekata, ali treba voditi računa o alternativnim procedurama. Analiza korišćenjem metode N2 PO se primenjuje za određivanje ciljnog pomeranje konstrukcije, kao tačka preseka seizmičkog zahteva (preko spektra odgovora) i seizmičkog kapaciteta konstrukcije. Prikazan postupak relativno pojednostavljene procedure za određivanje uticaja NSA i NDA dinamičke interakcije tlo-šip-konstrukcija. Radi dobijanja sveobuhvatnije slike performansi konstrukcije osim više različitih modela, sa i bez interakcije, neophodno je primeniti više različitih metoda, oblika opterećenja, više različitih vrsta i skaliranja akcelerograma, zatim procedura i programskih paketa.

Numeričkim istraživanjima uticaja u tlu, utvrđeno je da su sile reakcija link elementa male veličine u odnosu na ukupnu seizmičku silu u osnovi. Iako su intenziteti reakcija link elemenata, tokom dejstva zemljotresa, u odnosu na vrednost sila u osnovi, relativno male veličine, one su veoma značajne za ukupnu seizmičku otpornost objekta. Uočene su određene zakonitosti promene dijagrama sila po dubini link elemenata, ali je iste neophodno tumačiti na dijagramima, što je u ovom radu urađeno.

Analizom seizmičkog ponašanja pri ulaznim podacima (akcelerogrami za PGA 0.20; 0.25 i 0.30*g*) razmatrani sistem je veoma osetljiv na uvođenje plastičnih zglobova u sredinama raspona greda. U krovnim gredama došlo je do loma, za sva 3 PGA (0.20; 0.25; 0.30*g*), ostale grede i stubovi se povoljnije ponašaju jer se plastični zglobovi „sele“ u šipove, već pri PGA 0.20g. Zbog toga je veoma važno pri projektovanju AB ramovskih konstrukcija adekvatnim dimenzionisanjem i detaljima izbeći formiranje plastičnih zglobova u poljima greda.

Može se zaključiti da se uvođenjem SSI postiže pozitivan efekat naročito ako se radi o krućim konstrukcijama zgrada, da bi se izbegle veće deformacije tavanica i potencijalni sudar sa susednim objektima u gušćim urbanim sredinama, što je potvrđeno u [1] i [9].

Naredna istraživanja potrebno je proširiti, na sve šipove rama, i za *p-y* krive za različite relativne zbijenosti peska. Takođe je potrebno uvesti i vertikalnu interakciju sa tlom, koja je u ovom radu zanemarena. Takođe je, kod pušover krivih potrebno utvrditi da li postoji jaka zakonitost oblika vertikalnog opterećenja od gornje konstrukcije, sa oblikom odziva *p-y* krivih po dubini šipova (oblik odziva pomeranja čvorova i drift šipova). Ovaj odziv može se posmatrati i kod TH analize, a da li postoji jasna zakonitost to tek treba utvrditi.

large-span bridges and some engineering structures require inclusion of the structure-foundation-soil interaction. Introduction of the analysis based on 3D models is very complex, so in this paper it is shown that by replacing the spatial frame structure of a building, with a 2D frame, the problem is considerably simplified.

When determining the seismic performance of a structure using the NSA (pushover analysis), it is important to determine the point at which the structure becomes a mechanism. The change in the number and states of plastic hinges resulting from the increase in displacements, (in steps) of PO curves using SAP2000 v14 software cannot be easily determined. ETABS software has better displays of PO curves, although this can be achieved in SAP2000 as well (especially in the case of engineering structures) if alternative procedures are taken into account. A PO analysis is applied within the N2 method in order to determine the target structure displacement, as an intersection point of the seismic requirements (through spectrum response) and of the seismic capacity of structures. The presented relatively simplified procedure for determining the effects of NSA and dynamic NDA soil-pile-structure interaction is provided in this paper. In order to obtain a more comprehensive insight about the structure's performance, it is necessary to apply several different models, load shapes, types and scales of accelerograms, procedures and software packages, with and without interactions.

Numerical research of effects in the soil, determined that reaction forces of link elements are small in relation to the total base force. Even though the intensities of link elements reactions during earthquakes are relatively small in comparison to the value of base forces, they are very important for the total seismic resistance of the structure. Certain regularities in the variation of the force diagram, along the depth of the link elements are observed, but they need to be interpreted on the diagrams, which has been done in this paper.

The analysis of the seismic behaviour using the input data (accelerograms with PGA 0.20; 0.25 and 0.30*g*) showed that the considered system is very sensitive at early formation (introduction) of plastic hinges at mid-spans of the beams. There was a failure of the roof beams, for all 3 PGA (0.20; 0.25; 0.30*g*), while the remaining beams and columns behave more favourably, because the plastic hinges migrate to piles, as early as at PGA 0,20*g*. For that reason, it is very important to avoid formation of plastic hinges in the beam spans using adequate design and details of RC frame structures.

It may be concluded that by introduction of SSI a positive effect could be achieved especially if stiff building structures are in question, in order to avoid severe ceiling deformations and potential collision with adjacent structures in densely populated urban environments, which is confirmed in [1] and [9] as well.

The following research must be extended to all the frame piles, and to *p-y* curves for different relative sand densities. It is also necessary to introduce a vertical interaction with the soil which is ignored in this paper. Also, in pushover curves, it is necessary to determine whether there is a strong regularity of the vertical load upon the superstructure, with the form of response of *p-y* curves, along the depth of the piles (shape of the nodal displacement response and the pile drift). This response may be analysed through the TH analysis as well, and it still needs to be determined if there is a clear regularity.

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5 LITERATURA
REFERENCES

1. Carbonari, S., Dezi, E., Graziano, L. (2012): Nonlinear seismic behaviour of wall-frame dual systems accounting for soil-structure interaction, Earthquake Eng. and Structural Dynamic, **41,** pp. 1651-1672. DOI: 10.1002/eqe.1195.
2. Čaušević M.: Dinamika konstrukcija. Golden Marketing. Tehnička knjiga. Zagreb, 2010.
3. Ćosić, M., Folić, R., Brčić, S. (2017): An overview of modern seismic analyses with different ways of damping introduction (Pregled savremenih seizmičkih analiza i načina uvođenja prigušenje u njima), Building Materials and Structures (Građevinski materijali i konstrukcije) (**60**), br.1, pp. 3 – 30.
4. Folić B.: Seizmička analiza betonskih konstrukcija fundiranih na šipovima. Doktorska disertacija. FTN. Univerzitet u Novom Sadu. Novi Sad 2017.
5. Folić, B., Folić, R. (2009): Design methods analysis of seismic interaction soil-foundation-bridge structures for different foundations, in: Coupled Site and Soil-Structure Interaction Effects with Application to Seismic Risk Mitigation, Ed. T. Schanz and R. Jankov, Springer Sciences+ Business Media, pp. 179-191.
6. Folić B., Ladjinović Đ., Sedmak S., Liolios A.:Comparative nonlinear analysis soil-pile interaction 2D frame. 7th international conference: “Geotehnicis in Civil Engineering”, ACE of Serbia, Proc. Ed. R. Folić, Šabac, November 14-17. 2017. pp. 473-484.
7. Folić, R.**,** Liolios, A.: Application inclined piles in seismic prone area, useful or not? 7th international conference: “Geotehnicis in Civil Engineering”, ACE of Serbia, Proc. Ed. R. Folić, Šabac, November 14-17. 2017. pp. 461-472.
8. Forootan, F., Moghadam, A.S. (2006): Comparison of 2D and 3D pushover analysis with time history analysis in asymertic building, First European Conf. on Earthq. Eng. and Seismology, Geneva, 3-8 September, Paper Number: 447
9. Jawad Arefi, M. (2008): Effects of soil-structure interaction of the seismic response of existing RC frame buildings, A dissert. of master degree, Universita degli studi di Pavia, Italy
10. Kraus, I., Džakić, D. (2013): Soil-Structure interaction effects on seismic behaviour of RC frames, 5th Intern. Conf. CE EEE 1963. IZIS, Skopje, p. 8.
11. MacLeod I.A.(1990): Analytical modelling of structural system. Ellis Horwood limited. London.
12. Mayer, B.J., Reese,L.C. (1979): Analysis of single piles under lateral loading, Res. St. 3-5-78-244, Texas Sdof Highways PT
13. Maymond P. J. (1998): Shaking table scale model test of nonlinear soli-pile –superstructure interaction in soft clay, University of California, Berkley. Ph.D.
14. Milović, D., Đogo. M.: Problemi interakcije tlo-temelj-konstrukcija. Srpska akademija nauka i umetnosti ogranak u Novom Sadu. Novi Sad 2009.
15. Mosher R., Dawkins W.: Theoretical Manual for Pile Foundations, U.S. Army Corps of Engineers, Report ERDC/ITL TR-00-5, Washington, USA, 2000.
16. Pando, M. (2013): Analyses of Lateral Loaded Piles with *p-y* Curves - Observations on the Effect of Pile Flexural Stiffness and Cyclic Loading. NCDOT 7thGeo3T2, Raleigh, NC, Thursday, April 04, 2013. Paper: 3B-1_A49
17. Petronijević, M.(1993): Analiza dinamičkog sadejstva tla i objekta primenom metode konačnih elemenata. Doktorska disertacija. Građevinski fakultet, Univerzitet u Beogradu.
18. Prakash, S. Ed, (1992): Soil under Dynamic Loads. Geotechnical Special Publication. Geotechnical Engineering Division of ASCE. 1992. No. 34
19. Reese L., Van Impe W. (2001): Single pile and pile groups under lateral loading, Balkema, Rotterdam, 2001
20. Stewart,J.P., Fenves, G.L., Seed, R.B. (1999): Seismic soil-structure interaction in buildings.1: Analytical methods, Journal Environm. Engineering, ASCE, V. 125:1, pp. 26-37
21. Suarez, V.(2005): Implementation of Direct Displacement Based Design for Pile and Drilled Shaft Bents. NCSU. North Caroliona State University. October. 2005.
22. Tabatabaiefara,H.R., Massumi, A. (2010): A simplified method to determine seismic response of reinforced concrete moment resisting building frames under influence of soil-structure interaction, Soil Dynamics and Earthquake Eng. **30, 11,** pp. 1250-1267.

REZIME

KOMPARATIVNA NELINEARNA ANALIZA INTERAKCIJE ŠIP-TLO AB 2D RAMA

Boris FOLIĆ
Radomir FOLIĆ

U radu je sprovedena komparativna nelinearna statička (NSA) i nelinearna dinamička analiza (NDA) seizmičkog ponašanja rama kao dela skeletne konstrukcije AB zgrade fundirane na šipovima. Da bi se dobila realnija slika ponašanja ramovske konstrukcije u analizu je uključena interakcija konstrukcija – temelj – tlo. Pri tome u proračunski model je uključena i linearno-nelinearna dinamička interakcija šip-tlo korišćenjem link elemenata.

Konstrukcija temelja sastoji se od bušenih šipove prečnika 60cm. Tlo je modelovano sa više (linijskih) plastičnih veznih elemenata, kao p-y krivama, sa obe strane šipa, za potopljen krut pesak, i uz pretpostavku da *p-y* krive (eksperimentalno određene nelinearne krive zavisnosti: pomeranje/pritisak, u tlu po dubini šipa) primaju samo pritisak. Analizom je ukazano na probleme, koje prate izdvajanje 2D ramova kao reprezenta regularne prostorne 3D konstrukcije. Proučen je uticaj pojave i lokacije pojedinih plastičnih zglobova na seizmičke performanse analiziranog konstruktivnog sistema, i analizirana relativna spratna pomeranja (driftovi). Zaključeno je da se analizom 2D rama u interakciji sa temeljom i tlom, mogu dobiti dovoljno tačni rezultati ponašanja i ocene seizmičkih performansi skeletne AB višespratne zgrade. To je značajno jer uvođenje prostorne konstrukcije u ovakve analize je veoma kompleksno i zahtevno.

Ključne reči: Dinamička interakcija tlo-šip, nelinearna dinamička analiza (NDA), nelinearna statička (puš-over) analiza (NSA), Interakcija tlo-konstrukcija (SSI), višelinijski plastični link element MPLE, *p-y* krive, raspodela uticaja po dubini tla link elemenata

SUMMARY

COMPARATIVE NONLINEAR ANALYSIS OF A RC 2D FRAME SOIL-PILE INTERACTION

Boris FOLIC
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Comparative non-linear static (NSA) and non-linear dynamic analyses (NDA) of 2D frames (as parts of skeletal 3D structures) of RC buildings founded on piles are presented in this paper. In order to produce a more realistic presentation of behaviour of a frame structure, the analysis involves a structure-foundation-soil interaction. Also, the model involves a linear-non-linear dynamic pile-soil interaction, using link elements. The foundation consists of drilled piles having 60 cm in diameter. The soil is modelled using Multi-linear plastic link elements, as well as with p-y curves, on both sides of the pile, assuming that *p-y* curves transfer only compression (p-y curves are experimentally determined non-linear relationships of displacement/pressure in soil, along the depth of a pile). The analysis shows the problems which accompany extraction of a 2D frame, as a representative of a regular 3D space frame. The impact of onset and location of individual plastic hinges on seismic performances of the analyzed structural system are investigated, and relative floor drifts are analyzed. It was concluded that the analysis of 2D frame, in the interaction with the foundation and soil, may provide sufficiently accurate results of behaviour and assessments of seismic performances of skeletal RC multi-storey building. It is important, because introduction of a spatial structure in such analyses is very complex and challenging.

Key words: Dynamic soil-pile interaction (DSPI), non-linear dynamic analysis (NDA), non-linear static (pushover) analysis (NSA), soil-structure interaction (SSI), multiline plastic link elements (MPLE), *p-y* curves, after-shock, distribution of influence with depth of soil

VALIDACIJA I IMPLEMENTACIJA *HASP* KONSTITUTIVNOG MODELA ZA PREKONSOLIDOVANE GLINE

VALIDATION AND IMPLEMENTATION OF *HASP* CONSTITUTIVE MODEL FOR OVERCONSOLIDATED CLAYS

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1 UVOD

Značajan deo u oblasti konstitutivnog modeliranja tla predstavlja opisivanje naponsko-deformacijskih relacija prekonsolidovanih glina. Prekonsolidovane gline su u prošlosti bile opterećene vertikalnim efektivnim naponom koji je veći od tekuće veličine vertikalnog efektivnog napona. Prekonsolidacija može biti i posledica izvođenja različitih građevinskih radova na tlu i u tlu. U poređenju s normalno konsolidovanim glinama, imaju manji koeficijent poroznosti i veću smičuću čvrstoću. U prirodi su najčešće ispucale, što dovodi do nehomogenog polja deformacija. Iz tog razloga, ispoljavaju složen oblik ponašanja pri lomu.

Veliki broj konstitutivnih modela za prekonsolidovane gline razvijen je koristeći koncept kritičnog stanja [35, 38] i Modifikovani Cam Clay (MCC) model [36]. MCC model se može, pri monotonom opterećenju, koristiti s velikom pouzdanošću za normalno konsolidovane i lako prekonsolidovane gline. Za jako prekonsolidovane gline, MCC model precenjuje smičući napon pri lomu i predviđa nagli prelaz iz elastične oblasti u elasto-plastičnu oblast, što nije u skladu sa eksperimentalnim podacima koji pokazuju postepeno smanjenje krutosti prilikom opterećivanja.

Za prevazilaženje nedostataka MCC modela, korišćeni su različiti koncepti. Zienkiewicz i Naylor [52] u relacije konstitutivnog modela uveli su matematički opis površi Hvorsleva, što su u svojim modifikacijama sledili i

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1 INTRODUCTION

A significant part in the area of constitutive soil modelling is the description of the stress-strain relationships of overconsolidated clays. In the past, overconsolidated clays were exposed to the vertical effective stress that is greater than the current magnitude of vertical effective stress. Overconsolidation can also be a consequence of carrying out various construction works on the soil and in the soil. Compared to the normally consolidated clays, they have a lower void ratio and higher shear strength. In nature, they are mostly cracked, leading to a nonhomogeneous field of strains. For this reason, they exhibit a complex form of shear failure.

A large number of constitutive models for overconsolidated clays has been developed using the critical state concept [35, 38] and Modified Cam Clay (MCC) model [36]. The MCC model can be used for normally consolidated and lightly overconsolidated clays under monotonic load, with great certainty. For heavily overconsolidated clays, the MCC model overestimates the failure shear stress and predicts a sudden transition from elastic to elastic-plastic region, which is not in accordance with experimental data that indicate a gradual stiffness reduction during loading.

Different concepts were used to overcome the deficiencies of the MCC model. Zienkiewicz and Naylor [52] have incorporated the mathematical description of

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drugi autori [15, 25, 50, 46, 37]. Na taj način se modifikuje granica mogućih naponskih stanja iznad linije kritičnog stanja i realnije opisuje veličina smičućeg napona pri lomu u dreniranim i nedreniranim uslovima.

Pored toga, razvijen je koncept s više površi tečenja – *Multi Surface Plasticity* – MSP [16, 26], koji adekvatnije opisuje zakon ojačanja materijala, postepen prelaz iz elastične u plastičnu oblast, ponašanje prekonsolidovanog tla, kao i ponašanje tla pri cikličnom opterećenju. Predstavljao je generalni okvir u kome su razvijeni mnogi konstitutivni modeli. Koncept granične površi – *Bounding Surface Plasticity* – BSP [8, 9, 22] je zasnovan na MSP konceptu i predstavljao je poboljšanje u opisivanju postepenog prelaza iz elastične oblasti u elasto-plastičnu oblast. Osnovna ideja je da se – umesto klasične površi tečenja kod Cam Clay modela koja ograničava elastični region – definiše granična površ unutar koje je dozvoljen razvoj plastične deformacije. Prednost ovog koncepta jeste uzimanje u obzir prethodne istorije opterećivanja. Takođe, omogućena je simulacija ponašanja tla pod cikličnim opterećenjem, jer površ popuštanja koja ograničava elastični region može da se translatorno pomera unutar granične površi. Brojni konstitutivni modeli za prekonsolidovano tlo zasnovani su na MSP ili BSP konceptima: Bubble model [2], MIT-E3 [48], 3 SKH model [45], Two Kinematic Hardening Constitutive Models [12], Modified 3 SKH model [24], SANICLAY model [10], UH-model [50]. Navedeni modeli, u matematičkom smislu, složeniji su od MCC modela i imaju veći broj materijalnih parametara. Matematička složenost zahteva napredne numeričke metode i odgovarajući softver, što u današnje vreme ne predstavlja veliki problem, jer su takvi komercijalni softveri dostupni inženjerima u praksi. Znatno veći problem za primenu ovih modela u praksi jeste to što se dodatni materijalni parametri uglavnom ne mogu dobiti iz standardnih laboratorijskih opita. Upravo zahvaljujući jednostavnosti i lakoj identifikaciji parametara modela, MCC model se još uvek najčešće koristi u analizi geotehničkih problema, iako predviđanja naponsko-deformacijskih relacija ne odgovaraju realnom ponašanju prekonsolidovanih glina. Jedan od načina da se unapredi konstitutivni model, a da se ne povećava broj materijalnih parametara, jeste da se koriste unutrašnje promenljive koje adekvatno definišu stanje tla – kao bitnu odrednicu njegovog mehaničkog ponašanja. Jedna od takvih promenljivih je parametar stanja (*state parameter*) koji se još uvek ne koristi dovoljno u konstitutivnom modeliranju.

2 KONCEPT PARAMETRA STANJA

Koncept parametra stanja prvi su predstavili Been i Jefferies [4] za opisivanje ponašanja peska. Umesto koeficijenta poroznosti koji se koristio kao bitna karakteristika za ponašanje peska, predloženo je korišćenje parametra stanja kao fundamentalne promenljive. Veličina srednjeg normalnog efektivnog napona p' značajno utiče na ponašanje tla, tako da se krupnozrno tlo za dati koeficijent poroznosti pri velikoj vrednosti srednjeg efektivnog napona ponaša kao rastresito, dok se za manje vrednosti srednjeg efektivnog napona ponaša kao zbijeno. To znači da je za karakterizaciju krupnozrnog tla – pored koeficijenta poroznosti – neophodna i veličina srednjeg efektivnog

the Hvorslev surface, which was followed by other authors in their modifications [15, 25, 50, 46, 37]. That imposes a more realistic limit to possible stress states above the critical state line and gives a more realistic description of peak shear stress value in drained and undrained conditions.

In addition, the concept of Multi Surface Plasticity – MSP [16, 26] has been developed, which describes more specifically the hardening rule, a gradual transition from elastic to elastic-plastic region, mechanical behaviour of overconsolidated soil, as well as soil behaviour at cyclic loads. It was a general framework in which many constitutive models were developed. The boundary surface concept – Bounding Surface Plasticity – BSP [8, 9, 22] is based on the MSP concept and has been an improvement in describing the gradual transition from elastic to elastic-plastic region. The basic idea is to define, instead of the classic Cam Clay yield surface that limits the elastic region, the boundary surface within which development of plastic strain is allowed. The advantage of this concept is taking into account previous history of loads. Also, simulation of the soil behaviour under a cyclic load is made possible, since the yield surface that limits the elastic region can be moved within the boundary surface. Numerous constitutive models for overconsolidated soil are based on MSP or BSP concepts: Bubble model [2], MIT-E3 [48], 3 SKH model [45], Two Kinematic Hardening Constitutive model [12], Modified 3 SKH model [24], SANICLAY model [10], UH-model [50]. These models are mathematically more complex than the MCC model and have a greater number of material parameters. The mathematical complexity requires advanced numerical methods and appropriate software, which is not a problem because such commercial software is available to engineers in practice. Much greater problem for practical application of these models is that additional material parameters mostly cannot be obtained from standard laboratory tests. Due to the simplicity and easy identification of model parameters, the MCC model is still most often used in analysis of geotechnical problems, although the prediction of stress-strain relations do not correspond to the real behaviour of overconsolidated clays. One way to improve the constitutive model, without increasing the number of material parameters, is to use internal variables that adequately define soil state as an essential determinant of its mechanical behaviour. One such variable is state parameter, which is used insufficiently in constitutive modelling.

2 STATE PARAMETER CONCEPT

The state parameter concept was first introduced by Been and Jefferies [4] to describe the behaviour of sand. Instead of the void ratio that was used as an essential characteristic of the sand behaviour, they suggested to use the state parameter as the fundamental variable. The size of the mean normal effective stress p' significantly influences the behaviour of the soil, so that the coarse-grained soil for the given void ratio, at a large value of the mean effective stress behaves as loose, while for lower values of the mean effective stress behaves compacted. This means that besides the void ratio, the magnitude of the mean effective stress is also necessary for the characterization of the coarse-grained napona. Parametar stanja predstavlja razliku izmeču trenutnog koeficijenta poroznosti e i koeficijenta poroznosti e_c na liniji referentnog (kriticnog) stanja, pri istom srednjem normalnom efektivnom naponu (Slika 1a):

soil. The state parameter is the difference between the current void ratio e and void ratio e_c on the reference state (critical) line at the same mean effective stress (Figure 1a):

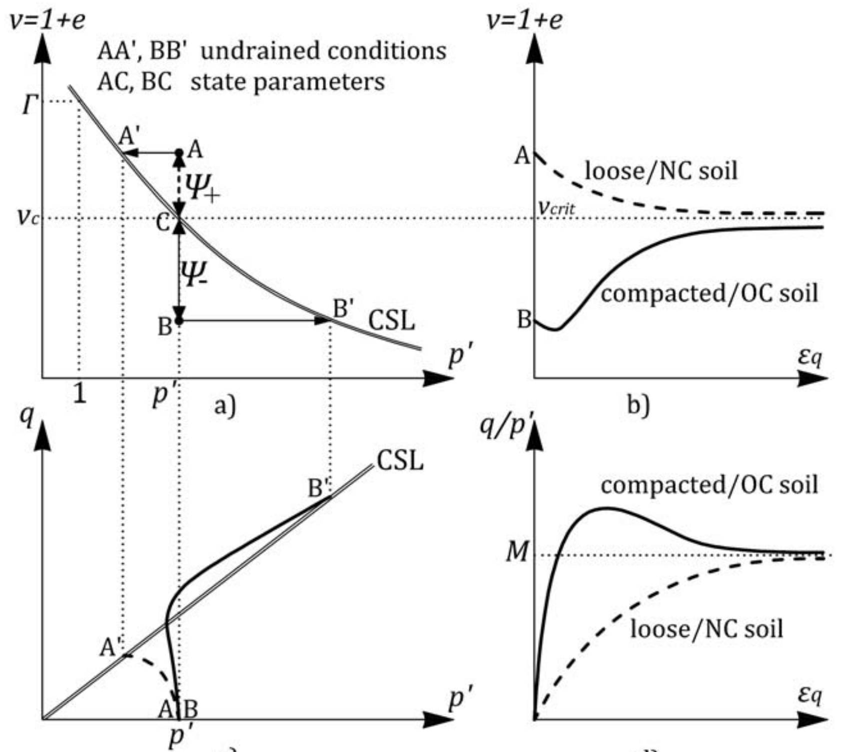
$$\Psi = e - e_c \tag{1}$$

Ovakav koncept podrazumeva da postoji referentno stanje (*steady state condition*) koje treba da ima jedinstvenu strukturu. Za konstitutivne modele, definisane u okviru teorije kriticnog stanja, referentno stanje jeste upravo kriticno stanje, kada se smicu´e deformacije razvijaju bez promene zapremine i efektivnog napona. Takoče, mora biti ispunjen uslov da je linija kriticnog stanja CSL u v–p' ravni jedinstvena, gde je v specificna zapremina tla.

Za inicijalnu vrednost parametra stanja ve´u od nule, karakteristicnu za rastresita i normalno konsolidovana tla, tacka A na Slici 1a, zapremina tla se smanjuje (kontrakcija) sve do dostizanja kriticnog stanja (Slika 1b). Dolazi do plasticnog smicu´eg loma bez pojave vršne vrednosti (Slika 1d). Ako je inicijalna vrednost parametra stanja manja od nule, kao što je slucaj sa zbijenim i prekonsolidovanim tlom – tacka B na Slici 1a – tlo ´e nakon pocetne kompresije težiti da pove´ava zapreminu (Slika 1b). Tlo ispoljava krto plasticni lom koji podrazumeva pove´anje smicu´eg napona do maksimalne velicine (vršna smicu´a cvrsto´a), a zatim opadanje smicu´eg napona (omekšanje) pri daljem deformisanju do konstantne velicine (Slika 1d). U nedreniranim uslovima, karakteristicne putanje efektivnih napona prikazane su na Slici 1c.

Such concept implies that there is a steady state condition that needs to have a unique structure. For the constitutive models defined within the critical state theory, the reference state is the critical state, when shear strains develop without changing the volume and effective stresses. Also, the condition that the critical state line CSL in v–p' plane is unique (where v is the specific soil volume) must be fulfilled.

For the initial value of the state parameter greater than zero, characteristic for loose and normally consolidated soil, point A in Figure 1a, the soil volume is decreasing (contraction) until the critical state is reached (Figure 1b). This leads to plastic shear failure (Figure 1d). If the initial value of the state parameter is less than zero, as it is the case with compacted and overconsolidated soil, point B in Figure 1a, after the initial compression the soil will tend to increase the volume (Figure 1b). The soil exhibits a brittle failure, which implies an increase in the shear stress up to the maximum value (peak shear strength), and then decrease in shear stress (softening) during further deformation to the constant value (Figure 1d). In undrained conditions, characteristic effective stress paths are shown in Figure 1c.



Slika 1. a) parametar stanja; b) promena koeficijenta poroznosti tla; c) putanje efektivnih napona u nedreniranim uslovima; d) naponsko-deformacijske krive

Figure 1. a) State parameters b) Change of the void ratio c) Effective stress paths in undrained conditions d) Stress-strain relations

Konstitutivni modeli za pesak – nastali iz koncepta parametra stanja – jesu: Nor-Sand model [17], Severn-Trent sand model [11], model koji su razvili Li & Dafalias [23].

Može se uspostaviti analogija između ponašanja zbijenih granularnih materijala i ponašanja prekonsolidovane gline, odnosno između zbijenosti i stepena prekonsolidacije, tako da se parametar stanja može uspešno koristiti i za opisivanje ponašanja prekonsolidovanih glina. Jedan od takvih modela je CASM model (Clay And Send Model) [51].

Constitutive models for sand formulated from the state parameter concept are: Nor-Sand model [17], Severn-Trent sand model [11], model developed by Li & Dafalias [23].

An analogy can be established between the behaviour of compacted granular materials and behaviour of overconsolidated clay, i.e. between compactness and overconsolidation ratio, so that the state parameter can also be used successfully to describe the behaviour of overconsolidated clays. One such model is the CASM model (Clay and Send Model) [51].

3 FORMULACIJA HASP MODELA

U okviru koncepta parametra stanja, razvijen je i HASP (*HArdening State Parameter*) model [18]. Polazna tačka za formulisanje novog konstitutivnog modela je Modifikovani Cam Clay model. U okviru koncepta granične površi [9], izvršena je modifikacija zakona ojačanja koristeći parametar stanja. Granična površ (*bounding surface*) je MCC površ čiju veličinu definiše vrednost maksimalnog srednjeg efektivnog napona $\bar{p}'_0$ (Slika 2). Ova površ može se nazvati i površ normalne konsolidacije:

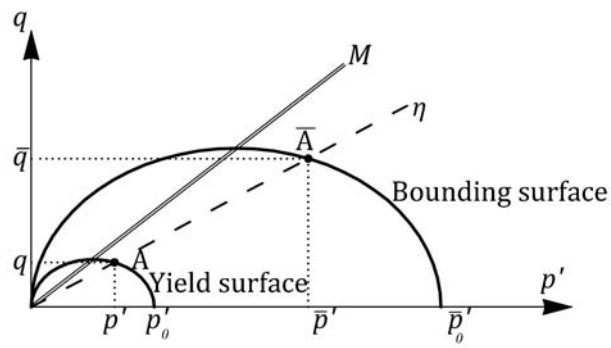
3 FORMULATION OF THE HASP MODEL

The HASP (HArdening State Parameter) model [18] was developed within the state parameter concept. The starting point for formulating a new constitutive model is the Modified Cam Clay model. Within the bounding surface concept [9] a modification of the hardening rule was made by using the state parameter. The bounding surface is the MCC surface, the size of which is defined by the value of maximum mean effective stress $\bar{p}'_0$ (Figure 2). The bounding surface can be called the surface of normal consolidation:

$$\frac{\bar{p}'}{\bar{p}'_0} = \frac{M^2}{M^2 + \eta^2} \tag{2}$$

gde je η – trenutni naponski odnos, a M – nagib linije kritičnog stanja (CSL) u naponskoj ravni.

where η is the current stress ratio and M is the slope of the critical state line (CSL) in the stress plane.



Slika 2. Koncept granične površi
Figure 2. Bounding surface concept

Tačka *A (p',q)* koja predstavlja trenutno naponsko stanje nalazi se na unutrašnjoj površi tečenja (*loading surface*), čiju veličinu definiše vrednost srednjeg efektivnog napona p'_0 :

Point *A (p',q)* that represents current stress state is located on the inner yield surface, the size of which is defined by the value of the mean effective stress p'_0 :

$$\frac{p'}{p'_0} = \frac{M^2}{M^2 + \eta^2} \tag{3}$$

Pretpostavka na kojoj se zasniva HASP model jeste da se plastične deformacije razvijaju od početka opterećivanja i da se tačka *A* uvek nalazi na površi tečenja. Tački *A* odgovara konjugovana tačka $\bar{A}(\bar{p}',\bar{q})$ na graničnoj površi, tako da je ispunjeno:

The assumption on which the HASP model is based is that plastic strains develop from the beginning of loading and point *A* is always located on the yield surface. Conjugate point $\bar{A}(\bar{p}',\bar{q})$ on the bounding surface corresponds to point *A*, so the following is fulfilled:

$$\eta = \frac{q}{p'} = \frac{\bar{q}}{\bar{p}'} \tag{4}$$

Važi asocijativni zakon tečenja, odnosno to da je vektor priraštaja plastičnih deformacija uvek upravan na površ tečenja. Granična površ ima sve karakteristike MCC površi: za naponski odnos ispod linije kritičnog stanja smanjuje se zapremina i površ se širi, dok se za naponski odnos iznad linije kritičnog stanja povećava zapremina i površ se skuplja. S druge strane, površ tečenja se širi (ojačanje) do dostizanja vršne čvrstoće pri naponskom odnosu $\eta=M_f$, a zatim se skuplja (omekšanje) do dostizanja kritičnog stanja $\eta=M$.

Associated flow rule applies, i.e. plastic strain increment vector is always normal to the yield surface. Bounding surface possesses all the characteristics of the MCC surface. For stress ratio below the critical state line, the volume decreases and the surface expands, while for stress ratio above the critical state line, the volume increases and the surface shrinks. On the other hand, yield surface expands (hardening) until peak strength is reached at stress ratio $\eta=M_f$, after which it shrinks (softening) until critical state is reached $\eta=M$.

3.1 Zakon ojačanja HASP modela

Zakon ojačanja MCC modela zavisi samo od zapreminske plastične deformacije. Generalni zahtev za prekonsolidovana tla je prelaz iz kompresije u ekspanziju pre dostizanja vršne čvrstoće. Zakon ojačanja – koji je u funkciji samo zapreminske plastične deformacije – ne omogućava adekvatno opisivanje dilatancije i ojačanja kod prekonsolidovanih glina. Da bi površ tečenja nastavila da se širi i za vrednosti naponskog odnosa $M<\eta<M_f$, potrebno je koristiti kombinovani zakon ojačanja i formulisati ga u funkciji i plastične smičuće deformacije [28, 50]:

3.1 The hardening rule of the HASP model

The hardening rule of the HASP model depends only on plastic volumetric strains. General requirement for overconsolidated soil is transition from contractive to dilatant behaviour before the peak strength is reached. The hardening rule that is only the function of volumetric plastic strain does not allow adequate description of dilatancy and hardening for overconsolidated clays. In order for the yield surface to continue expanding also for stress ratio values $M<\eta<M_f$, it is necessary to use the combined hardening and express the hardening rule as a function of plastic shear strain also [28, 50]:

$$dp_0' = \frac{v}{\lambda - \kappa} p_0' \left(d\varepsilon_v^p + \xi d\varepsilon_q^p \right) \tag{5}$$

gde je ξ parametar koji treba definisati, a p_0' parametar ojačanja MCC modela. Parametri λ i κ predstavljaju nagibe linije izotropne konsolidacije i linije bubrenja u *v-lnp'* dijagramu. Kombinovani zakon ojačanja utiče na putanju napona koja prelazi liniju kritičnog stanja i dostiže se vršna čvrstoća u dreniranim uslovima. U nedreniranim uslovima, kombinovano ojačanje omogućava predviđanje putanje efektivnih napona „S” oblika, što je karakteristično za prekonsolidovane gline. Ako definišemo dilatanciju kao odnos priraštaja zapreminske i smičuće komponente plastične deformacije:

where ξ is the parameter to be defined, and p_0' is hardening parameter of the MCC model. Parameters λ and κ are slopes of isotropic consolidation line and swelling lines in *v-lnp'* plane. The combined hardening rule influences the stress path that crosses the critical state line and the peak strength is reached in drained conditions. In undrained conditions, the combined hardening is key to achieve "S" shaped effective stress path, which is typical for overconsolidated clays. If we define dilatancy via the ratio of increment of volumetric and shear component of plastic strain:

$$d = \frac{d\varepsilon_v^p}{d\varepsilon_q^p} \tag{6}$$

a trenutni stepen prekonsolidacije u toku procesa deformisanja kao:

and the current overconsolidation ratio during the deformation process as:

$$R = \frac{\bar{p}'}{p'} = \frac{\bar{q}}{q} = \frac{\bar{p}_0'}{p_0'} \tag{7}$$

izraz za zakon ojačanja postaje:

the expression for the hardening rule becomes:

$$dp_0' = \frac{v}{\lambda - \kappa} p_0' d\varepsilon_v^p \left(1 + \frac{\xi}{d} \right) R = \frac{v}{\lambda - \kappa} p_0' d\varepsilon_v^p \omega \tag{8}$$

gde je ω koeficijent ojačanja (*hardening coefficient*):

where ω is the hardening coefficient:

$$\omega = \left(1 + \frac{\xi}{d} \right) R \tag{9}$$

Kompletne konstitutivne relacije HASP modela mogu se sada predstaviti kao:

Complete constitutive relations of the HASP model can be presented as:

$$\begin{Bmatrix} d\varepsilon_v \\ d\varepsilon_q \end{Bmatrix} = \begin{bmatrix} \frac{1}{K} + \frac{\lambda - \kappa}{vp'}\frac{1}{\omega}\frac{M^2 - \eta^2}{M^2 + \eta^2} & \frac{\lambda - \kappa}{vp'}\frac{1}{\omega}\frac{2\eta}{M^2 + \eta^2} \\ \frac{\lambda - \kappa}{vp'}\frac{1}{\omega}\frac{2\eta}{M^2 + \eta^2} & \frac{1}{3G} + \frac{\lambda - \kappa}{vp'}\frac{1}{\omega}\frac{4\eta^2}{\left(M^2 + \eta^2\right)\left(M^2 - \eta^2\right)} \end{bmatrix} \begin{Bmatrix} dp' \\ dq \end{Bmatrix} \tag{10}$$

Koeficijent ojačanja ω direktno utiče i na veličinu plastičnih deformacija, tako da se adekvatnom formulacijom koeficijenta ojačanja mogu značajno redukovati plastične deformacije prekonsolidovane gline u početnoj fazi opterećivanja, kada MCC model predviđa samo elastične deformacije. Na taj način je moguće pretpostaviti da tlo od samog početka opterećivanja trpi i plastične deformacije, koje su tada veoma male. Kako se u procesu deformisanja polako smanjuje i stepen prekonsolidacije tla, tako se i koeficijent ω smanjuje ($\omega \to 1$) i plastične deformacije postaju dominantne. Pri dostizanju vršne čvrstoće (prelaz iz ojačanja u omekšanje), uočava se maksimalni gradijent promene zapremine – maksimalna dilatancija i iz izraza (8) sledi da je ω=0. Tada važi relacija $\xi = -d_{max}$, što znači da parametar ξ predstavlja maksimalnu vrednost dilatancije pri vršnoj čvrstoći u dreniranim uslovima [29].

U izrazu za koeficijent ojačanja (9) odnos ξ/d je definisan preko parametra stanja. Parametar stanja za trenutnu naponsku tačku (Slika 3) može se izraziti kao:

The hardening coefficient ω directly affects the value of the plastic strains, and thus, with the adequate formulation of the hardening coefficient, it is possible to significantly reduce the plastic strains of overconsolidated clay in the initial load phase, when the MCC model predicts only elastic strains. It is then possible to assume that soil deforms plastically from the very beginning of loading. As the overconsolidation ratio of soil decreases in the deformation process, the hardening coefficient ω also decreases ($\omega \to 1$) and plastic strains become dominant. When reaching the peak strength (transition from hardening to softening) the maximum volume change gradient is observed – maximum dilatancy and from expression (8) it can be concluded that ω=0. Then the relation $\xi = -d_{max}$ applies, which means that parameter ξ is the maximum dilatancy value at peak strength in drained conditions [29].

In the expression for hardening coefficient (9) the ration ξ/d is defined via the state parameter. State parameter for the current stress point (Figure 3) can be expressed as:

$$\Psi = v + \lambda \ln p' - \Gamma \tag{11}$$

gde je Γ – parametar koji definiše položaj CSL u kompresionoj p'-v ravni. Parametar stanja je negativan za jako prekonsolidovane gline, dok je za lako prekonsolidovane i normalno konsolidovane gline – pozitivan. Parametar stanja za imaginarnu naponsku tačku iznosi:

where Γ is the parameter that defines the position of CSL in compression p'-v plain. State parameter is negative for highly overconsolidated clays, while for lightly overconsolidated and normally overconsolidated clays it is positive. State parameter for conjugate stress point is:

$$\bar{\Psi} = \left(\lambda - \kappa\right) \ln\left(\frac{2M^2}{M^2 + \eta^2}\right) \tag{12}$$

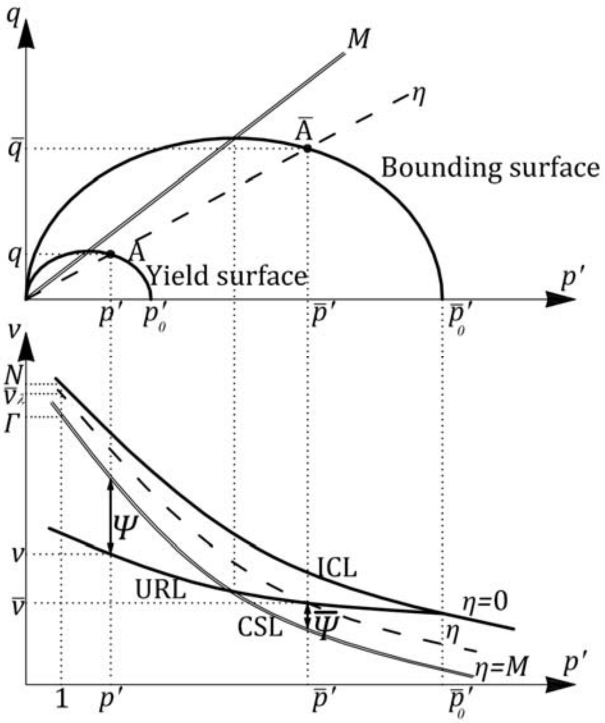
Stepen prekonsolidacije (7) može se takođe izraziti kao funkcija parametara stanja:

The overconsolidation ratio (7) can also be expressed as a function of state parameter:

$$R = \frac{\bar{p}'}{p'} = \frac{\bar{q}}{q} = \exp\left(\frac{\bar{\Psi} - \Psi}{\lambda - \kappa}\right) \tag{13}$$

Na osnovu velikog broja triaksijalnih opita na pesku i prekonsolidovanoj glini, Parry [34] je pokazao da je dilatancija pri vršnoj čvrstoći u dreniranim uslovima proporcionalna stepenu prekonsolidacije, a Been & Jefferies [4] su pokazali da je parametar stanja linearno proporcionalan dilatanciji. U skladu s navedenim i imajući u vidu vezu između parametra stanja i stepena prekonsolidacije (13), pretpostavljeno je da je maksimalna vrednost dilatancije u direktnoj zavisnosti od $\bar{\Psi} - \Psi$. Takođe, može se pokazati da se dilatancija menja na sličan način kao parametar stanja za imaginarnu tačku $\bar{\Psi}$. Na osnovu navedenog, sledi da se odnos ξ/d može izraziti preko parametra stanja kao:

On the basis of a large number of triaxial tests on sand and overconsolidated clays, Parry [34] showed that the dilatancy at peak strength in drained conditions is in proportion to the overconsolidation ratio. Also, Been & Jefferies [4] showed that the state parameter is in linear proportion to the dilatancy. In accordance with the aforementioned and taking into account the relationship between the state parameter and overconsolidation ratio (13), it is assumed that the maximum value of dilatancy is directly dependent on $\bar{\Psi} - \Psi$. On the other hand, it can be shown that the dilatancy changes in a similar manner as the state parameter for conjugate stress point $\bar{\Psi}$. Based on the above, it can be concluded that the ratio ξ/d can be expressed via the state parameter:



Slika 3. Parametri stanja za trenutnu i imaginarnu naponsku tačku
Figure 3. State parameters for current and conjugate stress points

$$\frac{\xi}{d}=\frac{\bar{\Psi}-\Psi}{\bar{\Psi}} \tag{14}$$

pa je izraz za koeficijent ojačanja:

and the expression for the hardening coefficient becomes:

$$\omega=\left(1+\frac{\bar{\Psi}-\Psi}{\bar{\Psi}}\right)R \tag{15}$$

Deo izraza (15) u zagradi određuje znak koeficijenta ojačanja i zajedno sa stepenom prekonsolidacije određuje magnitudu koeficijenta ojačanja, a samim tim i veličinu plastičnih deformacija u skladu sa izrazom (10). Za normalno konsolidovane gline važi da je $\Psi=\bar{\Psi}$ i koeficijent ojačanja je $\omega=1$. HASP model tada automatski prelazi u MCC model. Za opis kompletne konstitutivne veze potrebno je pet materijalnih parametara (*M, λ, κ, Γ, μ* - Poisson-ov koeficijent), kao i kod MCC modela i mogu se odrediti iz konvencionalnog triaksijalnog opita, opita direktnog smicanja i edometarskog opita. HASP model, uvođenjem parametra stanja kao unutrašnje promenljive, prevazišao je nedostatke MCC modela, zadržavajući isti set ulaznih parametara, što predstavlja prednost u inženjerskoj implementaciji u poređenju s drugim modelima za prekonsolidovane gline.

The part of the expression (15) in parenthesis controls the sign of the hardening coefficient and with the overconsolidated ratio determines the magnitude of the hardening coefficient and hence affects the magnitude of plastic strains according to expression (10). For normally consolidated clays, the HASP model automatically transforms into the MCC model since $\Psi=\bar{\Psi}$ and the hardening coefficient is $\omega=1$. For the description of stress-strain relations, five material parameters (*M, λ, κ, Γ, μ* - Poisson's coefficient) are needed, just like with the MCC model, and all parameters can be determined from the conventional triaxial test, direct shear test and oedometer test. By introducing the state parameter as an internal variable, the HASP model overcomes many deficiencies of the MCC model, while keeping the same set of input parameters, which is an advantage in engineering implementation compared to other constitutive models for overconsolidated clays.

4 VALIDACIJA HASP MODELA

Validacija HASP modela sprovedena je poređenjem rezultata simulacije laboratorijskih opita sa publikovanim eksperimentalnim rezultatima s različitim putanjama totalnih napona. Da bi se potvrdila efikasnost HASP modela, urađeno je i poređenje s predviđanjem MCC modela. U postupku validacije, izabrane su gline s različitim stepenima prekonsolidacije, za koje u literaturi postoje dobro dokumentovana ispitivanja u triaksijalnom aparatu i za koje su već određeni parametri konstitutivnog MCC modela (Tabela 1). Navedeni parametri predstavljaju ujedno i parametre HASP modela.

4 VALIDATION OF THE HASP MODEL

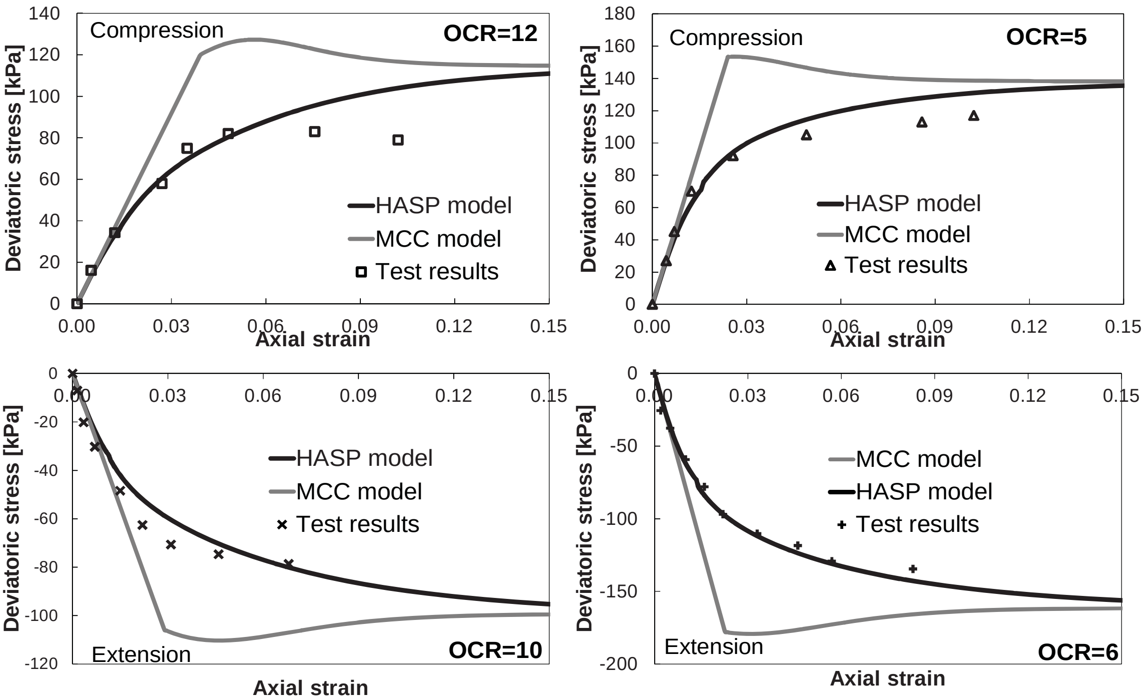
The HASP model validation is performed by comparing the results of simulation of laboratory tests with published experimental results with different total stress paths. In order to confirm the HASP model efficiency, comparison was also made with the prediction of the MCC model. Clays with different overconsolidation ratios were selected, for which in literature there are well-documented triaxial test results and for which parameters of the MCC model have already been determined (Table 1). These parameters are at the same time the parameters of the HASP model.

Tabela 1. Parametri MCC i HASP modela
Table 1. Parameters of MCC and HASP model

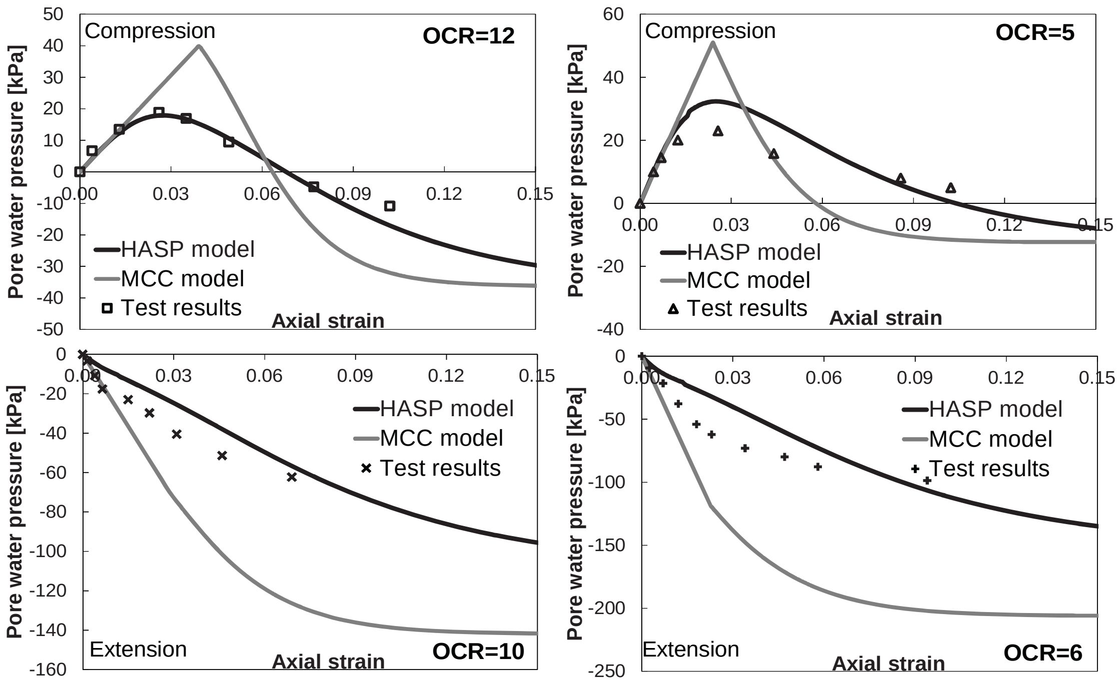
.	λ	κ	M_c	M_e	Γ	μ
Cardiff glina [3] – CU opiti *Cardiff clay* [3] – *CU tests*	0.140	0.050	1.05	0.85	2.63	0.2
Kaolin glina [5] – CD opiti *Kaolin clay* [5] – CD tests	0.230	0.030	0.81	/	3.44	0.2

Prikazani su rezultati dva nedrenirana opita triaksijalne kompresije na prerađenim uzorcima Cardiff gline [3] sa stepenima prekonsolidacije 5 i 12, kao i rezultati dva nedrenirana opita triaksijalne ekstenzije sa stepenima prekonsolidacije 6 i 10 (CU opiti).

The results shown are from two undrained triaxial compression tests on remolded samples of Cardiff clay [3] with overconsolidation ratios 5 and 12, as well as results of two undrained triaxial extension tests with overconsolidation ratios 6 and 10 (CU tests).



Slika 4. CU opiti, Cardiff glina – naponsko-deformacijske relacije
Figure 4. CU tests, Cardiff clay – stress-strain relations



Slika 5. CU opiti, Cardiff glina – promena pornog pritiska
Figure 5. CU tests, Cardiff clay – pore water pressure

Naponsko-deformacijske relacije (Slika 4) i promene pornog pritiska (Slika 5), dobijene HASP modelom pokazuju veoma dobro slaganje sa eksperimentalnim rezultatima, za sve stepene prekonsolidacije pri triaksijalnoj kompresiji i ekstenziji. Može se uočiti da MCC model ne opisuje adekvatno ponašanje prekonsolidovane gline u nedreniranim uslovima. Vrednosti devijatora napona i pornog pritiska znatno su precenjene i odstupanja su veća što je veći stepen prekonsolidacije.

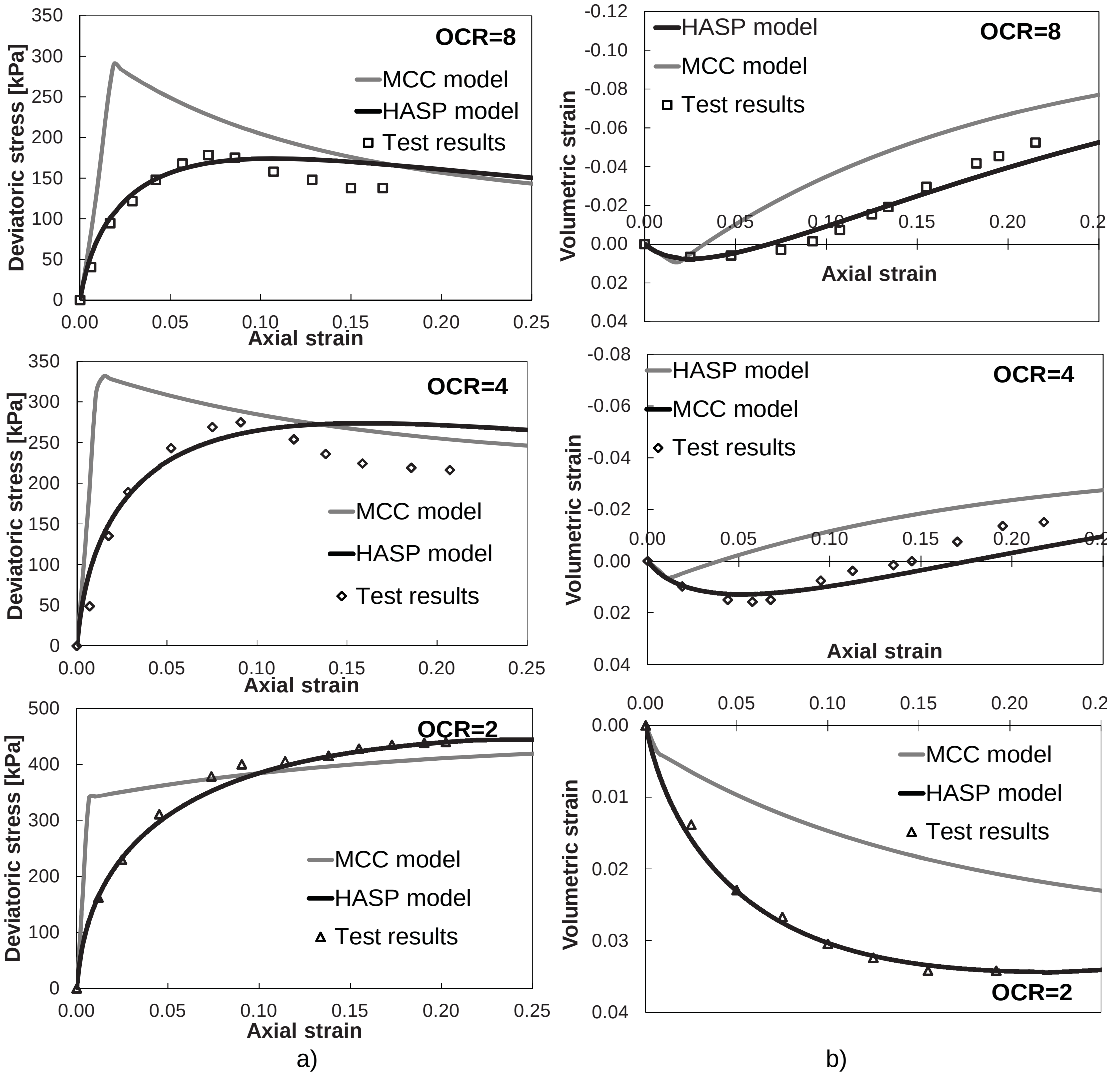
Na Slici 6 su prikazani rezultati dreniranih opita triaksijalne kompresije (CD opiti) na kaolinskoj glini [5] sa stepenima prekonsolidacije 8, 4 i 2.

Ponašanje prekonsolidovanih glina tokom ojačanja veoma je dobro opisano HASP modelom. Za uzorke sa stepenima prekonsolidacije 8 i 4, HASP model predviđa pad čvrstoće – omekšanje pri deformacijama većim od oko 10% (Slika 6a). Za jako prekonsolidovane uzorke (OCR=8, 4), nakon početne kompresije uzoraka, dolazi do ekspanzije i povećanja zapremine (Slika 6b), što je u skladu sa eksperimentalnim rezultatima i uočava se odlično predviđanje promene zapreminskih deformacija s promenom smičućih deformacija. Nedostaci MCC modela, pri opisu mehaničkog ponašanja prekonsolidovanih glina, mogu se uočiti i u dreniranim uslovima. Vršna čvrstoća je precenjena i do dva puta. Detaljan prikaz validacije HASP modela na nekoliko prekonsolidovanih glina s različitim stepenima prekonsolidacije dat je u radu [18].

Stress-strain relations (Figure 4) and changes in pore water pressure (Figure 5) obtained using the HASP model correspond well to the experimental results, for all overconsolidation ratios at triaxial compression and extension. It can be seen that the MCC model fails to adequately describe the behaviour of overconsolidated clays in undrained conditions. Values of deviatoric stresses and pore water pressure are significantly overestimated and deviations are bigger with greater overconsolidation ratio.

Figure 6 shows the results of drained triaxial compression tests (CD tests) on kaolin clay [5] with overconsolidation ratios 8, 4 and 2.

The behaviour of overconsolidated clays during hardening is very well described with the HASP model. For samples with overconsolidation ratios 8 and 4, the HASP model predicts a drop in strength – softening, at strains greater than about 10% (Figure 6a). For highly overconsolidated samples (OCR=8, 4), after the initial compression of the samples, there is an increase in volume (Figure 6b) which is in accordance with experimental results, and excellent prediction of the change in volumetric strains is observed. Deficiencies of the MCC model in describing mechanical behaviour of overconsolidated clays can also be seen in drained conditions. The peak strength is overestimated up to twice the real value. Detailed overview of the validation of the HASP model on several overconsolidated clays with different overconsolidation ratios is shown in the paper [18].



Slika 6. CD opiti, kaolinska glina a) naponsko-deformacijske relacije; b) zapreminske deformacije
Figure 6. CD tests, kaolin clay a) stress-strain relations b) volumetric strains

5 IMPLEMENTACIJA HASP MODELA

Praktična primena složenih elasto-plastičnih konstitutivnih modela u proračunu geotehničkih konstrukcija zahteva korišćenje numeričkih metoda kao što je metoda konačnih elemenata (MKE). Da bi se jedan takav model implementirao u MKE, neophodno je izvršiti numeričku integraciju konstitutivnih relacija, tj. izvršiti integraciju napona za dati inkrement deformacije. Postupak numeričke integracije mora biti stabilan i dovoljno tačan, jer od tačnosti postupka integracije zavisi tačnost rešenja razmatranog graničnog problema

Postoje eksplicitne i implicitne metode za numeričku. integraciju. U slučaju eksplicitnih metoda integracije, do priraštaja napona dolazimo koristeći poznato naponsko stanje na početku inkrementa, u konfiguraciji *t*. U literaturi se mogu naći brojne eksplicitne metode integracije [27, 33, 31, 43, 39, 40, 44]. Razvoj implicitnih metoda počinje sa Wilkins-om [49]. U implicitnim metodama integracije, do priraštaja napona dolazimo koristeći poznate veličine na kraju inkrementa, u

5 IMPLEMENTATION OF THE HASP MODEL

Practical implementation of complex elastic-plastic constitutive models in geotechnical analysis requires the use of numerical methods such as the Finite Element Method (FEM). In order for constitutive model to be implemented in the FEM, it is necessary to perform numerical integration of the constitutive relations, i.e. to perform integration of stresses for the given strain increment. The procedure of numerical integration must be stable and sufficiently accurate, because accuracy of the solution of the considered boundary value problem depends on the accuracy of the integration procedure.

There are explicit and implicit methods for numerical integration. With explicit methods of integration, stress increment is determined by using known stress state at the beginning of the increment, in configuration *t*. In the literature, there are numerous explicit methods of integration [27, 33, 31, 43, 39, 40, 44]. Development of implicit methods begins with Wilkins [49]. In implicit methods of integration, stress increment is determined konfiguraciji $t+\Delta t$. Pocedura se generalno sastoji od dva koraka: proračuna elastičnog rešenja za dati inkrement (elastično predviđanje) i povratka na površ tečenja (plastični korektor). Ovaj pristup kasnije su koristili i razvijali brojni autori i tako je nastala klasa procedura integracije koja se naziva povratno preslikavanje [30, 41, 32, 6, 7, 42, 14]. Implicitnu šemu integracije – nazvanu Metoda vodećeg parametra (Governing Parameter Method) GPM – razvijali su Kojić i Bathe [19-21]. Predstavlja generalizaciju *radial return* metode koju je predstavio Wilkins [49]. Osnovni princip jeste da se sve nepoznate veličine izraze u funkciji jednog parametra (vodeći parametar) i problem se svodi na rešavanje jedne nelinearne jednačine po nepoznatom vodećem parametru. Za HASP model je korišćena GPM metoda, gde je kao vodeći parametar korišćen srednji normalni efektivni napon p' [47] kao veličina s jasnim fizičkim značenjem i s definisanim intervalom mogućih vrednosti. HASP model je implementiran u Abaqus/Standard [1], koristeći korisnički potprogram UMAT i numeričku proceduru za integraciju napona GPM.

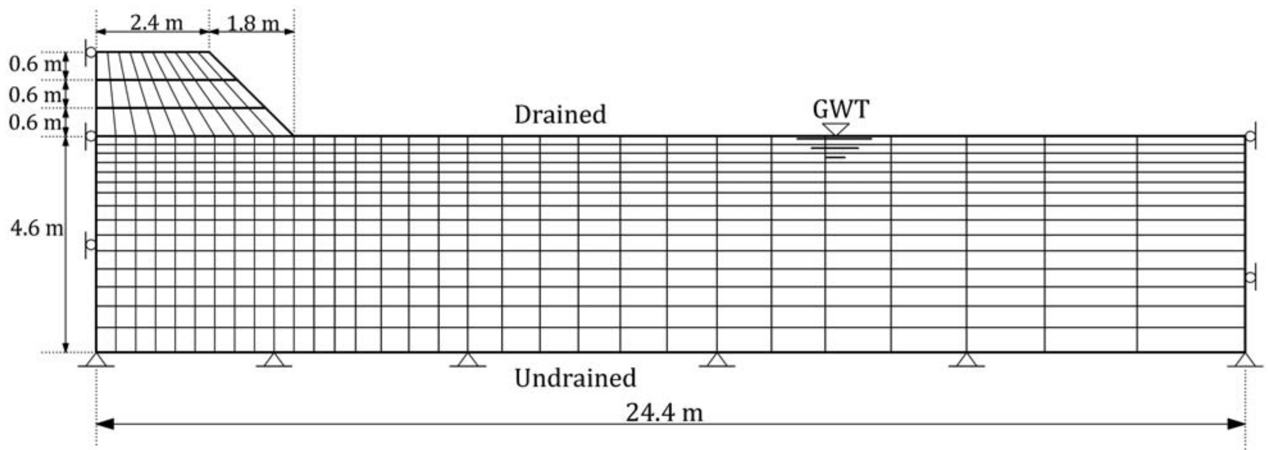
5.1 Konsolidacija sloja gline

Kao primer implementacije HASP modela, urađena je analiza konsolidacionog sleganja tla usled fazne izgradnje nasipa na površini terena (primer u knjizi *Applied Soil Mechanics with Abaqus Applications* [13]). Model se sastoji od sloja gline, debljine 4.6 m, koji leži na nepropusnoj i nestišljivoj podlozi. Nivo podzemne vode se nalazi na površini terena, kao što je prikazano na Slici 7. Nasip se gradi u tri jednaka sloja debljine 0.6 m. Ukupna visina nasipa iznosi 1.8 m. Konstrukcija nasipa se izvodi po fazama/slojevima, a izgradnja jednog sloja traje dva dana, dok izgradnja čitavog nasipa traje šest dana. U modelu, konsolidacija gline nakon izgradnje nasipa traje još 200 dana.

by using known variables at the end of the increment, in configuration $t+\Delta t$. The procedure generally consists of two steps: estimate of the elastic solution for the given increment (elastic prediction) and return to the yield surface (plastic corrector). This approach was later used and further developed by numerous authors and so the class of integration procedures was created, called return mapping [30, 41, 32, 6, 7, 42, 14]. The implicit integration scheme that is called the Governing Parameter Method (GPM) was developed by Kojić and Bathe [19-21]. It is a generalization of the *radial return* method which was introduced by Wilkins [49]. The basic principle is that all unknown variables are expressed in the function of one parameter (the governing parameter) and the problem is reduced to the solving of one non-linear equation with respect to the governing parameter. For the HASP model, the mean effective stress p' [47] was selected as the governing parameter as a value with clear physical meaning and with defined interval of possible values. The HASP model is implemented in Abaqus/Standard [1] using the user subroutine UMAT and GPM as numerical procedure for stress integration.

5.1 Consolidation of clay layer

As an example of implementation of the HASP model, analysis of the soil consolidation as the result of phased construction of the embankment on the clay surface was performed (example in book *Applied Soil Mechanics with Abaqus Applications* [13]). The FEM model consists of a layer of clay, 4.6 m thick, which lies on impermeable and incompressible base. The ground water table is on the clay surface, as shown in Figure 7. The embankment is built in three equal layers, 0.6 m thick. Total height of the embankment is 1.8 m. The structure of the embankment is made by phases/layers and construction of one layer takes two days, while the construction of the entire embankment takes six days. The consolidation of clay after construction of the embankment takes another 200 days.



Slika 7. Model s mrežom konačnih elemenata za numeričku analizu
Figure 7. Model with the finite element mesh for numerical analysis

Ispitana je mogućnosti HASP modela da tokom simulacije navedenog procesa predvidi promenu pornog natpritiska, kao i veličine vremenskog sleganja nasipa i sloja gline.

The possibility of the HASP model to predict the change of pore water pressure, as well as the value of the consolidation settlement of the embankment and clay layer was performed.

Materijali

Materials

Nasip se gradi od prašinastog peska i modeliran je linearno-elastičnim modelom. Parametri linearno-elastičnog modela prikazani su u Tabeli 2. Sloj visokoplastične gline ispod nasipa modeliran je HASP modelom (Tabela 3).

The embankment is built of silty sand and is modelled using the linear-elastic model. Parameters of the linear-elastic model are shown in Table 2. The layer of highly overconsolidated clay below the embankment is modelled using the HASP model (Table 3).

Tabela 2. Parametri nasipa
Table 2. Parameters of the embankment

Linearno-elastični model *Linear-elastic model*		Karakteristike materijala nasipa *Characteristics of embankment material*		
E [MPa]	μ	γ[kN/m^3]	k [m/s]	e_0
5	0.3	18.85	0.001	0.889

Tabela 3. Parametri HASP modela
Table 3. Parameters of the HASP model

λ	κ	M	Γ	μ
0.174	0.026	1.5	3.87	0.28

Analiza je rađena s različitim inicijalnim uslovima, odnosno različitim početnim stepenom prekonsolidacije sloja gline (prikazanim u Tabeli 4) i sprovedena je u pet proračunskih koraka, za svaki stepen prekonsolidacije. U prvom proračunskom koraku, nasip je uklonjen iz mreže konačnih elemenata. U sledeća tri koraka je simulirana izgradnja nasipa u tri sloja, pri čemu je svaki naredni sloj dodat na već deformisani prethodni. Peti korak je konsolidacija gline i nasipa u trajanju od 200 dana.

The analysis was performed with different initial conditions, i.e. different initial overconsolidation ratios of clay layer (Table 4) in five calculation steps. In the first calculation step, the embankment is removed from the finite element mesh. The next three steps consist of simulation of the construction of the embankment in three layers, where each subsequent layer was added to the already deformed previous one. The fifth step is consolidation of clay and the embankment for a period of 200 days.

Tabela 4. Inicijalni uslovi
Table 4. Initial conditions

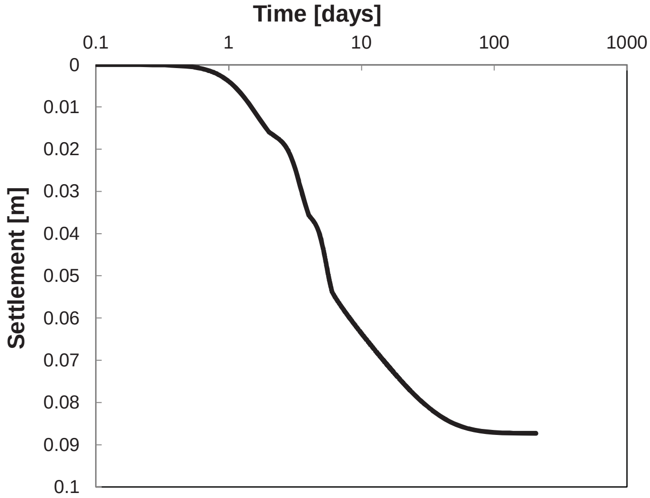
e_0	γ[kN/m^3]	OCR	k_0
1.1	17.75	2	0.75
1.0	18.15	5	0.85
0.9	18.60	8	1.0
0.8	19.10	12	1.3
0.7	19.60	18	1.9

Rezultati

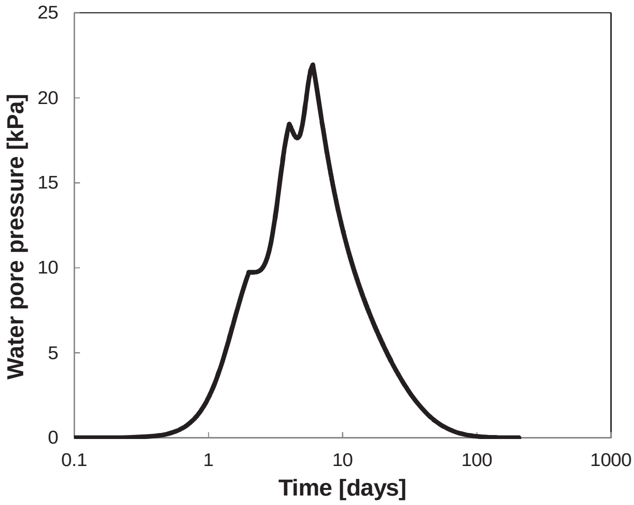
Results

Za analizu pojedinačnih rezultata – kao ilustracija – odabrani su rezultati za stepen prekonsolidacije OCR=5. Na Slici 8 je prikazan vremenski tok sleganja ispod centra nasipa (površina sloja gline) u polulogaritamskoj razmeri. Deformacije se najbrže razvijaju (najveći gradijent) tokom prvih šest dana koliko traje izgradnja nasipa i do tada se desilo oko 50% od ukupnih sleganja. Na slici 9 je prikazana istorija razvoja pornog natpritiska u sredini sloja gline ispod centra nasipa. Porni natpritisak raste tokom izgradnje nasipa (šest dana) i tokom procesa konsolidacije dolazi do njegove potpune disipacije.

For analysis of individual results, the results for the overconsolidation ratio OCR=5 were selected as an illustration. Figure 8 shows the timeline of the settlement under the centre of the embankment (surface of the clay layer) in semi-logarithmic plot. Strains develop most quickly (the highest gradient) during the first 6 days, which is how long the construction of the embankment lasts, and about 50% of the total settlement occurred by that time. Figure 9 shows the history of development of pore water pressure in the middle of the clay layer under the centre of the embankment. The pore water pressure increases during the construction of the embankment (six days) and during the consolidation process its full dissipation occurs.



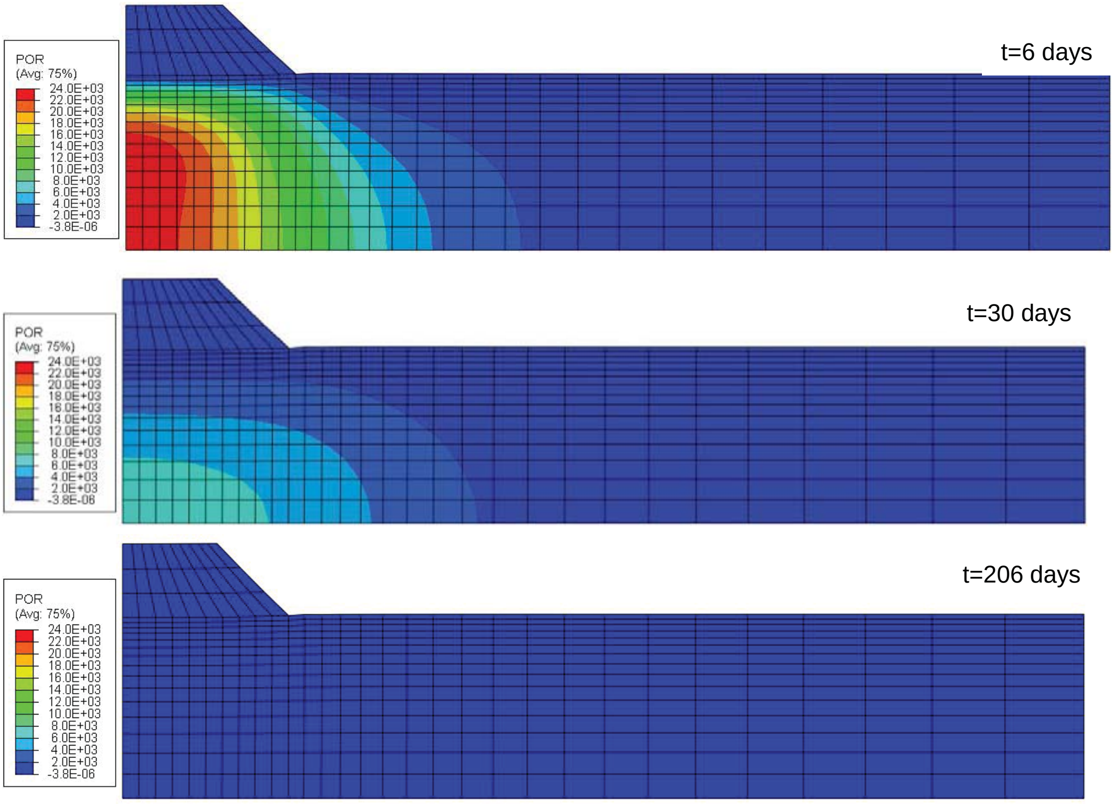
Slika 8. Sleganje gline ispod centra nasipa tokom vremena, OCR=5
Figure 8. Settlement of clay layer under the centre of the embankment over time, OCR=5



Slika 9. Razvoj pornog natpritiska u sredini sloja gline ispod centra nasipa, OCR=5
Figure 9. Development of pore water pressure in the middle of the clay layer under the centre of the embankment, OCR=5

Raspodela pornog natpritiska i disipacija tokom vremena data je na Slici 10. Usled brzog opterećivanja sloja zasićene gline male vodopropusnosti, ispod nasipa se odmah nakon nanošenja opeterećenja razvija porni natpritisak. S obzirom da je omogućeno dreniranje vode samo preko gornje površine, do najbrže disipacije dolazi upravo na gornjoj površini sloja gline.

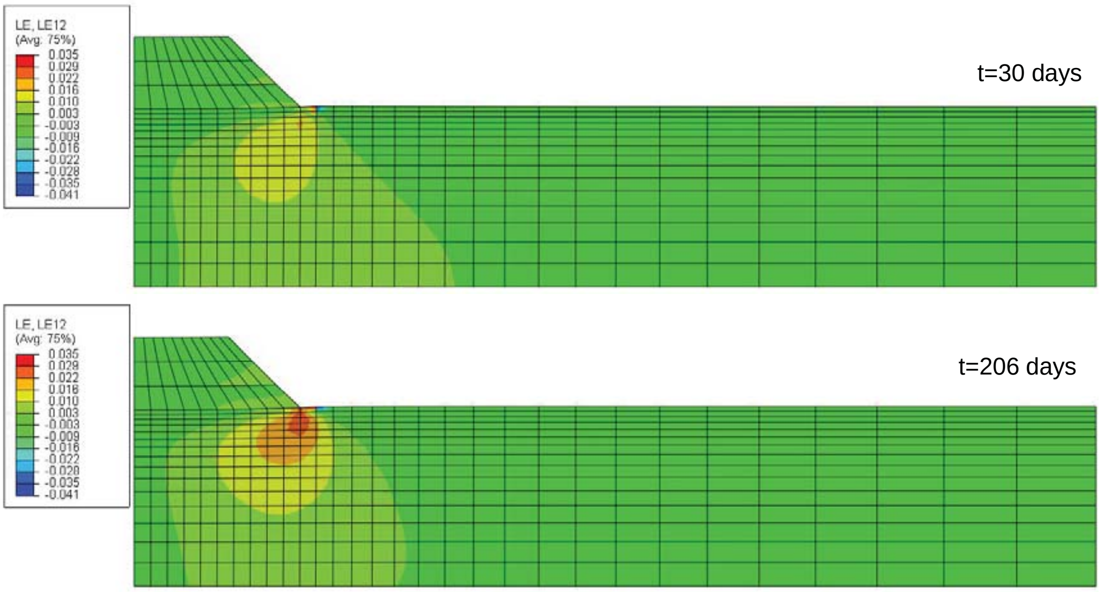
Distribution of the pore water pressure and dissipation over time is shown in Figure 10. As the result of rapid loading of the layer of saturated clay of low water permeability, the pore water pressure develops under the embankment immediately after placing the load. Since water draining is enabled only over the upper surface, the fastest dissipation occurs exactly on the upper surface of the clay layer.



Slika 10. Razvoj pornog natpritiska tokom vremena, OCR=5
Figure 10. Development of pore water pressure over time, OCR=5

Raspodela smičućih deformacija je prikazana na Slici 11, gde se može uočiti da se maksimalne vrednosti smičućih deformacija javljaju u nožici kosine nasipa.

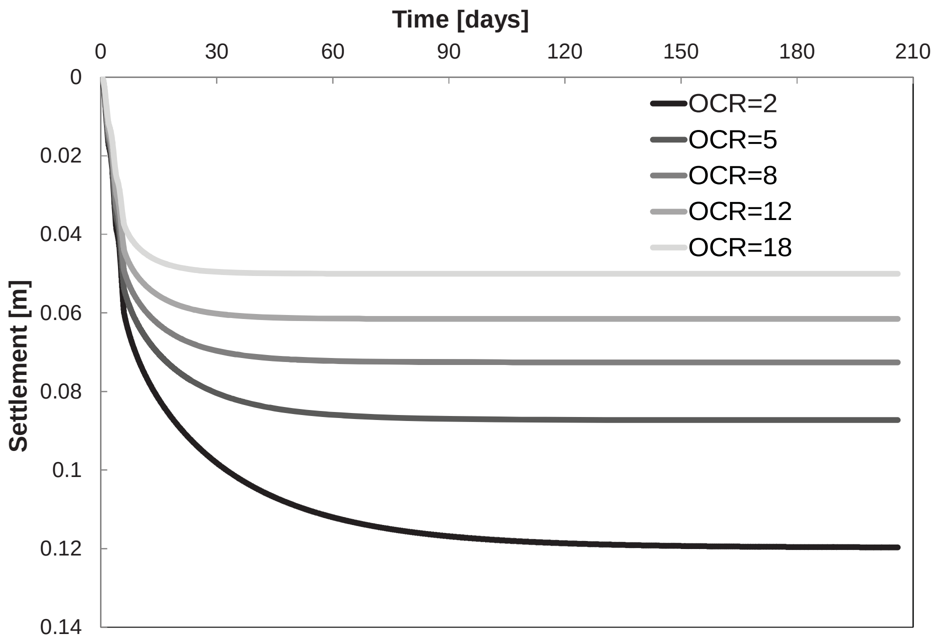
Distribution of shear strains is shown in Figure 11, where it can be observed that the maximum values of shear strains appear in the toe of the slope of the embankment.



Slika 11. Razvoj smičućih deformacija tokom vremena, OCR=5
Figure 11. Development of shear strains over time, OCR=5

Sleganja sloja gline tokom 206 dana ispod centra nasipa, za sve stepene prekonsolidacije, data su na Slici 12. Najveća sleganja, kao što se i očekuje, dobijena su za blago prekonsolidovane gline.

Settlements of clay layer over 206 days under the centre of the embankment for all overconsolidation ratios are shown in Figure 12. The largest settlements were, as expected, obtained for lightly overconsolidated clays.



Slika 12. Sleganje sloja gline za različite stepene prekonsolidacije posle 206 dana
Figure 12. Settlements of clay layer for different overconsolidation ratios after 206 days

5.2 Poređenje s MCC modelom

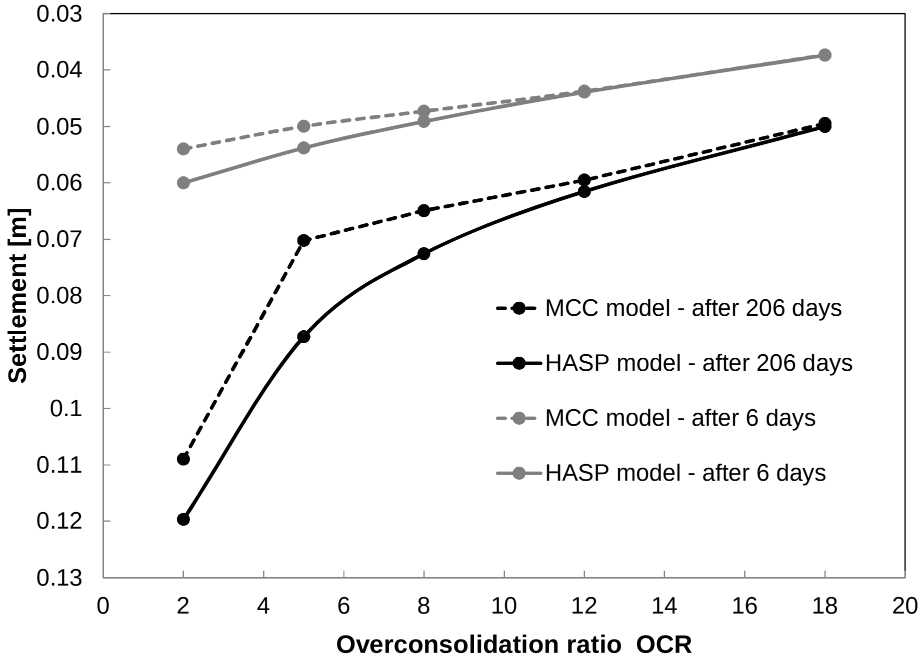
Isti granični problem je analiziran i koristeći MCC model koji već postoji kao standardni materijalni model u Abaqus-u. Predviđa se slična promena pornog pritiska, sleganja i smičućih deformacija tokom vremena, dok je osnovna razlika u veličini zapreminskih i smičućih deformacija. Koristeći HASP model, generalno se dobijaju veće vrednosti deformacija i sleganja u odnosu na MCC model, naročito za manje stepene prekonsolidacije. Takvi rezultati su očekivani, s obzirom na to što HASP model predviđa elasto-plastično ponašanje od samog početka procesa deformisanja, dok MCC model predviđa samo elastično ponašanje unutar inicijalne površi tečenja.

Za veliki stepen prekonsolidacije (u datoj analizi OCR>12), predviđaju se slične vrednosti sleganja za oba modela (Slika 13). HASP model, zahvaljujući velikom koeficijentu ojačanja ω za veliki stepen prekonsolidacije, predviđa male vrednosti plastičnih deformacija i ukupne vrednosti deformacija se ne razlikuju značajno od veličine elastičnih deformacija. Pri manjim vrednostima stepena prekonsolidacije, odstupanja u veličini deformacije značajno su veća. Dok materijal opisan MCC modelom ostaje u elastičnoj zoni za prikazana opterećenja i za manje vrednosti stepena prekonsolidacije, HASP model predviđa veće vrednosti plastičnih deformacija usled manje vrednosti koeficijenta ojačanja ω. U prikazanoj analizi, razlike u sleganjima iznose i do 20–25%.

5.2 Comparison with the MCC model

The same boundary value problem was analyzed by using the MCC model, which already exists as a standard material model in Abaqus. It predicts similar change in pore water pressure, settlements and shear strains over time, while the main difference is in the magnitude of the volumetric and shear strains. By using the HASP model, generally higher values of deformations are obtained compared to those from the MCC model, especially for lower overconsolidation ratios. Such results are expected, since the HASP model predicts elastic-plastic behaviour from the very beginning of the deformation process, while the MCC model predicts only elastic behaviour within the initial yield surface.

For higher overconsolidation ratios (in the given analysis OCR>12), similar values of settlements are predicted for both models (Figure 13). The HASP model, due to the high value of hardening coefficient ω for high overconsolidation ratio, predicts small values of plastic strains and total values of strains are not much different from the values of elastic strains. For lower values of overconsolidation ratio, differences in strain magnitude are more pronounced. While the material described with the MCC model remains in the elastic zone for the given loads and for the lower values of the overconsolidation ratio also, the HASP model predicts higher values of plastic strains as the result of lower values of hardening coefficient ω. In the presented analysis, the differences in the consolidation settlements are up to 20-25%.



Slika 13. Zavisnost veličine sleganja od stepena prekonsolidacije, HASP model i MCC mod
Figure 13. Settlement dependency on the overconsolidation ratio, HASP model and MCC model

6 ZAKLJUČCI

HASP model uspešno prevazilazi mnoge nedostatke MCC modela prilikom opisivanja mehaničkog ponašanja prekonsolidovanih glina, a pri tome je zadržana jednostavnost MCC modela i isti broj parametara. Koristeći kombinovani zakon ojačanja u funkciji plastične zapreminske, plastične smičuće deformacije i parametar stanja, formulisan je koeficijent ojačanja koji kontroliše sve elemente ponašanja prekonsolidovane gline. Koeficijent ojačanja je istovremeno i koeficijent redukcije plastičnih deformacija, čime je omogućeno elasto-plastično ponašanje od samog početka deformisanja.

U dreniranim uslovima, model predviđa postepen prelaz iz kontrakcije u ekspanziju, pre nego što je dostignuta vršna smičuća čvrstoća, kao i postepen prelaz iz ojačanja u omekšanje, bez dodatnog matematičkog opisivanja. U nedreniranim uslovima, model predviđa putanju efektivnih napona "S" oblika, kao i negativan porni pritisak pri lomu za jako prekonsolidovane gline. Što je veća vrednost parametra stanja i veći stepen prekonsolidacije, veća je i vrednost koeficijenta ojačanja, te model predviđa veću krutost tla. Za normalno konsolidovane gline, HASP model automatski prelazi u MCC model, jer je tada koeficijent ojačanja jednak jedinici.

U postupku validacije modela, prikazani rezultati simulacije opita pri različitim putanjama totalnih napona, pokazuju veoma dobro slaganje sa eksperimentalnim rezultatima, za sve stepene prekonsolidacije. U poređenju s predviđanjem MCC modela, značajan napredak postignut je u sledećim elementima: a) HASP model predviđa postepen razvoj plastičnih deformacija od samog početka deformisanja; b) postoji postepen prelaz iz elastične u elasto-plastičnu oblast; c) postoji dobro predviđanje smičućeg napona pri lomu, kao i

6 CONCLUSIONS

The HASP model successfully overcomes many deficiencies of the MCC model when describing the mechanical behaviour of overconsolidated clays, while keeping the simplicity of the MCC model and the same number of parameters. By using the combined hardening rule in the function of plastic volumetric and shear strain and state parameter, the hardening coefficient has been formulated which controls all elements of the mechanical behaviour of overconsolidated clays. The hardening coefficient is at the same time the reduction coefficient for plastic strains, which allows elastic-plastic behaviour from the very beginning of deformation process.

In drained conditions, the model predicts gradual transition from contractive to dilatant behaviour before the peak strength is reached, as well as gradual transition from hardening to softening without additional mathematical description. In undrained conditions, the model predicts effective stress path of "S" shape, as well as negative failure pore pressure for highly overconsolidated clays. The higher the values of state parameter and overconsolidation ratio, higher the value of the hardening coefficient and the model predicts stiffer response. For normally consolidated clays, the HASP model automatically transforms into the MCC model, because the hardening coefficient equals one.

In the model validation process, the presented results of test simulations at different total stress paths are very well aligned with experimental results for all overconsolidation ratios. In comparison with the prediction of the MCC model, a significant progress was achieved in the following elements: a) the HASP model predicts gradual development of plastic strains from the very beginning of the deformation process; b) there is a pornog pritiska za prekonsolidovana tla.

HASP model je implementiran u Abaqus/Standard putem dostupnog korisničkog potprograma UMAT. Za numeričku integraciju konstitutivnih relacija, vrlo uspešno je primenjena Metoda vodećeg parametra.

U razmatranom primeru konsolidacije sloja zasićene prekonsolidovane gline usled fazne izgradnje nasipa, prikazana je sposobnost HASP modela da predvidi vremenski tok promene pornih pritisaka, zapreminskih i smičućih deformacija. Rezultati su poređeni s MCC modelom. Usled brzog opterećivanja sloja zasićene gline male vodopropusnosti, HASP model predviđa pojavu pornog natpritiska, koji raste tokom izgradnje nasipa, te u procesu konsolidacije dolazi do potpune disipacije pornog natpritiska. Deformacije se najbrže razvijaju tokom izgradnje nasipa, a najveća sleganja dobijena su za blago prekonsolidovane gline. U poređenju s MCC modelom, osnovna razlika jeste u veličini zapreminskih i smičućih deformacija. Koristeći HASP model, generalno se dobijaju veće vrednosti deformacija nego u MCC modelu, s obzirom na to što HASP model predviđa elasto-plastično ponašanje od samog početka procesa deformisanja, dok MCC model predviđa samo elastično ponašanje unutar inicijalne površi tečenja.

Na osnovu prikazanih rezultata, može se zaključiti da HASP model ima dobar balans između sofisticiranosti i jednostavnosti, što omogućava njegovu široku praktičnu primenu u rešavanju geotehničkih problema.

gradual transition from elastic into elastic-plastic region; c) there is good prediction of failure shear stress, as well as pore water pressure for overconsolidated soil.

The HASP model is implemented in Abaqus/Standard through the available user subroutine UMAT. For numerical integration of constitutive relations, the Governing Parameter Method was used very successfully.

Through the discussed example of consolidation of saturated overconsolidated clay layer, as the result of phased construction of the embankment, the ability of the HASP model to predict the changes of pore water pressure, volumetric and shear strains was presented. The results were compared with the MCC model. As the result of rapid increase of load on the saturated clay layer with low permeability, the HASP model predicts the increase of pore water pressure during the construction of the embankment and full dissipation of the pore water pressure in the process of consolidation. Strains develop most rapidly during the construction of the embankment and greatest amount of settlement were obtained for slightly overconsolidated clays. In comparison with the MCC model, the main difference is in the magnitude of the volumetric and shear strains. By using the HASP model, higher values of strains are generally obtained against the MCC model, since the HASP model predicts elastic-plastic behaviour from the very beginning of the deformation process, while the MCC model predicts only elastic behaviour within the initial yield surface.

Based on the presented results, it can be concluded that the HASP model has good balance of sophistication and simplicity, which allows its wide practical use in solving various geotechnical problems.

7 LITERATURA
REFERENCES

[1] ABAQUS. User manual Hibbit, Karlsson & Sorensen, Inc. 2011.

[2] Al-Tabbaa, A. & Wood, D.M. An experimentally based „bubble" model for clay. Numerical Models in Geomechanics NUMOG III (eds S Pietruszczak and GN Pande), Elsevier Applied Science 1989; 91–99.

[3] Banerjee, P.K. & Stipho, A.S. Elastoplastic model for undrained behavior of heavily overconsolidated clays. Int J Numer Anal Mech Geomech 1979; 3(1): 97–103.

[4] Been, K. & Jefferies, M.G. A state parameter for sands. Géotechnique 1985; 35(2): 99–112.

[5] Biarez, J. & Hicher, P.Y. Elementary Mechanics of Soil Behaviour. Saturated Remoulded Soils. A. A. Balkema, Rotterdam; 1994.

[6] Borja, R.I. & Lee, S.R. Cam clay plasticity, part I: Implicit integration of constitutive relations, Comput. Meth. Appl. Mech. Eng. 1990; Vol. 78, 49–72.

[7] Borja R.I. Cam clay plasticity, part II: Implicit integration of constitutive equations based on nonlinear elastic stress prediction, Comput. Meth. Appl. Mech. Eng. 1991; Vol. 88, 225–240.

[8] Dafalias, Y.F. & Popov, E.P. Plastic internal variables formalism of cyclic plasticity. J. of Applied Mechanics 1976; Vol. 98, 645–651.

[9] Dafalias, Y.F. & Herrmann, L.R. A bounding surface soil plasticity model. Proc. Int. Symp. on Soils under Cyclic and Transient Loading 1980; Vol. 1, 335–345.

[10] Dafalias Y.F., Manzari M.T., Papadimitriou A.G. SANICLAY: simple anisotropic clay plasticity model. International Journal for Numerical and Analytical Methods in Geomechanics 2006; 30: 1231–1257. DOI: 10.1002/nag. 524.

[11] Gajo, A. & Wood, D.M. A new approach to anisotropic, bounding surface plasticity: General formulation and simulations of natural and reconstituted clay behaviour. Int J Numer Analyt Meth Geomech 2001; 25-3: 207–241.

[12] Grammatikopoulou, A., Zdravkovic, L., Potts, D.M. General Formulation of Two Kinematic Hardening Constitutive Models with a Smooth Elastoplastic Transition. Int J Geomech 2006; 6:291–302. doi:10.1061/(ASCE) 1532–3641 (2006) 6:5 (291).

[13] Helwany, S. Applied soil mechanics with ABAQUS applications. John Willey & Sons, Canada, 2007.

[14] Hickman, R.J. & Gutierrez, M. An internally consistent integration method for critical state models. Int J Numer Anal Methods Geomech 2005; 29: 227–48.

[15] Houlsby, G.T., Wroth, C.P., Wood, D.M. Predictions of the results of laboratory tests on a clay using a critical state model. In Proc. int. workshop on constitutive behaviour of soils, Grenoble, Balkema (Rotterdam) 1982; 99–121.
[16] Iwan, W.D. On a class of models for the yield behaviour of continuous and composite systems. Journal of Applied Mechanics 1967, ASME 34; 612–617.
[17] Jefferies, M.G. Nor-Sand: a simple critical state model for sand. Géotechnique 1993; 43(1):91–103.
[18] Jocković, S. & Vukićević, M. Bounding surface model for overconsolidated clays with new state parameter formulation of hardening rule. Comput Geotech 2017; 83:16–29. doi:10.1016/j.compgeo.2016.10.013.
[19] Kojić, M. & Bathe K.J. The effective-stress-function algorithm for thermo-elastoplasticity and creep, Int. J. Num. Meth. Engng. 1987; Vol. 24, pp. 1509–1532.
[20] Kojić, M. The governing parameter method for implicit integration of viscoplastic constitutive relations for isotropic and orthotropic metals, Computational Mechanics 1996; Vol. 19, 49–57.
[21] Kojić, M. & Bathe, K.J. Inelastic Analysis of Solids and Structures, Berlin, Springer-Verlag, 2003.
[22] Krieg, R. D. A practical two-surface plasticity theory. Journal of Applied Mechanics 1975; Vol. 42, No. E3, pp. 641–646.
[23] Li, X.S. & Dafalias, Y.F. Dilatancy for cohesionless soils. Géotechnique 2000; 50:449–60. doi:10.1680/geot.2000.50.4.449.
[24] McDowell, G.R. & Hau, K.W. A simple non-associated three-surface kinematic hardening model. Géotechnique 2003, No. 4, 433‒437.
[25] Mita, K.A., Dasari, G.R., Lo, K.W. Performance of a three-dimensional Hvorslev-Modified Cam clay model for overconsolidated clay. ASCE, Int J Geomech 2004; 4(4): 296–309.
[26] Mróz, Z. On the description of anisotropic workhardening. Journal of the Mechanics and Physics of Solids 1967; 15, 163–175.
[27] Nayak, G.C., & Zienkiewicz, O.C. Elastoplastic stress analysis: a generalization for various constitutive relations including strain softening. International Journal for Numerical Methods in Engineering 1972; 5, 113–135.
[28] Nova, R. & Wood, D.M. A constitutive model for sand in triaxial compression. Int J Numer Anal Methods Geomech 1979; 3: 255–278.
[29] Nova, R. Modelling of bonded soils with unstable structure. International Workshop on Modern Trends in Geomechanics – Vienna Springer, 2006.
[30] Ortiz, M., Pinsky, P.M., Taylor, R.L. Operator split methods for the numerical solution of the elasto-plastic dynamic problem, Comp. Meth. Appl. Mech. Engng. 1983; Vol. 39, pp. 137–157.
[31] Ortiz, M. & Popov, E.P. Accuracy and stability of integration algorithms for elastoplastic constitutive relations. International Journal for Numerical Methods in Engineering 1985; 21:1561–1576.
[32] Ortiz, M. & Simo, J.C. An analysis of a new class of integration algorithms for elasto-plastic constitutive relations, Int. J. Num. Meth. Eng. 1986; Vol. 23, 353–366.
[33] Owen, D.R.J. & Hinton, E. Finite Elements in Plasticity: Theory and Practice 1980. Pineridge Press, Swansea, Wales, 594 p.
[34] Parry, R.H.G. Correspondence. Géotechnique 1958; 8:183–186.
[35] Roscoe, K.H., Schofield, A.N., Wroth, C.P. On the yielding of soils. Géotechnique 1958; 8, 22–53.
[36] Roscoe, K.H. & Burland, J.B. On the generalised stress-strain behaviour of ‘wet’ clay. In: Heyman J, Leckie FA, editors. Engineering plasticity 1968. Cambridge, UK: Cambridge University Press; 535–609.
[37] Schädlich, B. & Schweiger, H.F. Modelling the shear strength of overconsolidated clays with a Hvorslev surface. Geotechnik 2014; 37:47–56. doi: 10.1002/gete. 201300016.
[38] Schofield, A.N. & Wroth, C.P. Models and soil mechanics. Critical state soil mechanics. McGraw-Hill, London, New York, 1968; 20–21.
[39] Sheng, D.C., Sloan, S.W., Gens, A., Smith, D.W. Finite element formulation and algorithms for unsaturated soils. Part 1: Theory. International Journal for Numerical and Analytical Methods in Geomechanics 2003; 27(9):745–765.
[40] Sheng, D.C., Smith, D.W., Sloan, S.W., Gens, A. Finite element formulation and algorithms for unsaturated soils. Part 2: Verification and Applications. International Journal for Numerical and Analytical Methods in Geomechanics 2003; 27(9):767–790.
[41] Simo, J.C. & Taylor, R.L. Consistent tangent operators for rate independent elasto-plasticity, Comp. Meth. Appl. Mech. Engng. 1985; Vol. 48, pp. 101–118.
[42] Simo, J.C. & Hughes, T. Computational inelasticity. Interdisciplinary applied mathematics series. Springer, 1998.
[43] Sloan, S.W. Substepping schemes for the numerical integration of elastoplastic stress-strain relations. Int J Numer Methods Eng 1987; 24: 893–911.
[44] Sołowski, W.T. & Gallipoli, D. Explicit stress integration with error control for the Barcelona Basic Model. Part II: Algorithms efficiency and accuracy. Computers & Geotechnics 2010; 37(1-2):68–81. http://dx.doi.org/10.1016/j.compgeo. 2009.07.00.
[45] Stallebrass, S.E. & Taylor, R.N. The development of a constitutive model for the prediction of ground movements in overconsolidated clay. Géotechnique 1997; 47, 235 - 253.
[46] Tsiampousi, A., Zdravković, L., Potts, D.M. A new Hvorslev surface for critical state type unsaturated and saturated constitutive models. Comput Geotech 2013; 48:156–66. doi:10.1016/j.compgeo.2012.09.010.
[47] Vukićević, M. Governing parameter method for implicit stress integration of Modified Cam Clay model, using the mean stress as the governing parameter. XIV Danube – European Conference on Geotechnical Engineering, DECGE 2010, Bratislava 2010; 274-275.
[48] Whittle, A. & Kavvadas, M. Formulation of MIT-E3 constitutive model for overconsolidated clays. J. Geotech. Eng. 1994; 120 (1), 173–198.

[49] Wilkins, M. L. Calculation of elastic-plastic flow. Methods of Computational Physics. Academic Press, N. Y. 1964; Vol 3.
[50] Yao, Y.P., Hou, W., Zhou, A.N. UH model: three-dimensional unified hardening model for overconsolidated clays. Géotechnique 2009; 59:451–69. doi:10.1680/geot.2007.00029.
[51] Yu, H.S. CASM: A unified state parameter model for clay and sand. International Journal for Numerical and Analytical Methods in Geomechanics 1998; 22, No. 8: 621-653.
[52] Zienkiewicz, O.C. & Naylor, D.J. Finite element studies of soils and porous media. Lecture, finite elements, Oden and de Arantes (eds), UAH Press 1973; 459–493.

REZIME

VALIDACIJA I IMPLEMENTACIJA *HASP* KONSTITUTIVNOG MODELA ZA PREKONSOLIDOVANE GLINE

Sanja JOCKOVIĆ
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Za široku primenu konstitutivnih modela za tlo u savremenoj inženjerskoj praksi postoje dva bitna uslova: a) model treba dovoljno dobro da predviča ponašanje tla pri razlicitim putanjama napona; b) materijalne konstante modela mogu da se odrede iz standardnih opita. Uvažavaju´i oba uslova, formulisan je HASP model za opisivanje mehanickog ponašanja prekonsolidovanih glina, koriste´i teoriju kriticnog stanja i koncept granicne površi. HASP model, na jednostavan nacin, prevazilazi mnoge nedostatke Modifikovanog Cam Clay modela, bez uvočenja dodatnih materijalnih parametara. Formulacijom zakona ojacanja u funkciji parametra stanja i stepena prekonsolidacije, omogu´eno je opisivanje brojnih elemenata mehanickog ponašanja prekonsolidovanih glina. HASP model je implementiran u program Abaqus koriste´i Metodu vode´eg parametra za numericku integraciju konstitutivnih relacija. U radu je prikazana validacija HASP modela – poređenjem s publikovanim rezultatima triaksijalnih opita, kao i mogu´nosti modela da adekvatno predvidi ponašanje prekonsolidovanih glina putem analize granicnog (konturnog) problema metodom konacnih elemenata. Razmatran je problem konsolidacionog sleganja tla usled fazne izgradnje nasipa na površini zasi´ene prekonsolidovane gline, za razlicite stepene prekonsolidacije.

Ključne reči: konstitutivni model, prekonsolidovane gline, parametar stanja

SUMMARY

VALIDATION AND IMPLEMENTATION OF *HASP* CONSTITUTIVE MODEL FOR OVERCONSOLIDATED CLAYS

Sanja JOCKOVIC
MirjanaVUKICEVIC

There are two important conditions for the wide application of constitutive models for soil in contemporary engineering practice: a) the model should predict sufficiently well the soil behaviour at different stress paths; b) the material constants of the model can be determined from standard laboratory tests. Taking into account both conditions, a HASP model has been formulated to describe the mechanical behaviour of the overconsolidated clays, using the critical state theory and the boundary surface concept. The HASP model in a simple way overcomes many deficiencies of the Modified Cam Clay model, without introducing any additional material parameters. The formulation of the hardening rule in the function of the state parameter and overconsolidation ratio, allows the description of numerous elements of the mechanical behaviour of the overconsolidated clays. The HASP model has been implemented in software Abaqus using the Governing Parameter Method for the numerical integration of constitutive relations. The paper presents validation of the HASP model in comparison with the published results of triaxial tests as well as the possibilities of the model to adequately predict the behaviour of the overconsolidated clays through the analysis of the boundary value problem using the finite element method. The problem of the clay settlements due to phased construction of the embankment on the saturated clay surface was analyzed, assuming different overconsolidation ratios.

Key words: constitutive model, overconsolidated clays, state parameter

BOČNA NOSIVOST I POMERANJA VERTIKALNIH ŠIPOVA OPTEREĆENIH HORIZONTALNIM SILAMA

LATERAL CAPACITY AND DEFORMATIONS OF VERTICAL PILES LOADED BY HORIZONTAL FORCES

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1 UVOD

Temelji graČevinskih objekata fundiranih na šipovima uglavnom prenose vertikalno optereŒenje, a šipovi su optereŒeni aksijalnim silama pritiska/zatezanja [7]. MeČutim, ponekad su vertikalni šipovi optereŒeni i znacajnim horizontalnim silama koje mogu da budu posledica stalnog optereŒenja, ali i vetra i/ili zemljotresa. U takvim slucajevima, potrebno je da se odredi bocna nosivost vertikalnih šipova [13]. Ona je posledica horizontalnog pomeranja šipova i usled toga mobilisanja njihove cvrstoŒe i cvrstoŒe okolnog tla. ImajuĆi to u vidu, bocna otpornost šipova može da bude prekoracena s obzirom na:

– nosivost okolnog tla, što je tzv. geotehnicka nosivost;

– nosivost poprecnog preseka šipa, što je tzv. konstruktivna nosivost.

U ovom radu Œemo, pre svega, analizirati geotehnicku nosivost šipova i - saglasno tome - obradiŒemo sledeŒe metode: Rankinovu, Bromsovu i Brinc-Hansenovu. Osim toga, pokazaŒemo kako se mogu odrediti horizontalne deformacije bocno optereŒenih vertikalnih

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1 INTRODUCTION

Pile foundations are mostly loaded by vertical forces, which means that they are loaded by axial compression or tension forces [7]. However, in some cases vertical piles are loaded by high horizontal forces due to the dead loads, winds or earthquakes. In such cases it is necessary to determine lateral capacity of vertical piles which is due to the horizontal displacements of piles and therefore mobilized pile strength and the strength of surrounding soil [13]. So, the ultimate resistance of piles can be reached regarding

– ultimate capacity of surrounding soil i.e. geotechnical capacity

– ultimate capacity of pile cross section i.e. structural capacity.

In this paper, the geotechnical capacity of piles will be analyzed first and then, according to the findings the following methods will be presented: Rankine's, Broms' and Brinch-Hansen's methods. Afterwards, the following methods for determining horizontal deformations of vertical piles loaded by horizontal forces will be presented: applications of elastic theory, coefficient of

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šipova i to primenom teorije elastičnosti, primenom koeficijenta horizontalne krutosti tla ili p-y krivih. Ova problematika je vrlo složena i analizirali su je brojni autori, npr. Brinch-Hansen [15], Broms [2,3,4], Meyerhof and Ranjan [16], Meyerhof [17], Milović i Đogo [18, 19], Poulos and Davis [22], Reese and Van Impe [26].

Kada je reč o konstruktivnoj nosivosti šipova, ona se određuje na isti način kao kod armirano-betonskih stubova opterećenih na savijanje. U vezi sa tim naglašavamo da ako je vertikalni šip opterećen istovremeno horizontalnom i vertikalnom (aksijalnom) silom onda se proračun vrši tako što se uzima u obzir interakcija momenta savijanja i aksijalne sile.

2 BOČNA NOSIVOST POJEDINAČNOG ŠIPA

Određivanje bočne/horizontalne nosivosti vertikalnog šipa opterećenog horizontalnom silom složen je inženjerski problem koji je posledica interakcije šipa i okolnog tla [20, 21]. Ona zavisi od čvrstoće okolnog tla, krutosti i dužine šipa, kao i od načina oslanjanja njegove glave.

Stoga, za njeno određivanje, pre svega, potrebno je da se sprovedu adekvatna geotehnička istraživanja, terenska i laboratorijska, te na osnovu toga da se definiše geotehnički model terena na mestu budućeg objekta. A zatim, na tako definisanom modelu, radi se proračun bočne nosivosti šipova [12, 13, 24].

Prilikom određivanja bočne otpornosti tla oko šipa, po pravilu, čine se određena uprošćavanja, kako bi se dobilo rešenje koje je prihvatljivo za geotehničku praksu [6, 9, 16, 17]. Neka od ovih rešenja prikazaćemo u nastavku teksta.

2.1 Rankinova metoda

U geotehničkoj praksi (i ne samo našoj [11]), ovaj problem još uvek se tretira ravanski (ravna deformacija) i pretpostavlja se da se pomeranju šipa, od horizontalne sile H, suprotstavlja pasivni otpor tla (Slika 1), koji se može odrediti iz sledeće jednačine [25]:

subgrade reaction or p-y curves, too. This problem is very complex and has been analysed by many authors, e.g. Brinch-Hansen [15], Broms [2, 3, 4], Meyerhof and Ranjan [16], Meyerhof [17], Milović and Đogo [18, 19], Poulos and Davis [22], Reese and Van Impe [26].

Structural capacity of the piles is determined in the same way as for reinforced concrete columns loaded by bending moments. Following that, if the vertical pile is simultaneously loaded by horizontal and vertical (axial) forces than in calculation procedure has to be included interaction between bending moment and axial force.

2 LATERAL CAPACITY OF A SINGLE PILE

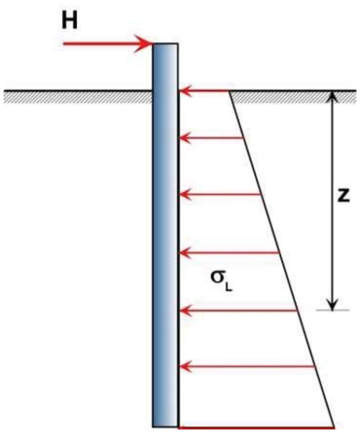
Determining the lateral/horizontal capacity of vertical piles loaded by horizontal forces is a complex problem which is the consequence of the interaction between pile and surrounding soil [20, 21]. The interaction depends on the strength of surrounding soil, the stiffness and the length of pile and its head support conditions.

Accordingly, at first, it is necessary to make adequate geotechnical investigations, in laboratory and in situ, and on the basis of that geotechnical model of terrain under the structure has to be defined. For such defined model, lateral capacity of piles has to be calculated [12. 13, 24].

Various simplifications are necessary for providing acceptable solutions for geotechnical practice [6, 9, 16, 17]. Some of these solutions will be presented in the following text.

2.1 Rankine's method

In geotechnical practice, not only in Serbia [11], this is treated as a plain strain problem using passive earth pressure theory. It is assumed that horizontal movements are restricted by passive resistance of the soil (Fig. 1) which can be determined using the following equation [25]:



Slika 1. Rankinova metoda
Figure 1. Rankin's method

$$\sigma_L = \sigma_V \cdot tg^2\left(45+\frac{\varphi}{2}\right) + 2\cdot c\cdot tg\left(45+\frac{\varphi}{2}\right) \tag{1}$$

gde je:

σ_L – bočni otpor tla na dubini z;
σ_V – vertikalni napon na dubini z;
c – kohezija;
φ – ugao unutrašnjeg trenja.

Sumiranjem horizontalnih napona – po dubini i prečniku/širini šipa – i rešavanjem jednačina ravnoteže koje definišu ponašanje šipa, dobija se granična horizontalna sila.

Međutim, ovakav način rada predstavlja konzervativni pristup određivanju bočne nosivosti šipova, jer se prostorni problem rešava ravanski. Na taj način, zanemaruje se uticaj treće dimenzije na veličinu bočnog otpora tla. Kao posledica toga, dobijaju se znatno manje sile bočnog otpora od onih koje okolno tlo može da prihvati.

where:

σ_L – lateral resistance of soil at depth z
σ_V – vertical stress at depth z
c - cohesion
φ – angle of internal friction

By summing the horizontal stresses over depth and diameter/width of a pile and by using equilibrium conditions which define the behaviour of a pile, the ultimate lateral force H_f should be determined.

This approach is, however, conservative because the three-dimensional problem is treated as it is two-dimensional one. In such a way the influence of the third dimension, on lateral force, is neglected. As a consequence, significantly lesser horizontal forces are obtained than the surrounding soil may withstand.

2.2 Bromsova metoda

Na osnovu rezultata terenskih opita, Broms je 1964. godine odredio bočnu nosivost vertikalnih šipova, fundiranih u homogenom koherentnom i nekoherentnom tlu [2, 3]. Pritom, kod koherentnog tla analizirao je samo slučaj nedreniranih terenskih uslova. Rezultati tih opita pokazali su da se bočni otpor tla σ_L može izračunati korišćenjem sledećih jednačina:

2.2 Broms' method

Broms (1964) was determined, on the basis of in situ test data, lateral capacity of vertical piles which are founded in homogeneous cohesive and cohesionless soils [2, 3]. However, in cohesive soil only the undrained case was analysed. The results of these tests have shown that lateral resistance of soil σ_L can be expressed by the following equations:

koherentno tlo: / cohesive soil:

$$\sigma_L = 9\cdot c_u \tag{2}$$

nekoherentno tlo: / cohesionless soil:

$$\sigma_L = 3\cdot\gamma\cdot z\cdot k_p \tag{3}$$

gde je:

c_u – nedrenirana kohezija;
γ – zapreminska težina;
$k_p = tg^2(45+\varphi/2)$ - koeficijent pasivnog pritiska;
φ – ugao unutrašnjeg trenja;
z – dubina na kojoj se traži bočni otpor.

Jednačine (2) i (3) uključuju trodimenzionalne uslove tla oko šipa.

Broms je u svojim radovima (1964, 1965) analizirao kratke (krute) i dugačke (fleksibilne) šipove (Slika 2) [2, 3, 4]. Pri tome:

- kod kratkih šipova maksimalno horizontalno opterećenje H_f, koje može da se nanese na šip, ograničeno je maksimalnim horizontalnim otporom koji može da mobiliše tlo oko šipa;
- kod dugačkih šipova maksimalno horizontalno opterećenje H_f, koje može da se nanese na šip, ograničeno je momentom savijanja koji šip može da prihvati.

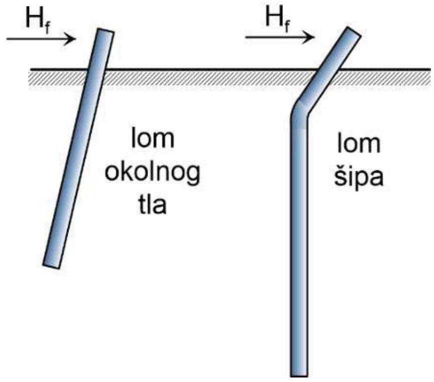
where:

c_u – undrained cohesion
γ – unit weight of soil
k_p – $tg^2(45+\varphi/2)$ – coefficient of passive resistance
φ – angle of internal friction
z – vertical distance from the ground surface to the location of lateral stress

The equations (2) and (3) included three-dimensional conditions of the soil surrounding the loaded pile.

In his papers (1964, 1965) Broms has analysed short (stiff) and long (flexible) piles (Fig. 2) [2, 3, 4]. So,

- for short piles, ultimate horizontal force H_f is limited by ultimate lateral resistance of the surrounding soil
- for long piles, ultimate horizontal force H_f is limited by yield moment of pile cross-section.

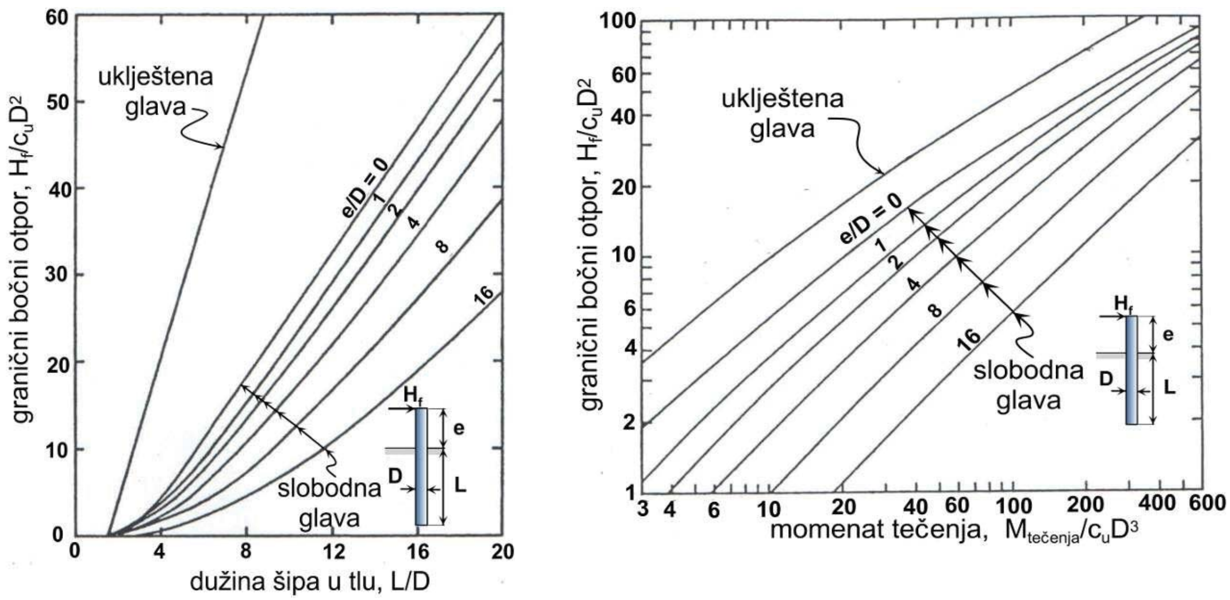


a) b)

Slika 2. Lom šipa opterećenog horizontalnom silom a) kratki šip; b) dugački šip
Figure 2. Soil/pile fails loaded by horizontal force a) short pile b) long pile

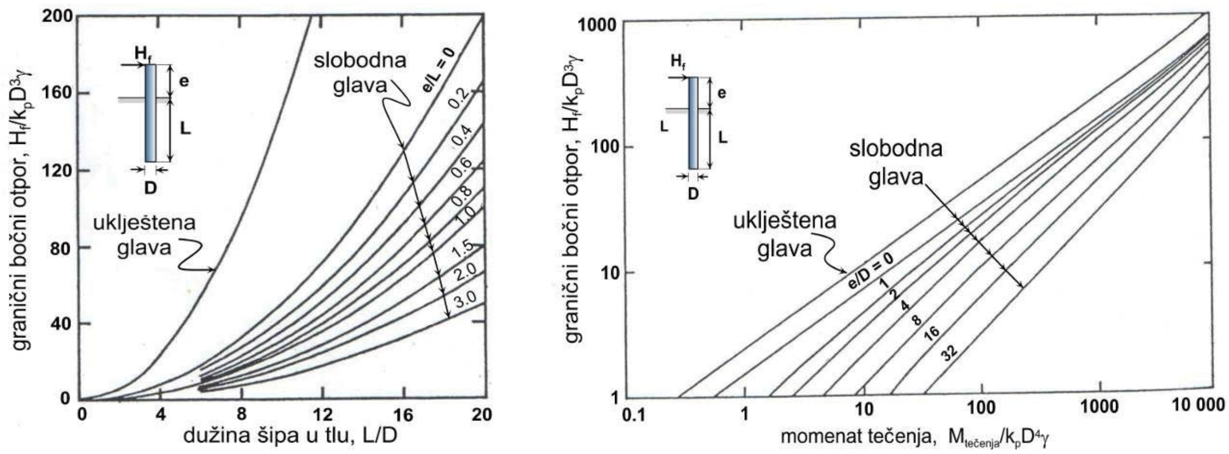
Broms je definisao načine loma i dijagrame otpornih sila koje deluju na vertikalne šipove – kako one sa slobodnom, tako i one sa uklještenom glavom. Na osnovu toga, postavljanjem odgovarajućih uslova ravnoteže, dobijaju se granične horizontalne sile. Dobijena rešenja Broms je prikazao i grafički – dijagramima na osnovu kojih se lako mogu odrediti granične horizontalne sile H_f za kratke i dugačke šipove, i u koherentnom, a i u nekoherentnom tlu (Slike 3 i 4).

For short and long vertical piles, Broms has defined the failure mechanisms and the values of lateral earth pressures. He did it for free-headed piles and for piles with restrained head as well. Therefore, ultimate lateral forces H_f were obtained from the equilibrium considerations. These values Broms presented graphically at Fig. 3 and 4.



a) kratki šip (*short pile*) b) dugački šip (*long pile*)

Slika 3. Granični bočni otpor šipova u koherentnom tlu (Broms, 1964)
Figure 3. Ultimate lateral resistance of piles in cohesion soils (Broms, 1964)



a) kratki šip (*short pile*) b) dugački šip (*long pile*)

Slika 4. Granični bočni otpor šipova u nekoherentnom tlu (Broms, 1964)
Figure 4. Ultimate lateral resistance of piles in cohesionless soils (Broms, 1964)

Na slikama 3 i 4: D je prečnik šipa; L – dubina ukopavanja; $M_{tečenja}$ – moment savijanja koji izaziva tečenje/lom poprečnog preseka šipa.

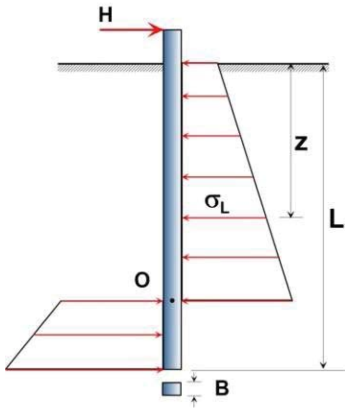
In Fig. 3 and 4 is noted: D – diameter of pile; L – length of embedment; M_{yield} – yield moment of pile cross-section.

2.3 Brinč-Hansenova metoda

Brinč-Hansen (1961) predložio je metodu za određivanje bočne otpornosti tla u slučaju vertikalnog šipa, širine B i dubine ukopavanja L, opterećenog horizontalnom silom H (Slika 5) [15].

2.3 Brinch-Hansen's method

Brinch-Hansen (1961) has presented the method for determination of ultimate lateral resistance of the soil surrounding the short vertical piles loaded by horizontal force H (Fig. 5) [15].



Slika 5. Brinč-Hansenova metoda
Figure 5. Brinch Hansen's method

Ova metoda odnosi se na kratke - krute šipove koji se pod dejstvom sile H rotiraju oko tačke O. Bočni pritisci σ_L uzimaju u obzir trodimenzionalne uslove u kojima se šip nalazi i predstavljaju razliku između bočnih pritisaka ispred i iza šipa. Veličina tako definisanih bočnih pritisaka, određuje se iz sledeće jednačine:

In the state of failure pile rotates, as a rigid body, about a point O. Lateral pressures σ_L take into consideration three-dimensional conditions of surrounding soil and they are resultant of pressures i.e. passive minus active pressures. So defined lateral pressures σ_L can be determined from the following equation:

$$\sigma_L = q \cdot k_q + c \cdot k_c \tag{4}$$

gde je:

σ_L – bočni pritisak na dubini z;
q = σ_V – vertikalni napon na dubini z;

where:

σ_L– lateral pressure at depth z
q = σ_V – vertical stress at depth z

c – kohezija;
k_q i k_c – koeficijenti bočnog pritiska tla.

Veličina koeficijenata k_q i k_c određuje se iz dijagrama datih na slikama 6 i 7. Na tim dijagramima φ je ugao unutrašnjeg trenja.

Veličina granične horizontalne sile H_f – koja deluje na šip (Slika 8) – određuje se rešavanjem sledećih jednačina ravnoteže:

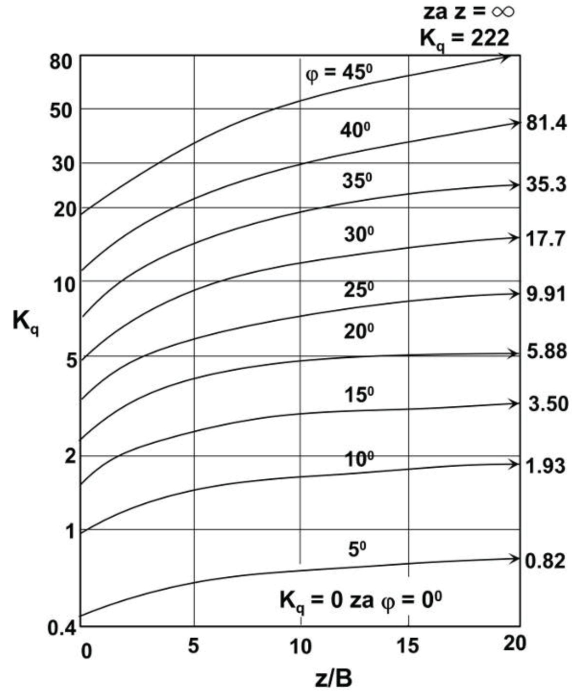
c – cohesion
k_q and k_c – coefficients of lateral pressures of soil

Coefficients k_q and k_c may be determined from curves given in Fig. 6 and 7. In these figures φ is the angle of internal friction.

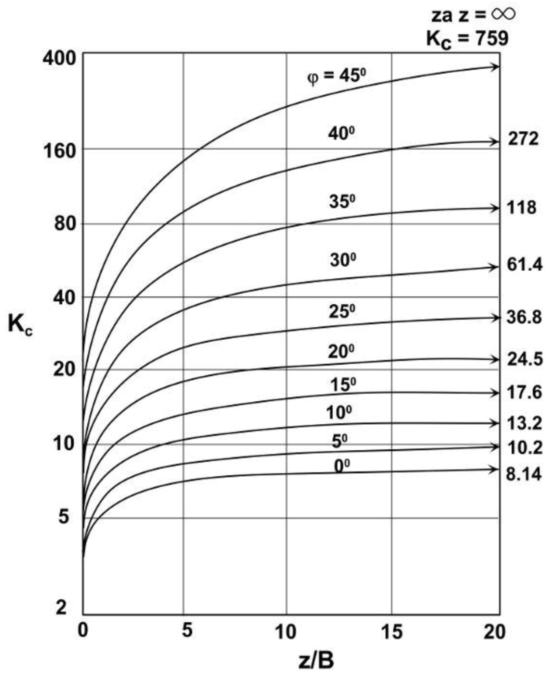
The ultimate horizontal force H_f (Fig. 8) is determined by means of following equilibrium conditions:

$$F_1 \cdot L_1 = F_2 \cdot L_2 \quad (5)$$

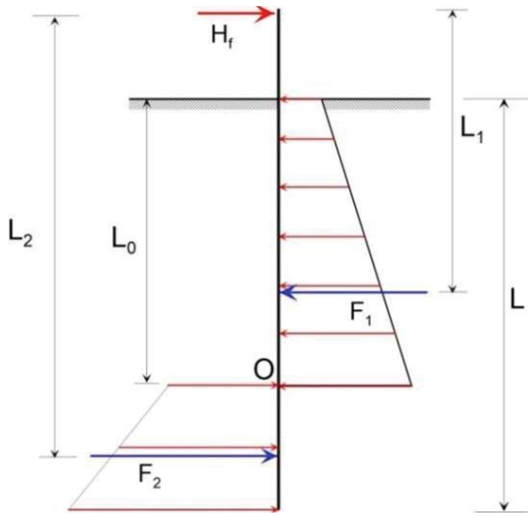
$$H_f = F_1 - F_2 \quad (6)$$



Slika 6. Koeficijent bočnog pritiska koji zavisi od vertikalnog napona (Brinch-Hansen, 1961)
Figure 6. Coefficient of lateral pressure which is dependent of vertical stress (Brinch-Hansen, 1961)



Slika 7. Koeficijent bočnog pritiska koji zavisi od kohezije (Brinch-Hansen, 1961)
Figure 7. Coefficient of lateral pressure which is dependent of cohesion (Brinch-Hansen, 1961)



Slika 8. Geotehnička nosivost šipa opterećenog horizontalnom silom
Figure 8. Geotechnical capacity of vertical pile loaded by horizontal force

2.4 Dozvoljeno bočno opterećenje

U poglavljima 2.1, 2.2 i 2.3 prikazani su postupci određivanja granične nosivosti pojedinačnog vertikalnog šipa, opterećenog horizontalnom silom. Pritom, za dobijanje dozvoljenog bočnog/horizontalnog opterećenja H_a, potrebno je da se njegova nosivost H_f redukuje faktorom sigurnosti F_s, tj.

2.4 Allowable lateral capacity

In Chapters 2.1, 2.2 and 2.3 are presented the procedures for determining bearing capacity for single vertical pile loaded by horizontal force. Accordingly, for determining allowable lateral/horizontal force H_a, it is necessary to reduce H_f by safety factor F_s i.e.

$$H_a = \frac{H_f}{F_s} \tag{7}$$

Veličina faktora sigurnosti kreće se između F_s = 2 i 3.

Napominjemo i to da ukoliko je konstruktivna nosivost šipa manja od njegove geotehničke nosivosti, onda je ona merodavna za određivanje horizontalne sile koju vertikalni šip može da prihvati.

Horizontalna sila H – koja deluje na šip – mora da bude manja od dozvoljene sile H_a. Osim toga, horizontalna pomeranja šipa treba da budu u dozvoljenim granicama.

The value of safety factor is between 2 and 3.

It is obvious that, if the structural capacity of a pile is less than geotechnical capacity of a pile, then it is proper for calculating allowable horizontal force of a pile.

Horizontal designed value H has to be less than allowable force H_a. Besides, lateral deformations of a pile have to be in allowable range.

2.5 Komentar

Uvažavajući sve što je rečeno u poglavlju 2, smatramo da Brinč-Hansenova metoda ima prednost u odnosu na druga dva prikazana postupka. Naime, ona uključuje trodimenzionalne uslove koji vladaju u tlu oko šipa, a može da se primeni u homogenom i heterogenom tlu i to u dreniranim ali i nedreniranim uslovima [29, 30]. Pri tome, ona je vrlo jednostavna za primenu, čak i u veoma složenim geotehničkim uslovima koji su često izraženi u Srbiji. To je veoma značajno prilikom fundiranja objekata, kao i prilikom sanacije klizišta [7, 8].

2.5 Comment

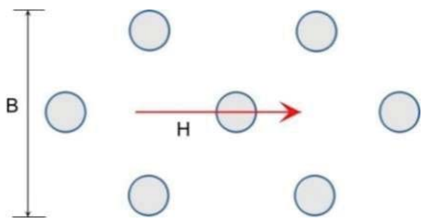
In accordance with Chapter 2, Brinch-Hansen's method has a priority over the other two presented methods. Namely, it involves three-dimensional conditions of surrounding soil around the pile. Besides, it can be applied in homogenous and heterogeneous soils, in drained or undrained conditions, too [29, 30]. Moreover, it is very simple for application even in very complex geotechnical conditions which are very often in Serbia. This is highly important for foundation of structures and landslide's remedial measures, too [7, 8].

3 BOČNA NOSIVOST GRUPE ŠIPOVA

Šipovi u temeljima nikad ne dolaze pojedinačno, već kao grupa šipova koja je povezana krutom temeljnom stopom. Stoga, prilikom proračuna bočne nosivosti šipova potrebno je da se ima u vidu i njihov grupni efekat [14]. S tim u vezi, Brinč-Hansen predlaže da se, prilikom proračuna, kao ekvivalentna širina B, usvoji ukupna širina grupe šipova - upravna na pravac sile H (Slika 9) [15].

3 LATERAL BEARING CAPACITY OF A PILE GROUP

In the foundation structure, piles are unlikely installed as single ones, but as group of piles which are jointed by stiff foundation cap. Therefore, calculation procedure should take into account their group effects [14]. Accordingly, Brinch-Hansen suggested that an equivalent width B has to be the width of a group perpendicular to the direction of the force H (Fig. 9) [15].



Slika 9. Ekvivalentna širina grupe vertikalnih šipova
Figure 9. Equivalent width for group of vertical piles

Treba reći da Poulos and Davis (1980) predlažu da se bočna nosivost grupe šipova odredi kao manja od sledeće dve vrednosti [22]:

– zbira bočne nosivosti pojedinačnih šipova;

In estimating the lateral bearing capacity of a pile group Poulos and Davis (1980) suggested the lesser of the following two values [22]:

– the sum of the lateral capacity of single piles

– bočne nosivosti ekvivalentnog temeljnog bloka koji obuhvata šipove i tlo između njih.

Dozvoljeno horizontalno opterećenje grupe šipova određuje se na isti način kao i u slučaju pojedinačnih šipova, odnosno redukcijom graničnog opterećenja.

– the lateral ultimate capacity of an equivalent single block containing the piles in the group and the soil between them.

The allowable lateral bearing capacity of a pile group determination is the same as for single piles i.e. by reduction of lateral ultimate capacity with safety factor.

4 POMERANJA BOČNO OPTEREĆENIH ŠIPOVA

Prilikom projektovanja temelja na šipovima, osim bočne nosivosti šipova, treba prvenstveno da se odrede i horizontalna pomeranja glave šipova i da se proveri da li su ona, za projektovano opterećenje, u dozvoljenim granicama [18, 19]. Ta pomeranja mogu da se odrede primenom teorije elastičnosti, pomoću koeficijenta horizontalne krutosti tla ili korišćenjem p-y krivih. Ovo ćemo obraditi u nastavku teksta.

4 DEFORMATIONS OF LATERALLY LOADED PILES

For designing pile foundations, not only lateral bearing capacity but the horizontal displacements have to be determined, too. They have to be, for designed loads, in allowable limits [18, 19]. Lateral deformations of piles have to be estimated by elastic analysis, by application the concept of coefficient of subgrade reaction or by use p-y curves. These will be presented in the following text.

4.1 Elastična analiza

Deformacije bočno opterećenog šipa u homogenom tlu, koje se može definisati kao linearno elastična sredina, mogu se odrediti primenom teorije elastičnosti [5]. Poulos and Davis (1980) horizontalno pomeranje ρ i rotaciju θ šipa na površini terena (tačka A), usled dejstva horizontalne sile H koja deluje na visini e iznad površine terena (Slika 10), definisali su sledećim jednačinama [22]:

4.1 Elastic analysis

On the basis of Theory of elasticity, Poulos and Davis (1980) presented solutions for lateral deflections of a single free-head pile within a linear-elastic uniform continuum [5]. The vertical pile is loaded by horizontal force H acting at a distance e above ground line (Fig. 10). Ground line displacement ρ and ground line rotation θ (point A at Fig. 10) are expressed as [22]:

$$\rho = \frac{H}{E_s \cdot L} \cdot \left(I_{\rho H} + \frac{e}{L} \cdot I_{\rho M} \right) \tag{8}$$

$$\theta = \frac{H}{E_s \cdot L^2} \cdot \left(I_{\theta H} + \frac{e}{L} \cdot I_{\theta M} \right) \tag{9}$$

gde je:

E_s – modul elastičnosti tla;

L – dubina ukopavanja vertikalnog šipa;

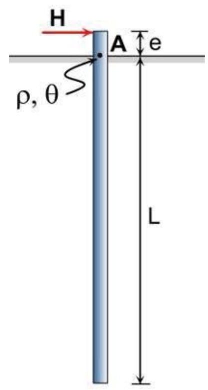
$I_{\rho H}$, $I_{\theta H}$, $I_{\rho M}$, $I_{\theta M}$ – uticajni faktori.

where:

E_s – modulus of elasticity of soil

L – embedment length

$I_{\rho H}$, $I_{\rho M}$, $I_{\theta H}$, $I_{\theta M}$– influence factors



Slika 10. Rešenje Poulos-a i Davis-a (1980)
Figure 10. Poulos and Davis solution (1980)

Vrednosti uticajnih faktora $I_{\rho H}$, $I_{\theta H}$, $I_{\rho M}$, $I_{\theta M}$ određuju se iz dijagrama datih na slikama 11, 12 i 13. Na tim slikama, vidi se da vrednost Poasonovog koeficijenta tla jeste ν = 0.5, a veličine uticajnih faktora zavise od faktora savitljivosti šipa K_R

Values of influence factors $I_{\rho H}$, $I_{\theta H}$, $I_{\rho M}$, $I_{\theta M}$ are given in Fig. 11, 12 and 13, and the Poisson's ratio of soil is ν = 0.5. From presented figures it is obvious that values of influence factors are functions of pile flexibility factor K_R

$$K_R = \frac{E_p \cdot I_p}{E_s \cdot L^4} \quad (10)$$

gde je:

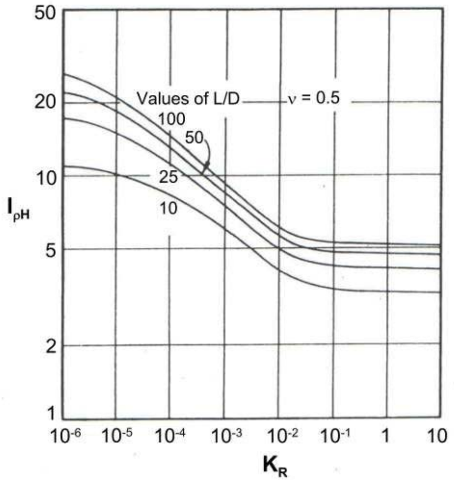
E_p – modul elastičnosti šipa;

I_p – momenat inercije šipa.

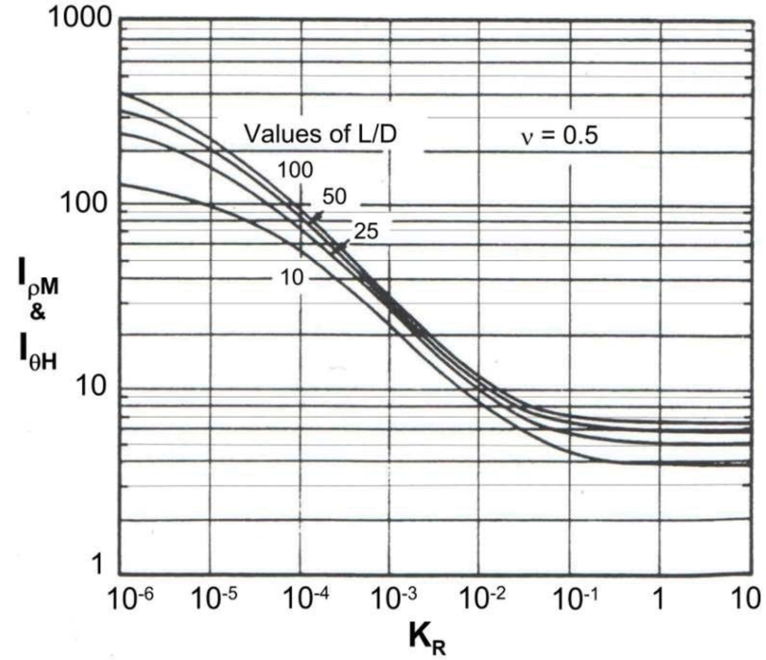
where:

E_p – modulus of elasticity of pile

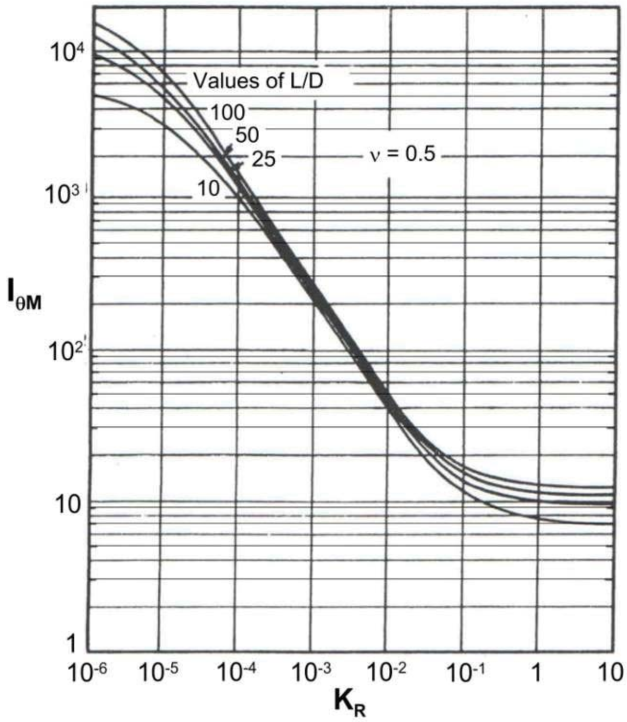
I_p – moment of inertia of pile



Slika 11. Vrednosti $I_{\rho H}$ (Poulos and Davis, 1980)
Figure 11. Values of $I_{\rho H}$ (Poulos and Davis, 1980)



Slika 12. Vrednosti $I_{\rho M}$ i $I_{\theta H}$ (Poulos and Davis, 1980)
Figure 12. Values of $I_{\rho M}$ and $I_{\theta H}$ (Poulos and Davis, 1980)



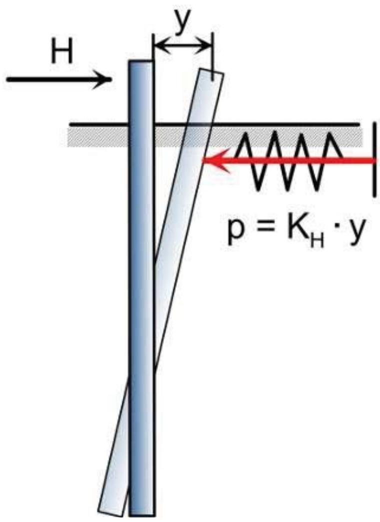
Slika 13. Vrednosti $I_{\theta M}$ (Poulos and Davis, 1980)
Figure 13. Values of $I_{\theta M}$ (Poulos and Davis, 1980)

4.2 Primena koeficijenta krutosti tla

Bočna pomeranja šipa, usled dejstva horizontalne sile H (Slika 14), najčešće se sračunavaju pomoću koeficijenta horizontalne krutosti (reakcije) tla [21]

4.2 Application of subgrade reaction coefficient

Lateral deformation of vertical pile, loaded by a horizontal force H (Fig. 14), may be estimated by coefficient of horizontal subgrade reaction of a soil K_H [21]



Slika 14. Horizontalno pomeranje šipa
Figure 14. Horizontal displacement of a pile

$$K_H = \frac{p}{y} \tag{11}$$

gde je:

K_H – koeficijent horizontalne krutosti tla;

p – bočni pritisak na mestu gde je pomeranje šipa jednako y;

y – horizontalno pomeranje šipa.

Koeficijent K_H ne zavisi samo od vrste tla i njegovih deformacionih karakteristika već i od prečnika/širine šipa [7].

U postupku proračuna tlo se zamenjuje serijom linearno-elastičnih opruga, s tim što se krutost svake opruge izražava koeficijentom horizontalne krutosti. Pri tome se usvaja da je njegova vrednost za koherentno tlo konstantna po dubini, a da se za nekoherentno tlo ona linearno povećava s dubinom.

Navedeni postupak proračuna deformacija u Srbiji koristi se u kompjuterskom programu TOWER.

Vrednosti koeficijenta horizontalne krutosti tla mogu da se odrede na sledeći način:

a) nekoherentna tla

Za nekoherentno tlo, K_H se određuje iz sledeće jednačine [28]:

where:

K_H – coefficient of horizontal subgrade reaction of soil

p – lateral pressure at point where the displacement of a pile is y

y – horizontal displacement of a pile.

Coefficient K_H is dependent not only of soil type and its deformation properties but on diameter/width of a laterally loaded pile, too [7].

In calculation procedure it is assumed that the soil around a pile can be replaced by the series of horizontal linear-elastic springs and the stiffness of each spring is expresses by its coefficient of subgrade reaction. It has been assumed that its value increases linearly with depth in the case of cohesionless soils and that it is constant with depth for cohesive soils.

In Serbia this concept is incorporated in computer program TOWER.

The values of coefficient of horizontal subgrade reaction may be estimated by the following procedures:

a) cohesionless soil

In cohesionless soil K_H is [28]:

$$K_H = n_h \cdot \frac{z}{D} \tag{12}$$

gde je:

n_h – koeficijent koji zavisi od gustine tla (Tabela 1);

z – dubina ispod površine terena;

D – prečnik/širina šipa.

where:

n_h – coefficient related to soil density (Table 1)

z – depth below ground surface

D – pile diameter/width

Tabela 1.Vrednosti koeficijenta n_h za nekoherentno tlo

Table 1. Values of n_h for cohesionless soils (Terzaghi, 1955)

zbijenost tla *Soil compaction condition*	n_h (kN/m^3)	
	iznad NPV *above groundwater*	ispod NPV *below groundwater*
rastresito / *loose*	2200	1300
srednje zbijeno / *compact*	6600	4400
zbijeno / *dense*	18000	11000

NPV – nivo podzemne vode

b) koherentna tla

U našoj geotehničkoj praksi, za koherentno tlo, često se koeficijent horizontalne krutosti tla K_H određuje pomoću sledeće jednačine [32]

b) cohesive soil

In Serbian geotechnical practice, for cohesive soil, the value of k_H is estimate, very often, from the following equation [32]:

$$K_H = 0.65 \cdot \sqrt[12]{\frac{E_s \cdot D^4}{E_p \cdot I_p}} \cdot \frac{E_s}{B \cdot (1-\nu^2)} \tag{13}$$

gde je:

E_s – modul elastičnosti tla;

E_p – modul elastičnosti šipa;

ν – Poasonov koeficijent tla;

D – širina/prečnik šipa;

I_p – momenat inercije šipa.

Ova jednačina može da se koristi i za određivanje K_H za nekoherentna tla [1].

Inače, u slučaju nedreniranih uslova u tlu, koristi se i sledeća jednačina [10]

where:

E_s – modulus of elasticity of soil

E_p – modulus of elasticity of pile

ν – Poisson's ratio of soil

B – pile diameter/width

I_p – modulus of inertia of pile

This equation may be used for estimation of K_H in cohesionless soils, too [1].

In the case of undrained conditions in soil, k_H may be estimated as [10]

$$K_H = \frac{67 \cdot S_u}{D} \quad (14)$$

gde je:
S_u – nedrenirana čvrstoća smicanja tla;
D – prečnik šipa.

c) komentar

Na kraju, posebno naglašavamo da vrednosti K_H izračunate u ovom poglavlju treba duplirati prilikom projektovanja šipova [1]. To je posledica znatnog otpora smicanja između šipa i okolnog tla (Slika 15) [27].

where:
S_u – undrained shear strength of the soil
D – pile diameter/width

c) comment

Finally, it has to emphasize that the values of K_H, estimated in this Chapter, should be doubled for pile design [1]. This is a consequence of considerable side shear resistance between pile and surrounding soil (Fig. 15) [27].

Slika 15. Otpor tla kod bočno opterećenog šipa (Smith, 1989)
Figure 15. Soil resistance to a lateral pile load (Smith, 1989)

4.3 Koncept p-y krive

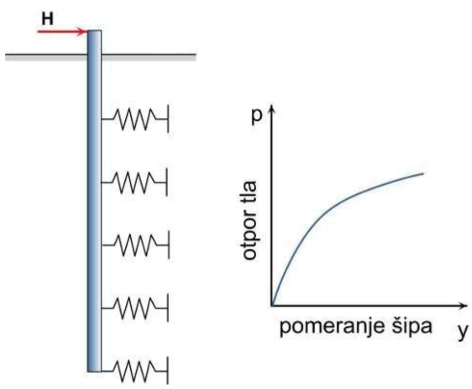
Ovom metodom se tlo oko šipa prikazuje serijom nelinearnih opruga, s tim što svaka opruga definiše zavisnost između bočnog otpora tla p i njegovog bočnog pomeranja y – na određenoj dubini ispod površine terena. Ta zavisnost određena je p-y krivama (Slika 16) [26, 31].

Ukoliko je tlo oko šipa višeslojno, onda se p-y krive određuju posebno za svaki sloj. One se mogu odrediti na osnovu rezultata laboratorijskih ili terenskih opita. Za brojna tla p-y krive već su određene i uključene u odgovarajuće kompjuterske programe (npr. LPILE) [26]. Na osnovu toga mogu da se dobiju horizontalna pomeranja šipova.

4.3 Concept of p-y curve

In this method the surrounding soil is simulated by using series of nonlinear horizontal springs. The each spring represents the relationship between horizontal soil resistance p and horizontal displacement y – at the particular depth under the ground line. This relationship is defined by p-y curve (Fig. 16) [26, 31].

If the surrounding soil is heterogeneous, than p-y curve has to estimate for each layer of soil. These curves may be determined by the results of laboratory or in situ tests. For different soils they had been already determined and were incorporated into adequate computer programs (e.g. LPILE) [26]. Based on that, horizontal displacements of piles can be obtained.



Slika 16. Koncept p-y krive
Figure 16. Concept p-y curve

5 BOČNA NOSIVOST I POMERANJA ŠIPOVA ZA SILOS KLINKERA U BEOČINU

Objekti za skladištenje klinkera, u okviru fabrike cementa „Lafarge” B.F.C. Beočin, sadrže tri vertikalna silosa. Dva silosa klinkera izgrađena su ranije i imaju kapacitet od 35.000 t, dok treći silos ima kapacitet od 50.000 t. Za potrebe izgradnje ovog trećeg silosa, „Hidrozavod DTD” iz Novog Sada izveo je geotehnička istraživanja terena (četiri istražne bušotine SB-1 do SB-4 dubine od po 15 m, iz kojih je uzeto i laboratorijski ispitano 30 uzoraka tla).

Na osnovu obavljenih istraživanja, teren na lokaciji silosa raščlanjen je na tri sredine: dobro do loše granulisane srednje zbijene peskove i dobro granulisane šljunkovite srednje zbijene peskove debljine od 6.0 do 7.0 m (SW/SP i GW/SW), loše granulisane peskove s proslojcima šljunka, debljine 7.0-8.0 m (SP, SP/GW), dok su na dubini od oko 14 m utvrđeni lapori. Kako su fizičko-mehaničke karakteristike prva dva sloja vrlo slične, formiran je pojednostavljeni geotehnički model terena, koji je poslužio da se uradi i numerička analiza geotehničke nosivosti i pomeranja bočno opterećenih šipova (Slika 17) [23, 24].

Intenzitet horizontalne sile, koju može da prihvati betonski šip prečnika D = 0.90 m i dužine L=10m, odredićemo primenom Brinč-Hansenove metode. Vrednosti bočnih pritisaka σ_L po 1 m prečnika šipa, prikazane su u Tabeli 2 i na Slici 17.

5 LATERAL BEARING CAPACITY AND HORIZONTAL DISPLACEMENTS OF PILES FOR CLINKER BIN IN BEOCIN

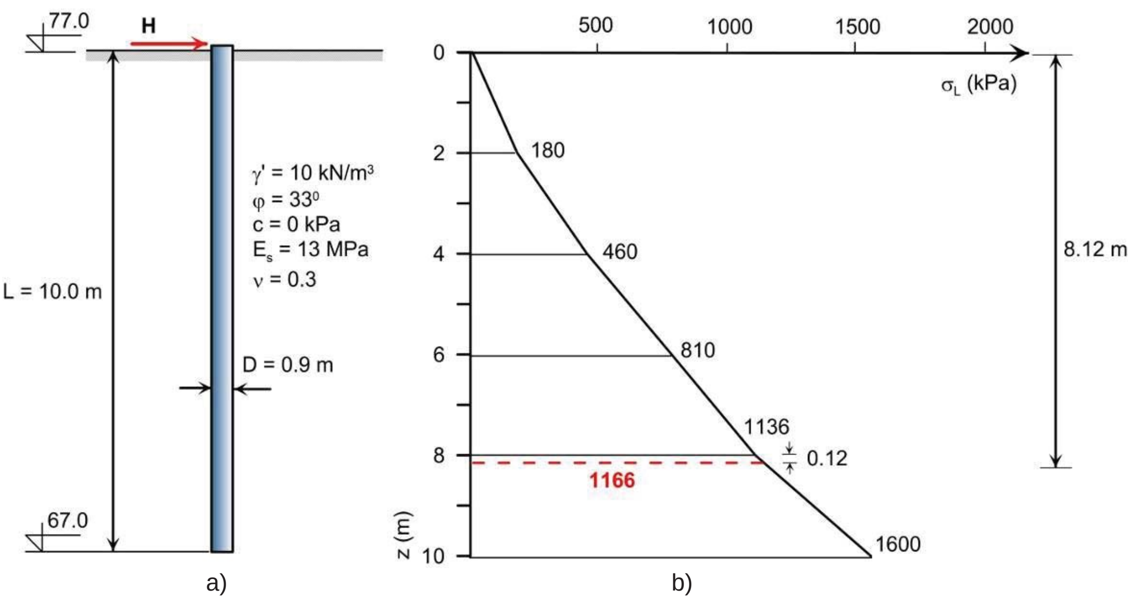
There are three vertical cement bins for binning clinkers in the area of cement factory “Lafarge” BFC in Beocin. Two cement bins, already, have been constructed and have the capacity of 35000 t each. A third one has the capacity of 50000 t. For constructing the third one, “Hidrozavod DTD” from Novi Sad made geotechnical investigations of terrain (4 boreholes SB-1 to SB-4 with depths of 15 m; from these boreholes 30 samples of soil were tested in laboratory).

On the basis of geotechnical investigations, the terrain under the third bin is divided in three layers: well to weak grained sands and compact well grained sandy-gravels with depths of 6,0-7,0 m (SW/SP and GW/SW), weak ground sands with interbeds of gravels, depths of 7,0-8,0 m(SP, SP/GW). At the depth of about 14,0 m there are marls. As the physical-mechanical properties of two upper layers are very similar, simplified geotechnical model of terrain was created. Numerical analysis was made in it for the calculation of geotechnical capacity and horizontal displacement of laterally loaded piles (Fig. 17) [23, 24].

Horizontal force intensity, which can be sustained by a pile with a diameter D = 0,90 m and length L = 10 m, will be determined by using Brinch-Hansen’s method. Values of laterally pressures σ_L for diameter of pile 1,0 m are presented in Table 2 and in Fig. 17.

Tabela 2. Bočni pritisci na šip
Table 2. Lateral pressures for pile

z (m)	0	2	4	6	8	10
z/D	0	2.2	4.4	6.7	8.9	11.1
k_q	0	9	11.5	13.5	14.2	16
q (kPa)	0	20	40	60	80	100
σ_L=q·k_q (kPa)	0	180	460	810	1136	1600



Slika 17. a) geotehnički model terena; b) dijagram bočnih pritisaka na šip
Figure 17. a) geotechnical model of terrain; b) lateral pressure diagram for pile

Rešavanjem jednačine (5) određuje se položaj centra rotacije šipa, odnosno dužina L_0. U našem slučaju je L_0 = 8.12 m. Tako da je F_1 = 3762 kN, a F_2 = 2332 kN. Iz uslova ravnoteže horizontalnih sila (jednačina 6) određuje se granična horizontalna sila H_f = 1430 kN. Ako usvojimo da je F_s = 2.5 onda je dozvoljena horizontalna sila H_a = 572 kN. Ona je višestruko veća od stvarne horizontalne sile koja deluje na šip i iznosi 166 kN. Napominjemo da se primenom Bromsove metode dobija H_f = 1525 kN i H_a = 610 kN [8].

Horizontalno pomeranje glave šipa ρ, određeno je primenom teorije elastičnosti (jednačina 8). Usvojeno je da je modul elastičnosti šipa E_p = 30 000 MPa, tako da je krutost šipa K_R = 0.00743 a $I_{\rho H}$ = 4.8. Delovanje horizontalne sile H = 166 kN izaziva horizontalno pomeranje glave šipa ρ = 6.13 mm.

Horizontalna pomeranja, bočno opterećenog šipa, odredićemo i pomoću koeficijenta horizontalne krutosti okolnog tla. Na slikama 18 i 19 prikazan je K_H koncept za numeričku analizu bočno opterećenog šipa. U postupku proračuna uzeta je u obzir smičuća otpornost između šipa i tla i stoga su duplirane vrednosti n_h iz Tabele 1 tj. n_h = 2x4400 = 8800 kN/m^3, kao i vrednosti K_H iz jednačine 13 tj. K_H = 2 x 12520 = 25040 kN/m^3. Numerička analiza urađena je primenom kompjuterskog programa TOWER.

Na ovim slikama vidi se da horizontalna pomeranja glave šipa, usled dejstva sile H = 166 kN, iznose ρ = 6.41 mm (Slika 18) i ρ = 6.73 mm (Slika 19). Te vrednosti dobro se slažu s pomeranjem koje je prethodno dobijeno elastičnom analizom.

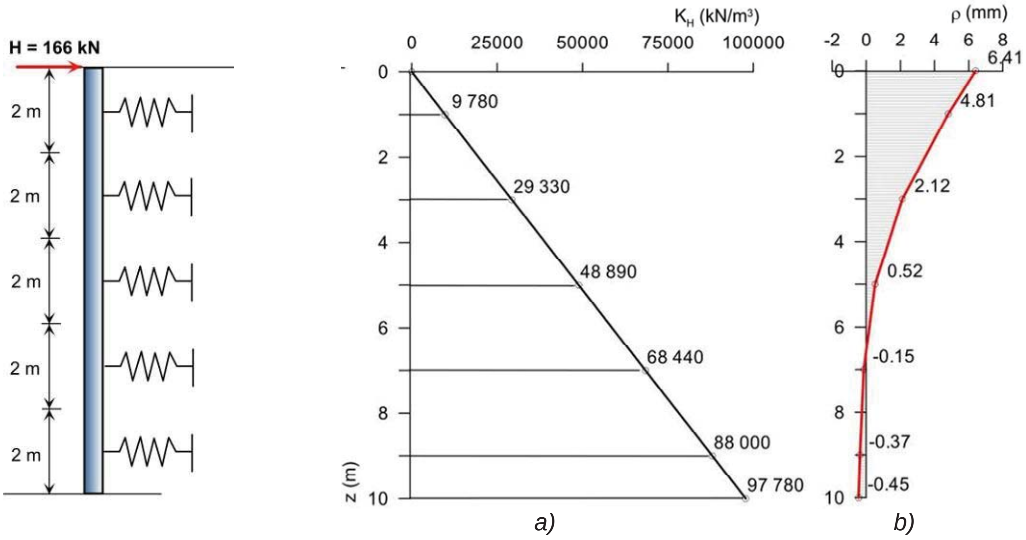
From the equation (5) the position of the rotation centre of the pile can be calculated i.e. the length L_0. In this case L_0 = 8,12 m. So, F_1 = 3762 kN and F_2 = 2332 kN.

From the equilibrium conditions of horizontal forces (eq. 6) ultimate horizontal force H_f = 1430 kN can be calculated. Allowable horizontal force, for safety factor F_s = 2,5, is H_a = 572 kN. It is much higher than the designed horizontal force H = 166 kN. It should be said that, by using Broms' method, it was estimated that H_f = 1525 kN and H_a = 610 kN [8].

Horizontal displacement of pile head ρ will be determined by elastic analysis (eq. 8). It was assumed that modulus of elasticity of a pile is E_p = 30000 MPa. In such a way, the pile flexibility factor is K_R = 0.00743 and influence factor is $I_{\rho H}$ = 4,8. So, designed horizontal force H = 166 kN causes horizontal displacement of the pile head ρ = 6.13 mm.

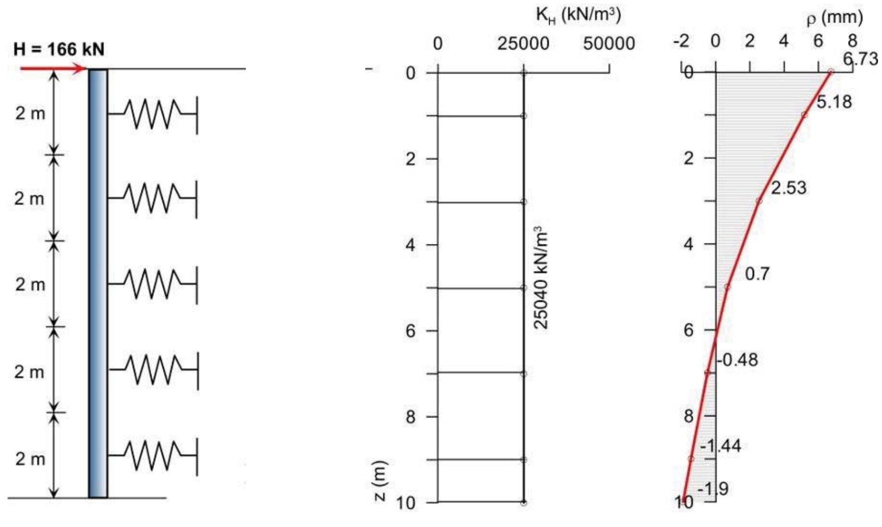
The horizontal displacements of the laterally loaded pile will be estimated by coefficient of horizontal subgrade reaction of a surrounding soil, too. In Fig. 18 and 19 is presented K_H concept for numerical analysis of a laterally loaded pile. Calculation procedure assumes that, because of considerable side shear resistance between pile and soil, the values n_h from Table 1 and K_H from equation 13 should be doubled i.e. n_h=2x4400=8800kN/m^3, and K_H = 2 x 12520 = 25040 kN/m^3. The numerical analysis has been performed by computer program TOWER.

From these figures may be observed that horizontal displacements of the pile head, caused by horizontal force H = 166 kN are ρ = 6.41 mm (Fig. 18) and ρ = 6.73 mm (Fig. 19). These values are in good agreement with previously obtained displacement by an elastic analysis.



Slika 18. Primena K_H u analizi bočno opterećenog šipa (Tercaghi, 1955)
a) vrednosti K_H za n_h = 8800 kN/m^3, b) horizontalna pomeranja šipa

Figure 18. Application of K_H for numerical analysis of laterally loaded pile (Terzaghi, 1955)
a) K_H values for n_h = 8800 kN/m^3, b) horizontal displacements of a pile



Slika 19. Primena K_H u analizi bočno opterećenog šipa (Vesić, 1961)
a) vrednosti K_H = 25040 kN/m^3, b) horizontalna pomeranja šipa

Figure 19. Application of K_H for numerical analysis of laterally loaded pile (Vesić, 1961)
a) K_H =25040kN/m', b) horizontal displacements of a pile

6 ZAKLJUČAK

Objekti koji su fundirani na šipovima često su izloženi značajnim horizontalnim silama. U tom slučaju, treba odrediti bočnu nosivost šipova i njihova horizontalna pomeranja. U ovom radu, pre svega, analizirali smo geotehničku nosivost šipova tj. bočnu nosivost koja je posledica loma okolnog tla, kao i deformacije bočno opterećenih šipova. Polazeći od toga, prvenstveno treba da se sprovedu adekvatna geotehnička istraživanja terena i da se formira – na osnovu dobijenih rezultata – geotehnički model terena na mestu budućeg objekta.

Na ovako definisanom modelu terena radi se proračun geotehničke bočne nosivosti šipova. S tim u vezi, treba voditi računa o tome da je reč o trodimenzionalnom problemu kao i da su u našoj zemlji često izraženi i složeni geotehnički uslovi. Uzimajući sve to u obzir, smatramo da je Brinč-Hansenova metoda vrlo pogodna za određivanje geotehničke bočne nosivosti šipova. Naravno, ukoliko je konstruktivna nosivost šipova manja od geotehničke nosivosti, onda je ona merodavna za određivanje maksimalne horizontalne sile koju šip može da prihvati.

Kod proračuna temelja oslonjenih na grupu šipova potrebno je da se uzme u obzir i grupni efekat šipova.

Prilikom određivanja deformacija bočno opterećenih šipova, u slučaju homogenog tla, mogu da se primene rešenja teorije elastičnosti. U složenim terenskim uslovima, međutim, pogodno je da se okolno tlo definiše odgovarajućim koeficijentima horizontalne krutosti ili p-y krivama i da se na osnovu toga odrede horizontalna pomeranja bočno opterećenih šipova.

Horizontalna pomeranja šipova treba da budu u dozvoljenim granicama. Ona su, pre svega, uslovljena karakteristikama objekta koji se fundira na šipovima.

U radu je prikazana i numerička analiza određivanja geotehničke nosivosti i horizontalnog pomeranja bočno opterećenih šipova. Budući da je tlo u kome se fundiraju šipovi homogeno i nekoherentno, urađen je proračun

6 CONCLUSION

Building structures which are founded with vertical piles are frequently loaded by high horizontal forces. In such cases, lateral bearing capacity and horizontal displacements of vertical piles have to be calculated. In this paper geotechnical capacity of piles i.e. lateral capacity which is governed by the strength of surrounding soil is analysed first. Accordingly, at first, it was necessary to make adequate geotechnical investigations, in laboratory and in situ and on the basis of the obtained results geotechnical model of terrain under the building structure had to be defined.

On such defined model geotechnical lateral capacity of piles is determined. In regard to that, it has to be considered that it is three-dimensional problem. Besides, in Serbia, there are very often complex geotechnical conditions. Accordingly, Brinch-Hansen's method is quite appropriate for determining geotechnical lateral bearing capacity. Surely, if the structural capacity of the piles is lesser than geotechnical capacity, maximum horizontal force that a pile can withstand should be estimated.

In calculation of lateral bearing capacity for a group of piles, their group effect has to be taken into account.

In the homogeneous soil, deformations of laterally loaded piles can be determined by elastic analysis. In complex geotechnical conditions, however, it is appropriate to define surrounding soil by coefficient of horizontal subgrade reaction or by p-y curves, too.

Such obtained deformations have to be in allowable limits which are restricted, at first, by structural characteristics of a building that is founded by the pile.

A numerical analysis for calculation geotechnical capacity and horizontal displacement of laterally loaded piles is presented in the paper. Taking into consideration that foundation soil is homogeneous and cohesionless, lateral bearing capacity is calculated not only by Brinch-Hansen's but Broms' method, too. The obtained results are in good agreement.

bočne nosivosti i po metodi Brinč-Hansena i po metodi Bromsa i dobijena su dobra slaganja. Osim toga, izračunata su i horizontalna pomeranja glave šipa elastičnom analizom, kao i pomoću koeficijenta horizontalne krutosti tla. Razlike u rezultatima su u uskim granicama, sasvim prihvatljivim za inženjersku praksu.

In addition, the pile head displacements are determined by elastic analysis and application theory of subgrade reaction. The calculated values are in narrow limits, quite acceptable for engineering practice.

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7 LITERATURA REFERENCES

[1] Bowles, J. E.: Foundation analysis and design, McGraw – Hill, New York, 4th Edition, 1988, pp 1004.
[2] Broms, B. R.: Lateral resistance of piles in cohesive soils, Journal of the Soil Mechanics and Foundations Division, ASCE, Vol 90, No. SM 2, 1964.
[3] Broms, B. R.: Lateral resistance of piles in cohesionless soils, Journal of the Soil Mechanics and Foundations Division, ASCE, Vol 90, No. SM 3, 1964.
[4] Broms, B. R.: Design of laterally loaded piles, Journal of the Soil Mechanics and Foundations Division, ASCE, Vol 91, No. SM 3, 1965.
[5] Canadian foundation engineering manual, 4th edition, Canadian Geotechnical Society, Calgary, Alberta, 2006.
[6] Conduto, D. R.: Foundation design "Principles and practice", Prentice Hall, New Jersey, 2001.
[7] Ćorić, S.: Geostatički proračuni – IV izdanje, Časopis Izgradnja i Srpsko društvo za mehaniku tla i geotehničko inženjerstvo, Beograd, 2017. str. 460.
[8] Ćorić, S., Rakić, D., Hadži-Niković, G., Basarić, I.: Bočna nosivost šipova opterećenih horizontalnim silama, Zbornik radova Geotehnički aspekti građevinarstva, 2017, str. 421-432.
[9] Das, B.M.: Principles of foundation engineering, Sixth Edition, Thomson Engineering, 2007, pp. 750.
[10] Davisson, M.T.: Lateral Load Capacity of Piles, Highway research Record, Transportation Research Boarde , Washington, DC, No. 333, 1970. pp. 104-112.
[11] Day, R. W.: Foundation engineering handbook, McGraw-Hill, New York, 2006.
[12] Berisavljević, D., Filipović, V., Ćorić, S., Rakić, D: Analiza bočno opterećenih šipova primenom rezultata DMT opita, Zbornik radova Geotehnički aspekti građevinarstva, 2017, str. 439-446.
[13] Folić, R., Liolios, A.: Application incilined piles in a seismic prone area, useful or not?, Zbornik radova Geotehnički aspekti građevinarstva, 2017, str. 461-472.
[14] Folić, B., Liolios, A., Liolios , K: Effects of horizontal interaction on redistribution of forces of piles in a group, Zbornik radova Geotehnički aspekti građevinarstva, 2017, str. 461-472.
[15] Hansen, J. B.: The ultimate resistance of rigid piles against transversal forces, Danish Geotechnical Institute, Bulletin No. 12, Copenhagen, 1961.
[16] Meyerhof, G. G. and Ranjan, G.: The bearing capacity of rigid piles under inclined loads in sand: Vertical piles, Canadian Geotechnical Journal, No. 9, pp. 430-446, 1972.
[17] Meyerhof, G. G.: Behaviour of pile foundations under special loading conditions: 1994 R. M. Hardy keynote address, Canadian Geotechnical Journal, No. 32, pp. 204-222, 1995.
[18] Milović, D., Đogo, M.: Šip opterećen horizontalnom silom – teorijski i eksperimentalni rezultati, Materijali i konstrukcije, 43, 3-4, 2000, str. 3-8.
[19] Milović, D., Đogo, M.: Ponašanje šipa pri dejstvu sile H određeno na osnovu rezultata statičke penetracije, Materijali i konstrukcije, 44, 3-4, 2001, str. 3-13.
[20] Milović, D., Đogo, M.: Problemi interakcije tlo-temelj-konstrukcija, Srpska akademija nauka i umetnosti – ogranak u Novom sadu, 2009, str. 248.
[21] Milović, S.: Interakcija tla i šipa opterećenog horizontalnom silom i momentom, Magistarska teza, Rudarsko-geološki fakultet Beograd, 1996.
[22] Poulos, H. G. and Davis, E. H.: Pile foundation analysis and design, John Wiley & Sons, New York, 1980.
[23] Rakić, D.: Geotehnički činioci i njihov uticaj na nosivost i sleganje vertikalno opterećenih šipova, Magistarska teza, Rudarsko-geološki fakultet Beograd, str. 193., 1997.
[24] Rakić, D. and Ćorić. S: Application of modern numerical methods in settlement analysis of a vertically loaded pile, Journal of mining and geological sciences, Volume 37. Belgrade, 1998. pp. 75-83.
[25] Rankine, W.M.J.: On the Stability of Lose Earth, Philosophical Transactions of the Royal Society, London Part I, 1857. pp. 9-27.
[26] Reese, C.L. and Van Impe, W: Single Piles and Pile Groups Under Lateral Loading, CRC Press, Taylor & Francis Group, London, 2011.
[27] Smith, T.D.: Fact or Friction: A Review of Soil Response to a Laterally Moving Pile, Part of: Foundation Engineering: Current Principles and Practices, F.H. Kulhawy editor, American society of civil engineers, ASCE, New York, Vol. 1., 1989, pp. 588-598.
[28] Terzaghi, K.: Evaluation of Coefficients of Subgrade Reaction . Geotechnique, Vol. 5, London, 1955, pp. 297-326.

[29] Tomlinson, M. J.: Foundation design and construction, The Pitman book, London, 1980.

[30] Tomlinson, M. J. and Woodward, J. C.: Pile design and construction practice, CRC Press, Boca Raton, 2015.

[31] U.S. Army Corps of Engineers, Deep Foundations, Unified Facilites Criteria (UFC), 2004, pp. 1-1 - D-2.

[32] Vesić, A. B.: Bending of Beams Resting on Isotropic Elastic Solid, Journal of the Engineering Mechanics Division, Vol. 87, Issue 2, 1961, pp. 35-54.

REZIME

BOČNA NOSIVOST I POMERANJA VERTIKALNIH ŠIPOVA OPTEREĆENIH HORIZONTALNIM SILAMA

Slobodan ĆORIĆ
Dragoslav RAKIĆ
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Temelji na šipovima često su izloženi značajnim horizontalnim silama. U takvim slučajevima, važno je da se odredi bočna nosivost vertikalnih šipova. Ona je uslovljena čvrstoćom okolnog tla (geotehnička nosivost) odnosno čvrstoćom poprečnog preseka šipa (konstruktivna nosivost). U radu je prvenstveno analizirana geotehnička nosivost šipova i primenjene su sledeće metode za određivanje bočne nosivosti pojedinačnih šipova: Rankinova, Bromsova i Brinč-Hansenova metoda. S tim u vezi, polazeći od složenih geoloških uslova koji su česti u Srbiji, smatramo da Brinč-Hansenova metoda ima prednost u odnosu na druge dve metode. Naime, ona može da se primeni i u homogenom i u heterogenom tlu i to za drenirane, kao i za nedrenirane uslove. To je veoma važno prilikom fundiranja objekata i prilikom sanacije klizišta. Zato je u radu prikazano i kako se u proračun uvodi grupno dejstvo šipova. Horizontalna pomeranja bočno opterećenih šipova mogu da se, u slučaju homogenog tla, odrede primenom teorije elastičnosti. U slučaju složenih geoloških uslova, međutim, ta pomeranja se određuju primenom koeficijenta horizontalne krutosti okolnog tla ili korišćenjem p-y krivih.Na kraju rada data je numerička analiza određivanja geotehničke nosivosti i horizontalnog pomeranja glave bočno opterećenih šipova koji se koriste za fundiranje silosa klinkera u Beočinu.

Ključne reči: pojedinačni šipovi, grupa šipova, bočna nosivost, dozvoljeno bočno opterećenje, bočne deformacije.

SUMMARY

LATERAL CAPACITY AND DEFORMATIONS OF VERTICAL PILES LOADED BY HORIZONTAL FORCES

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Stanko ĆORIĆ
Irena BASARIĆ

Pile foundations are frequently loaded by horizontal forces. In such cases, it is important to calculate lateral capacity of vertical piles. It is governed by the strength of the surrounding soil i.e. geotechnical capacity or pile strength parameters i.e. structural capacity of a pile. In this paper, geotechnical capacity is analysed first, and then the Rankine's, Broms' and Brinch-Hansen's methods for calculating ultimate bearing capacity of a single pile under lateral loads are presented. In accordance with complex geological conditions, which are very often in Serbia, Brinch-Hansen's method has an advantage over the other two methods. It can be applied both to uniform and layered soils under drained or undrained conditions. This is highly important for foundation of structures and landslide's remedial measures, too. Accordingly, load capacity calculation of a pile group is presented as well. In the case of homogenous surrounding soil, deformations of laterally loaded piles may be determined by elastic analysis. However, in the case of complex geological conditions, these deformations may be calculated by the concept of coefficient of subgrade reaction or by p-y curves, too. Finally, numerical analysis for calculation of geotechnical capacity and pile head displacement of laterally loaded piles for foundation of Clinker Bin in Beocin is presented.

Key words: single piles, pile groups, ultimate lateral capacity, allowable lateral load, lateral deformations.

SISTEMATIZACIJA ANALITIČKIH I NUMERIČKIH METODA PRORAČUNA STABILNOSTI KLIZIŠTA

THE SYSTEMATIZATION OF ANALYTICAL AND NUMERICAL METHODS OF LANDSLIDE STABILITY CALCULATION

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1 UVOD

S obzirom na kompleksnost geometrije reljefa zemljine površi, kosine su među problematičnijim geološkim formama u geotehnici. Kosine karakteriše nagla promena geometrije terena (denivelacija), s predispozicijom promene ove geometrije usled dejstva različitih faktora. Najčešći i najsloženiji vid narušavanja tla i geometrije kosine odnosi se na stabilnost terena – bilo prirodnih padina ili veštačkih kosina. Svako narušavanje postojeće ravnoteže na padinama ili kosinama izaziva pomeranja pod uticajem gravitacije: klizanje, odronjavanje ili tečenje površinskog dela tla, ali i dubljih delova stenske mase. Za ovako uspostavljeno klizanje, u geološkoj i geotehničkoj terminologiji i nomenklaturi, ustaljen je termin – klizište [11]. Uslovi za nastanak i razvoj klizišta jesu: geotehnički, geološki, geomorfološki, hidrogeološki, meteorološki, vegetacioni, antropogeni, dejstvo zemljotresa, dejstvo akumulacija, vibracije usled saobraćaja i drugi.

U poslednjih sto godina, zabeležen je znatan broj katastrofalnih klizišta, nastalih kao posledica dejstva zemljotresa, erupcije vulkana, nagomilavanja snega, višednevnih i intenzivnih kiša i uragana [16]. Zbog formiranja ovih klizišta, poginulo je nekoliko stotina hiljada ljudi koji su - u najvećem broju slučajeva - imali sagrađene

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1 INTRODUCTION

Given the complexity of geometry of the relief of the earth's surface, slopes represent one of the problematic geological forms in geotechnics. The slopes are characterized by a sudden change in geometry of the terrain (denivelation) with a predisposition to the change of this geometry due to the effects of various factors. The most common and most complex type of soil disturbance and slope geometry is the stability of the terrain, whether natural slopes or artificial slopes. Any disturbance of the existing balance on the slopes causes displacement under the influence of gravity: sliding, erosion or flowing the surface of the soil, but also the deeper parts of the rock mass. For the established sliding, in the geological and geotechnical terminology and nomenclature, the term "landslide" is established [11]. Conditions for the formation and development of landslides are: geotechnical, geological, geo-morphological, hydro-geological, meteorological, vegetation, anthropogenic, earthquake effects, accumulation effects, traffic vibrations, etc.

In the last hundred years there has been a significant number of catastrophic landslides that have occurred as a result of earthquakes, volcanic eruptions, snow accumulation, multi-day heavy rainfall and hurricanes [16]. Due to the formation of these landslides several

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domove na klizištima ili u njihovoj neposrednoj blizini. Prema [29], u najkatastrofalnija klizišta – zabeležena u poslednjih sto godina – ubrajaju se: *Haiyuan landslides* u Kini 1920, *Vargas tragedy* u Venecueli 1999, *Nevado del Ruiz debris flows* u Kolumbiji 1985, *Nevados Huascaran debris avalanche* u Peru 1970, *North India flood mudslides* u Indiji 2013, *Khait rock slide* u USSR 1949 i slično. U katastarskom listu evidencije klizišta u Srbiji, zabeleženo je više od 2.200 aktivnih, trenutno umirenih i reaktivnih klizišta [15]. Znatno je manji broj trenutno umirenih od aktivnih i reaktivnih klizišta. Vrlo često, u praksi se susrećemo s problemima stabilnosti klizišta, kada je nakon izvedenih terenskih istraživanja, laboratorijsko-geomehaničkih ispitivanja, definisanja uzroka i uslova nastanka klizišta potrebno definisati mere sanacije. Međutim, da bismo uspešno upravljali svim projektnim situacijama analize stabilnosti i sanacije klizišta, potrebno je da imamo kvalitetne matematičke modele i metode analize stabilnosti klizišta. Dosadašnja iskustva pokazuju da postoji potreba za implementacijom kompleksnijih (realističnijih) matematičkih modela u praktične svrhe, kao i za dodatnim unapređivanjem postojećih metoda analize stabilnosti klizišta.

Jedan od prvih radova u kojem su adekvatno teorijski razmatrani aspekti nekoliko analitičkih, ali i numeričkih metoda analize stabilnosti klizišta, jeste rad [9], gde je sprovedena klasifikacija, imajući u vidu: formulaciju graničnog stanja (*limit formulation*) i formulaciju stanja pomeranja (*displacement formulation*). Kod formulacije graničnog stanja, postoje dve opcije: gornja granična rešenja (*upper bound solution*) i donja granična rešenja (*lower bound solution*), pri čemu metoda karakteristika pomeranja (*method of characteristics for displacement*) pripada grupi gornjih graničnih rešenja, a metoda karakteristika napona (*method of characteristics for stress*) pripada grupi donjih graničnih rešenja. Metode definisane prema formulaciji graničnog stanja, zapravo su metode granične ravnoteže (LEM - *Limit Equilibrium Method*), od kojih se najčešće primenjuju gornja granična rešenja. U poređenju s njima, metode definisane prema formulaciji stanja pomeranja, zapravo su metode analize pomeranja (DFM - *Displacement Formulation Method*), odnosno numeričke metode. U radu [20] dat je pregled numeričkih metoda stabilnosti klizišta, pri čemu je korišćena formulacija po metodi konačnih razlika (FDM - *Finite Difference Method*). Studija performansi nekoliko različitih metoda stabilnosti klizišta prikazana je u radu [27], dok su u radu [21] prikazane metode stabilosti klizišta, imajući u vidu deterministički pristup, teoriju pouzdanosti i optimizacije. Primena numeričkih metoda analize stabilnosti kosina u izmenjenoj serpentinskoj stenskoj masi prikazana je u radu [25], pri čemu su, između ostalog, korišćeni i sledeći parametri: geološki indeks čvrstoće i deformabilnosti stenske mase, dok je kao kriterijum sloma primenjen *Hoek Brown*-ov kriterijum. Razmatranje kompleksne problematike stabilnosti klizišta, iz aspekta analize hazarda, analize povredljivosti, procene i upravljanja rizikom prikazani su u radu [6], gde je - zasnivajući se na prethodno navedenim teorijama - predložen GIS integralni model za analizu klizišta.

Cilj istraživanja prikazanog u ovom radu jeste da se detaljnije sistematizuju metode proračuna klizišta i algoritmi modeliranja, s posebnim osvrtom na numeričke analize stabilnosti.

hundred thousand people were killed, who in most cases had their own homes built on or near the landslide. According to [29], the most catastrophic landslides recorded over the last hundred years, were: *Haiyuan landslides* in China 1920, *Vargas tragedy* in Venezuela 1999, *Nevado del Ruizdebris flows* in Colombia 1985, *Nevados Huascaran debris avalanche* in Peru 1970, *North India flood mudslides* in India 2013, *Khait rock slide* in the USSR in 1949 and the like. In the cadastral register of landslide records in Serbia, more than 2200 active, currently calm and reactive landslides have been recorded [15]. There is a significantly lower number of currently calm, compared to active and reactive landslides. Very often, in practice, problems with the stability of the landslide are encountered, when after the conducted field investigations, laboratory-geomechanical tests, defining the causes and conditions of landslide formation, it is necessary to define repair measures. However, in order to successfully manage all project situations of the landslide stability analysis and landslide repair, it is necessary to have high quality mathematical models and landslide methods. Previous experience shows that there is a need for the implementation of more complex (more realistic) mathematical models for practical purposes and further improvement of existing landslide stability methods.

One of the first papers in which the aspects of several analytical and also numerical methods of landslide stability are adequately theoretically considered is the paper [9], where the classification was carried out taking into account the *limit formulation* and the *displacement formulation*. There are two options for the limit formulation: *upper bound solution* and *lower bound solution*, where the *method of characteristics for displacement* belongs to the group of *upper bound solutions*, and the *method of characteristics for stress* belongs to the group of *lower bound solutions*. The methods defined by the *limit formulation* are in fact the *Limit Equilibrium Method* (LEM), of which the *upper bound solutions* are most commonly applied. In relation to them, the methods defined by the *displacement formulation* are in fact *Displacement Formulation Methods* (DFM), or numerical methods. The paper [20] gives an overview of the numerical methods of landslide stability, using the *Finite Difference Method* (FDM) formulation. A study of the performances of several different landslide stability methods is presented in [27], while in [21] the methods of landslide stability are presented taking into account the deterministic approach, the theory of reliability and optimization. The use of numerical methods for analyzing the stability of slopes in the alternating serpentine rock mass was shown in [25], where, among other things, these parameters were used: the geological strength index and deformability of the rock mass, and for the criterion of failure *Hoek Brown*'s criterion was applied. Consideration of the complex problem of landslide stability, but also from the aspect of hazard analysis, vulnerability analysis, risk assessment and risk management, are presented in [6], where, based on the aforementioned theories, a GIS integral model for landslide analysis is proposed.

The aim of the research presented in this paper is to further systematize the methods of landslide calculations and modelling algorithms with a special emphasis on numerical stability analyses.

2 UOPŠTENO O RAZMATRANJIMA STABILNOSTI KLIZIŠTA I FAKTORIMA BITNIM ZA PRORAČUN

Generalno razmatrajući, kosine se mogu nalaziti u stabilnom ravnotežnom, nestabilnom neravnotežnom i indiferentnom poluravnotežnom stanju. Stabilno ravnotežno stanje karakteriše uspostavljen odnos destabilizujućih i stabilizujućih sila, tako da – ukoliko je uticaj stabilizujućih sila veći - veći je i faktor sigurnosti. Nestabilno neravnotežno stanje karakteriše narušen odnos destabilizujućih i stabilizujućih sila, tako da je uticaj destabilizujućih sila dominantan. Indiferentno (neodređeno) poluravnotežno stanje predstavlja prelaznu kategoriju između stabilnog ravnotežnog i nestabilnog neravnotežnog stanja. Odnos destabilizujućih i stabilizujućih sila indiferentnog poluravnotežnog stanja značajnije je narušen u stabilno ravnotežnom stanju i dovoljan je i mali priraštaj destabilizujućih sila, pa da transformiše ovo stanje u nestabilno neravnotežno stanje. S obzirom na kompleksnost indiferentnog stanja i mogućnost parcijalne promene geometrije kosine, odnosno poluformiranja klizišta, indiferentno stanje karakteriše skup više različitih poluravnotežnih stanja. Ovo je posebno karakteristično u situacijama kada nastupi narušavanje stabilitetnog ravnotežnog stanja, pri čemu ne mora doći do potpunog kretanja klizne mase tla, već se može uspostaviti novo poluravnotežno stanje. Detaljnija klasifikacija stabilitetnih i nestabilitetnih stanja kosina, podrazumevajući pritom i prelazne kategorije, prikazana je u [24]: stabilna kosina, potencijalno nestabilna kosina, rana faza rušenja, srednja faza rušenja, delimično ili totalno rušenje i potpuno rušenje,dok su mehanizmi nastanka i razvoja klizišta: rotacioni model, translacioni model, model formiran iz različitih geometrijskih formi blokova, model s klizanjem, kotrljanjem i padanjem kamena različitih dimenzija, model sa značajnim odvaljivanjem kliznog tla, klizište formirano usled obimnih kiša, klizište formirano kao obimni bujični tok, klizište formirano tečenjem tla i klizište formirano puzanjem tla uz pojavu prslina i rascepa u tlu. Jedan od ključnih parametara pri klasifikaciji klizišta jeste brzina kretanja klizne mase, kao i uticaj površinske i podzemne vode. Uopšte uzev, može se konstatovati da klizišta koja imaju veći nagib spoljašnje konture tla - imaju i veću brzinu kretanja klizne mase [18]. Ovo je posledica dejstva gravitacionih sila. Međutim, razmatranje uticaja brzine kretanja klizne mase tla i inkrementalnog povećanja vode u tlu zahteva primenu metoda za proračun stabilnosti klizišta u vremenskom domenu, a što je dosta kompleksnije od uobičajenih metoda proračuna.

Metodologija analize potencijalnog klizišta sastoji se iz sledećih nekoliko segmenata: geodetsko osmatranje terena i prikupljanje podataka, geotehnička *in-situ* ispitivanja, analiza fizičko-mehaničkih parametara tla u laboratoriji i proračun kosine primenom matematičkih metoda u geotehnici. Metodologija analize formiranog klizišta zasniva se na projektu sanacije klizišta, koji se takođe sastoji iz nekoliko segmenata: geodetsko osmatranje terena i prikupljanje podataka, geotehnička *in-situ* ispitivanja, analiza fizičko-mehaničkih parametara tla u laboratoriji, rekonstruktivna analiza prethodnog stanja klizišta, analiza faktora koji su doveli do formiranja klizišta, razmatranje varijantnih rešenja sanacije klizišta, proračuni varijantnih rešenja klizišta primenom matematičkih metoda u geotehnici, ekonomska analiza varijant-

2 GENERAL ON LANDSLIDE STABILITY CONSIDERATIONS AND FACTORS RELEVANT TO THE CALCULATION

Generally speaking, the slopes can be found in a stable equilibrium state, unstable imbalance state and indifferent semi-equilibrium state. A stable equilibrium state is characterized by the established relation of destabilizing and stabilizing forces, so if the effect of stabilizing forces is greater, the safety factor is greater. The unstable imbalance state is characterized by a disturbed relation between destabilizing and stabilizing forces, so the influence of destabilizing forces is dominant. The indifferent (indefinite) half-balance state represents a transition category between a stable equilibrium and an unstable imbalance state. The ratio of destabilizing and stabilizing forces of the indifferent semi-equilibrium state is significantly more disturbed than the stable equilibrium state, and it is sufficient that the small increment of destabilizing forces transforms this state into an unstable imbalance state. Given the complexity of the indifferent state and the possibility of a partial change in the slope geometry or the semi-forming of the landslide, the indifferent state is characterized by a set of several different half-equilibrium states. This is especially characteristic in situations where the disturbance of the stable equilibrium state occurs, without the complete movement of the sliding mass of the soil, but a new half-balance state can be established. A more detailed classification of the stable and instable states of the slopes, taking into account the transition categories, is shown in [24]: stable slope, potentially unstable slope, early demolition phase, medium demolition phase, partial or complete demolition and complete demolition, while the mechanisms of formation and development of landslides are: rotational model, translational model, model formed from different geometric shapes of blocks, model with sliding, rolling and falling of stone of different dimensions, model with significant sliding of the landslide soil, landslide formed due to heavy rainfall, landslide formed as voluminous torrential flow, landslide formed by soil flow and landslide formed by soil crawling with the appearance of cracks and clefts in the soil. One of the key parameters in landslide classification is the velocity of movement of the sliding mass, as well as the level of underground water in the soil. In general, it can be concluded that the landslides, which have a higher slope of the outer contour of the soil, have a higher velocity of movement of the sliding mass [18]. This is due to the effect of gravitational forces. However, the consideration of the influence of the rate of movement of the sliding mass of the soil and the incremental increase in water in the soil requires the application of methods for estimating the stability of landslides in the time domain, which is considerably more complex than the usual methods of calculation.

The methodology of the analysis of the potential landslide consists of several segments: geodetic survey of terrain and data collection, geotechnical in-situ testing, the analysis of physico-mechanical parameters of soil in the laboratory and calculation of slope using mathematical methods in geo-technics. The methodology of the analysis of the formed landslide is based on a landslide repair project consisting of several nih rešenja, višekriterijumska optimizacija varijantnih rešenja i detaljna analiza tehnologije sanacije klizišta za optimalno izabrano rešenje.

Prilikom analize klizišta, sprovode se prethodna geotehnička *in-situ* ispitivanja pomoću kojih se prvenstveno formira inženjersko-geološki profil terena. Ključna geotehnička ispitivanja koja se sprovode za formiranje inženjersko-geološkog profila terena jesu istražne bušotine. One se sprovode tehnikom bušenja jezgrovanjem, prilikom čega se uzorci tla pažljivo klasifikuju radi identifikacije tipa tla po dubini i analize fizičko-mehaničkih karakterstika tla. Izvođenje istražne bušotine potrebno je sprovesti dovoljno duboko, kako bi se na inženjersko-geološkom profilu klizišta utvrdila klizna površ. Broj potrebnih istražnih bušotina u korelaciji je s geometrijom klizišta, dimenzijama klizišta, dubinama klizne površi, promenljivosti geologije i tako dalje.

Za razliku od geotehničkih ispitivanja klizišta, geodetska ispitivanja sprovode se radi utvrđivanja geometrije, dimenzija i monitoringa klizišta. Na osnovu snimljene geometrije klizišta, formira se situacioni plan klizišta u 2D koordinatnom sistemu. Identifikacijom većeg broja kliznih ravni za odgovarajući broj inženjersko-geoloških profila i njihovom integracijom sa 2D situacionim planom klizišta, konstruiše se 3D model klizišta u softveru za geometrijsku prezentaciju (CAD - *Computer Added Design*). Ovako povezane klizne ravni formiraju kliznu površ. Konstruisan 3D model klizišta, formiran iz oblaka tačaka i linija, može se eksportovati u softver za numeričku analizu stabilnosti klizišta. Monitoring i analiza pomeranja klizne mase, geodetskim metodama, sprovodi se radi periodičnog ili kontinualnog praćenja stanja klizišta: direktno na terenu (geodetskim instrumentima, primenom radara na zemlji, brzih kamera), primenom radara iz satelita, bespilotnih letelica (dronova), aviona, terestričkog laserskog skeniranja ili kombinovano. Podaci dobijeni monitoringom iz inicijalnih stanica (GPS - *Global Positioning System*) direktno se transferuju u baznu stanicu, a zatim u kontrolni centar za dalju obradu. S obzirom na to što klizišta karakteriše pomeranje klizne mase, prvenstveno se prate horizontalna i vertikalna površinska pomeranja i horizontalna i vertikalna pomeranja u unutrašnjosti klizišta na određenim dubinama. Takođe, monitoring se sprovodi i za kontrolu varijacije nivoa podzemne vode primenom pijezometara i analizu vertikalne i ortogonalnih horizontalnih akceleracija primenom akcelerometara. Zapis akceleracija prikazuje se akcelerogramom koji se naknadno, u digitalizovanom formatu, procesira: skaliranjem, filtriranjem, korekcijom bazne linije (BLC - *base line correction*), kompatibilizacijom (SM - *spectral matching*) i algoritmom konvolucije/dekonvolucije. Svi ovi podaci – dobijeni geodetskim osmatranjem terena – mogu se interaktivno uključiti u matematički model kojim se sprovodi analiza stabilnosti klizišta, tako da se kroz vreme, kontinualnom korekcijom numeričkog modela, upravlja svim aspektima proračuna i dodatno smanjuje nivo nepouzdanosti ulaznih parametara (parametri proračunskog modela i parametri spoljašnjih/unutrašnjih dejstava). Ovakav numerički model predstavlja, zapravo, numerički model u realnom vremenu (*real time numerical model*).

Prilikom formiranja proračunskog modela klizišta, potrebno je razmotriti sve relevantne parametre i odrediti njihove vrednosti, budući da je konačno rešenje u

segments: geodetic surveying of the terrain and data collection, geotechnical in-situ testing, analysis of physico-mechanical parameters of soil in the laboratory, reconstructive analysis of the previous landslide state, analysis of the factors that led to the formation of landslides, the consideration of variant solutions for landslide repair, calculations of variant landslide solutions using mathematical methods in geotechnics, economic analysis of variant solutions, multicriteria optimization of variant solutions, and detailed analysis of landslide repair technology for the chosen optimal solution.

During the landslide analysis, preliminary geotechnical in situ testing are carried out, by which, in the first place, an engineering-geological profile of the terrain is formed. Key geotechnical investigations carried out for the formation of the engineering-geological profile of the terrain are exploratory boreholes. Exploratory boreholes are made using core drilling technique, where soil samples are carefully classified for soil type identification according to the depth and the analysis of physico-mechanical characteristics of the soil. The execution of the exploratory borehole must be carried out deep enough to determine the sliding surface on the engineering-geological profile of the landslide. The number of required exploratory boreholes is in correlation with: landslide geometry, landslide dimensions, depths of sliding surfaces, geological variations, and the like.

In relation to geotechnical landslides testing, geodetic testing are carried out in order to determine the geometry, dimensions and monitoring of the landslide. Based on the recorded landslide geometry, the situational plan of the landslide is formed in the 2D coordinate system. By identifying a greater number of sliding plates for the corresponding number of engineering-geological profiles and by integrating them with the 2D situational landslide plan, a 3D model of landslide in *Computer Added Design* (CAD) was constructed. The associated sliding plane forms a sliding surface. The constructed 3D landslide model, formed from cloud of nodes and lines, can be exported to the software for numerical analysis of landslide stability. Monitoring and analysis of sliding mass movement, by geodetic methods, are carried out in order to periodically or continuously monitor the landslide situation: directly on the ground (geodetic instruments, using radars on earth, fast cameras), using radar from satellites, unmanned aircraft (drones), planes, terrestrial laser scanning or combined. The data obtained from monitoring from the initial stations (GPS - *Global Positioning System*) are directly transferred to the base station, and then to the control centre for further processing. Since the landslides are characterized by the movement of the sliding mass, this is primarily followed by horizontal and vertical surface movements and horizontal and vertical movements in the interior of the landslide at certain depths. In addition, monitoring is also carried out to control the variation of groundwater level by using piezometers and analyzing vertical and orthogonal horizontal acceleration using accelerometers. The acceleration record is displayed with an accelerometer, which is subsequently processed in a digitized format: scaling, filtering, baseline correction, spectral matching, and convolution/deconvolution direktnoj korelaciji sa selekcijom i varijacijom vrednosti parametara. Relevantni parametri proračunskog modela klizišta mogu se klasifikovati u pet grupa: parametri geometrije proračunskog modela, parametri fizičko-mehaničkih karakteristika tla, parametri dejstava, posebni tipovi parametara i parametri proračuna. Parametrima geometrije proračunskog modela modelira se kompleksnost geometrije kosine i višeslojnost tla po dubini. S obzirom na to što slojevi mogu biti složene geometrije, a ne samo horizontalni ili približno horizontalni, to se pri proračunu kompleksne višeslojne geometrije tla primenjuju numeričke metode proračuna klizišta. Takođe, u ove parametre ubrajaju se i parametri geometrije konstrukcija koje se nalaze na klizištu ili u njihovoj blizini ili su to konstrukcije kojima se sprovodi sanacija klizišta, tako da se i za njih, pri proračunu, uzima u obzir efekat interakcije konstrukcija-tlo (SSI - *soil-structure interaction*). Pravilan unos ovih parametara zavisi od nivoa kvaliteta formiranog inženjersko-geološkog profila terena. Parametri fizičko-mehaničkih karakteristika tla dobijaju se iz laboratorijskog ispitivanja uzoraka, od kojih se izdvajaju: opit jednoaksijalne čvrstoće, opit direktnog smicanja, triaksijalni opit i edometarski opit stišljivosti. Za analize stabilnosti kosina značajni su sledeći parametri: zapreminska težina tla, težina tla u zasićenom stanju, kohezija, ugao unutrašnjeg trenja, *Young*-ov modul elastičnosti, edometarski modul stišljivosti, modul deformacije, *Poisson*-ov koeficijent, referentan modul smicanja, dilatancija, koeficijent poroznosti i tako dalje. U zavisnosti od tipa konstitutivnog modela ponašanja tla, definišu se i relevantni parametri, s tim što se kod konstitutivnih modela kojim se opisuje trodimenzionalno naponsko stanje znatno povećava broj parametara. Najčešće, kao konstitutivni model ponašanja tla, pri analizi klizišta, primenjuje se *Mohr-Coulomb*-ov model tla, dok se – u zavisnosti od specifičnosti tipa tla – mogu koristiti omekšavajući (*soft soil model*) ili ojačavajući (*hardening soil model*), *Cam-Clay* model, *Drucker-Prager*-ov model i drugi. Postoje i dodatni parametri kojima se unapređuje konstitutivni model ponašanja tla; na primer, parametri kojima se dodatno utiče na promenu čvrstoće i kohezije po dubini tla, uvođenje zatežuće čvrstoće tla i definisanje parametara konsolidacije. Takođe, dobro je poznavati konzistenciju tla (veoma meka, meka, srednja, kruta, veoma kruta). Prilikom definisanja parametara prema EN 1997-1:2004 propisima, potrebno je poznavati parcijalne faktore za ugao unutrašnjeg trenja, efektivnu koheziju i nedreniranu smičuću čvrstoću tla [7]. Parametrima dejstava definišu se: tipovi opterećenja (koncentrisano, linijsko, površinsko, prostorno), tipovi dejstva opterećenja (stalno, povremeno, incidentno), seizmičko dejstvo (preko seizmičkih koeficijenata, pri čemu se odgovor klizišta razmatra u domenu analize kapaciteta pomeranja ili preko akcelerograma, pri čemu se odgovor klizišta razmatra u vremenskom domenu) i projektne situacije (stalna, povremena, incidentna, seizmička). Posebnim tipovima parametara modeliraju se: konturni uslovi (samo komponente krutosti ili komponente krutosti i prigušenja), prelazni uslovi (*interface zone*), kruta tela (ne uzimaju se u obzir efekti njihovih deformacija, već samo pomeranja u ukupnim pomeranjima sistema), specifično trenje na relaciji konstrukcija-tlo (za konstrukcije koje se nalaze na klizištu ili u njegovoj blizini ili su to konstrukcije kojima se sprovodi sanacija klizišta),

algorithm. All these data obtained by geodetic field observation can be interactively included in the mathematical model that analyzes the stability of the landslide so that through time, the continuous correction of the numerical model is managed by all aspects of the budget and further decreases the level of inconsistency of the input parameters (parameters of the budget model and parameters of the external/internal actions). This numerical model is, in fact, real time numeric model.

When forming the calculated landslide model, it is necessary to consider all relevant parameters and determine their values, since the final solution is in a direct correlation with the selection and variation of parameter values. The relevant parameters of the calculated landslide model can be classified into five groups: parameters of the geometry of the calculated model, parameters of physical-mechanical characteristics of the soil, parameters of actions, special types of parameters and calculation parameters. The complexity of the slope geometry and the multi-layered soil depth are modelled by parameters of the geometry of the calculated landslide model. Since the layers can be complex geometries, and not only horizontal or approximately horizontal, the numerical methods of calculating the landslide are used in the calculation of complex multilayer soil geometry. In addition, these parameters include the parameters of the geometry of the structures located on or near the landslide, or they are constructions for which the landslide is being repaired, so that for them, the effect of the soil-structure interaction (SSI) is considered. The correct input of these parameters depends on the level of quality of the formed engineering-geological profile of the terrain. The parameters of the physical-mechanical characteristics of the soil are obtained from laboratory testing of samples, of which the following are distinguished: one-axial strength, direct shear strength, triaxial test and edometric compressibility test. For stability analyzes of slopes, parameters are important: soil weight, soil weight in saturated state, cohesion, internal friction angle, *Young*'s elastic modulus, edometric modulus of compressibility, deformation module, *Poisson*'s coefficient, reference shear modulus, dilatation, coefficient porosity and other. Depending on the type of constitutive model of soil behaviour, the relevant parameters are defined, whereas for the constitutive models describing the three-dimensional stress state the number of parameters is considerably increased. Most often, *Mohr-Coulomb*'s soil model is used as a constitutive soil model for analyzing landslide, while depending on soil type specificity, soft soil model or hardening soil model can be used, *Cam-Clay* model, *Drucker-Prager* model and others. There are also additional parameters that enhance the constitutive model of soil behaviour, such as, for example, parameters that additionally affect the change in strength and cohesion along the depth of the soil, the introduction of tensile strength of the soil and the definition of consolidation parameters. In addition, knowing the soil consistency (very soft, soft, medium, rigid, very rigid) is of significant interest. When defining the parameters according to EN 1997-1:2004 code, it is necessary to know the partial factors for: angle of internal friction, effective cohesion and undrained shear strength of soil[7]. The parameters of the actions are defined: types of loads (concentrated, linear, surface, elementi veze kojima se mogu, između ostalog, modelirati i specifični konstruktivni elementi (model ponašanja može biti linearno-elastičan ili nelinearan), podzemna voda (direktno modeliranje horizontalnog ili promenljivog nivoa podzemne vode NPV, modeliranje pornih pritisaka, modeliranje sile uzgona, modeliranje prslina na površini tla ispunjenih vodom i nastalih usled zatezanja) i fazna gradnja/sanacija (modeliranje promene geometrije kosine po fazama gradnje/sanacije, modeliranje promene tla po fazama gradnje/sanacije, modeliranje promene nivoa podzemne vode po fazama gradnje/sanacije, modeliranje promene opterećenja po fazama gradnje/sanacije, modeliranje promene dejstva zemljotresa po fazama gradnje/sanacije, modeliranje promene projektne situacije po fazama gradnje/sanacije). Parametri proračuna umnogome definišu aspekte numeričkih analiza stabilnosti kosina: broj inkremenata kod inkrementalno-iterativne analize, broj iteracija kod inkrementalno-iterativne analize, broj korekcija matrice krutosti sistema, vrednosti tolerancija (za pomeranje, neizbalansirane/rezidualne sile i energiju) i faktor optimizacije (algoritam pretraživanja minimalnog faktora sigurnosti za veći broj kliznih površi).

3 METODE PRORAČUNA STABILNOSTI KLIZIŠTA

3.1 Podela metoda proračuna stabilnosti klizišta

Metode proračuna stabilnosti klizišta generalno se mogu podeliti u četiri grupe: analitičke, numeričke, eksperimentalne i hibridne. U zavisnosti od toga koja će metoda biti primenjena, dobijaju se rešenja s manjim ili veći stepenom pouzdanosti, s tim što prednost treba dati numeričkim metodama. S obzirom na to što se analitičke i numeričke metode proračuna stabilnosti klizišta najviše primenjuju pri projektovanju i sanaciji klizišta, ali i za potrebe naučnih istraživanja, pregled istraživanja – prikazan u daljem tekstu rada – odnosi se samo na ove metode. U zavisnosti od načina dobijanja konačnog rešenja ispitivanja stabilnosti klizišta, moguće je sprovesti podelu na metode kojima se rešenje dobija putem jednog koraka ili jednokoračne analize (*one step*), putem više koraka ili višekoračne analize (*step by step*) i inkrementalno-iterativne nelinearne analize. Shodno prethodno definisanom, uvedena je podela na metode proračuna klizišta:

spatial), types of load action (permanent, occasional, incidental), seismic effect (through seismic coefficients, where the response of the landslide is considered in the capacity domain or through the accelerogram, where the response of the landslide is considered in the time domain) and the project situation (permanent, occasional, incidental, seismic). Specific types of parameters are modelled: contour conditions (only stiffness or stiffness and damping components), interface zone, rigid bodies (they do not take into account the effects of their deformations, but only displacements in overall system displacements), specific friction on construction-ground relation (for structures located on or near the landslide or structures that are used for landslide repair), link elements which can be used to model specific structural elements (the behaviour model can be linear-elastic or non-linear), groundwater (direct modelling of horizontal or variable level of groundwater NPV, modelling of the pore stress, modelling of the lifting force, modelling of cracks on the surface of the soil filled with water and caused by tensioning) and phase construction/repair (modelling the slope geometry change by construction/repair phases, modelling the soil change by construction/repair phases, modelling the groundwater level change by construction/repair phases, modelling the load change by construction/repair phases, modelling the change of the earthquake effects by construction/repair phases, modelling the change in the project situation by construction/repair phases). The calculated parameters define, as far as possible, the numerical analysis of the slope stability: number of increments in the incremental-iterative analysis, number of iterations in the incremental-iterative analysis, number of corrections of the system stiffness matrix, tolerance values (for displacement, unbalanced/residual forces and energy) and optimization factor (algorithm for search of the minimal safety factor, for a greater number of sliding surfaces).

3 METHODS OF LANDSLIDE STABILITY CALCULATION

3.1 Methods of landslide stability calculation division

Methods of landslide stability calculation can, generally, be divided into four groups: analytical, numerical, experimental and hybrid. Depending on the method applied, solutions with a lower or a higher degree of reliability are obtained, while the priority should be given to numerical methods. Since the analytical and numerical methods of landslide stability calculation are mostly applied in the design and repair of landslides, but also for the needs of scientific researches, this exactly is why the overview of the researches, presented in the following text, applies only to these methods. Depending on the method of obtaining the final solution of the landslide stability test, it is possible to divide the methods according to whether the solution is obtained through one step or one-step analyses, through several steps or step-by-step analyses, and incrementally-iterative nonlinear analyses. According to the previously defined, a division of landslide calculation methods was introduced:

– analitičke jednokoračne;
– analitičke višekoračne (iteracije kliznih površi);
– numeričke višekoračne (iteracije kliznih površi);
– numeričke inkrementalno-iterativne (nelinearne) analize;
– numeričke inkrementalno-iterativne (nelinearne) analize, s primenjivanjem numeričke integracije u vremenskom domenu.
–

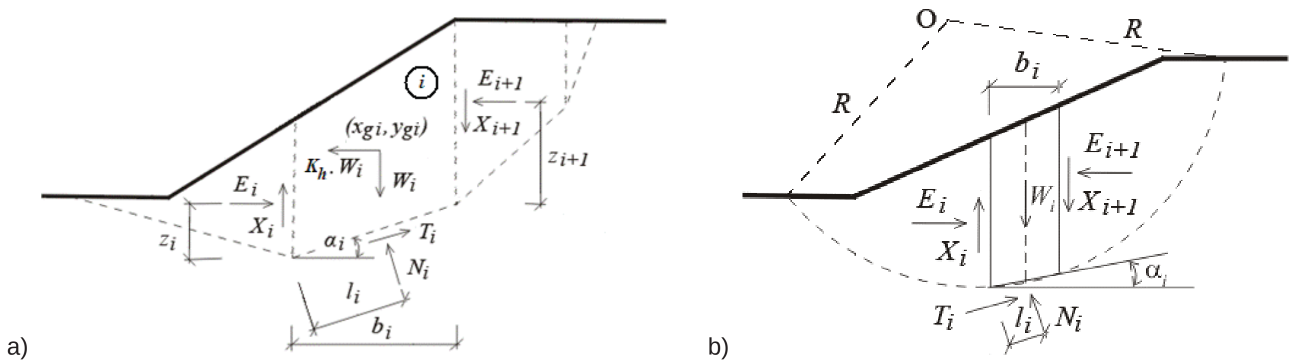
3.2 Analitičke metode proračuna stabilnosti klizišta

Ključni faktor u analizi klizišta jeste proračun stabilnosti klizišta, tako da se identifikuje da li je klizište u stanju ravnoteže, postoji li opasnost od gubitka ravnoteže ili nije u stanju ravnoteže. U opštem slučaju, kod analitičkih metoda stabilnosti klizišta, tlo se deli na vertikalne blokove, a za svaki blok se određuju odgovarajuće sile, pri čemu klizna površ može biti kružna ili poligonalna. U zavisnosti od matematičkog modela proračuna sila koje deluju između blokova i oblika blokova, postoji veliki broj razvijenih analitičkih metoda, od kojih su se u praksi i u nauci ustalile i izdvojile metode stabilnosti klizišta prema: *Sarma*-i, *Spencer*-u, *Janbu*, *Morgenstern-Price*-u, *Shahunyants*-u, *Bishop*-u, *Fellenius/Petterson*-u i tako dalje. Na slici 1 dat je šematski prikaz podele tla na blokove za opštu analizu stabilnosti kosine s poligonalnom i kružnom kliznom površi. Odgovarajuće sile za sve blokove glase: n normalnih sila N_i – koje deluju na svaki pojedinačan blok, n smičućih sila T_i – koje deluju po ivici klizne površi svakog pojedinačnog bloka, n-1 normalnih sila E_i – koje deluju između blokova, n-1 smičućih sila X_i – koje deluju između blokova, n-1 geometrijskih mesta z_i – na kojima deluju sile E_i i n geometrijskih mesta l_i – na kojima deluju sile N_i. Ukupno je 6n-2 nepoznatih koje treba odrediti iz 4n jednačina (uslova ravnoteže). Evidentno je da se 2n-2 nepoznatih mora ili aproksimirati ili unapred odrediti.

– analytical one-step,
– analytical step-by-step (iterations of sliding surfaces),
– numerical step-by-step (iterations of sliding surfaces),
– numerical incremental-iterative (nonlinear) analysis,
– numerical incremental-iterative (nonlinear) analysis, by applying numerical integration in the time domain.

3.2 Analytical methods of landslide stability calculation

The key factor in landslide analysis is landslide stability calculation, so as to identify whether the landslide is in the state of balance, whether there is a risk of its losing the balance, or if it is not in the state of balance. In general, with analytical landslide stability methods, the ground is divided into vertical blocks, and for each block corresponding forces are determined, whereby the sliding surface can be circular or polygonal. Depending on the mathematical model of the calculation of the forces acting between the blocks and the shapes of the blocks, there are many analytical methods developed, and the methods of landslide stability to which the practice and the science became accustomed with are those according to: *Sarma*, *Spencer*, *Janbu*, *Morgenstern-Price*, *Shahunyants*, *Bishop*, *Fellenius/Petterson* and the like. Figure 1 gives a schematic presentation of the ground division into blocks for general analysis of slope stability with a polygonal and a circular slide planes. The corresponding forces for all the blocks are: n normal forces $N_i$acting on each individual block, ns hear forces T_i which act on the edge of the slide plane of each individual block, n-1 normal forces E_i acting between the blocks, n-1 shear forces X_i which act between the blocks, n-1 geometric places z_i acted on by E_i forces and n the geometric places l_i where forces N_i act. In total, 6n-2 unknowns which should be determined from 4n equations (equilibrium conditions). It is obvious that 2n-2 unknowns have to be either approximated or predetermined.

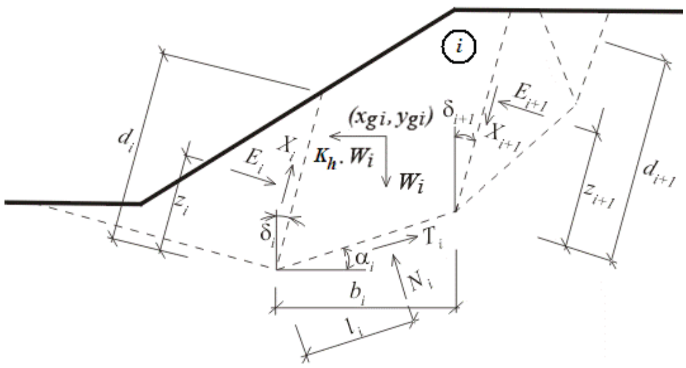


Slika 1. Podela tla na blokove za opštu analizu stabilnosti kosine: a) poligonalna klizna površ; b) kružna klizna površ [10]

Figure 1. Division of the ground into blocks for general analysis of slope stability: a) polygonal sliding surface, b) circular sliding surface [10]

Sarma-ina metoda zasniva se na podeli tla na blokove koji nisu strogo vertikalni, već imaju određeni ugao zakošenja, pri čemu su E_i i X_i normalne i smičuće sile između blokova, N_i i T_i – normalne i smičuće sile koje deluju po ivici klizne površi svakog pojedinačnog bloka, W_i – sopstvena težina bloka, K_hW_i – horizontalna sila kojom se obezbeđuje postizanje graničnog stanja [28]. K_h faktor predstavlja odnos horizontalnih i gravitacionih ubrzanja. Na slici 2 prikazana je podela tla na blokove za analizu stabilnosti kosine prema *Sarma*-inoj metodi.

The *Sarma* method is based on the division of the ground into blocks that are not strictly vertical, but rather have a certain inclination angle, where E_i and X_i are normal and shear forces between the blocks, N_i and T_i normal and shear forces acting on the edge of the sliding surface of each individual block, W_i the block's self weight, K_hW_i horizontal force which ensures reaching the limit state [28]. The K_h factor represents the ratio of horizontal and gravitational accelerations. Figure 2 shows the division of the ground into blocks for the analysis of slope stability according to the *Sarma* method.



Slika 2. Podela tla na blokove za analizu stabilnosti kosine prema Sarma-inoj metodi [28]
Figure 2. Division of the ground into blocks for slope stability analysis according to the Sarma method [28]

Algoritam proračuna stabilnosti kosine prema *Sarma*-inoj metodi zasniva se na jednačinama ravnoteže blokova:

The algorithm of the slope stability calculation according to the *Sarma* method is based on the balance of the blocks equations:

$$T_i\cos\alpha_i - N_i\sin\alpha_i = K_hW_i - F_{x,i} + X_{i+1}\sin\delta_i - X_i\sin\delta_i + E_{i+1}\cos\delta_i - E_i\cos\delta_i\,, \tag{1}$$

$$N_i\cos\alpha_i - T_i\sin\alpha_i = W_i - F_{y,i} + X_{i+1}\cos\delta_{i+1} - X_i\cos\delta_i - E_{i+1}\sin\delta_{i+1} + E_i\cos\delta_i\,, \tag{2}$$

$$N_il_i - X_{i+1}b_i\sec\alpha_i\cos(\alpha_i + \delta_{i+1}) + E_{i+1}\left[z_{i+1} + b_1\sec\alpha_i\sin(\alpha_i + \delta_{i+1})\right] - E_iz_i - W_i\left(x_{g,i} - x_i\right) +$$
$$+ K_hW_i\left(y_{g,i} - y_i\right) - F_{x,i}r_{x,i} + F_{y,i}r_{y,i} = 0\,, \tag{3}$$

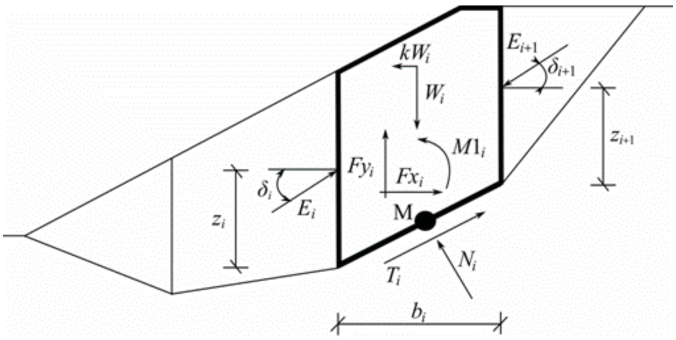
$$T_i = \left(N_i - U_i\right)\mathrm{tg}\varphi_i + c_1b_i\sec\alpha_i\,, \qquad X_i = \left(E_i - PW_i\right)\mathrm{tg}\bar{\varphi}_i + \bar{c}_i d_i\,, \tag{4}$$

gde su $F_{x,i}$ i $F_{y,i}$ komponente horizontalne i vertikalne projekcije sila, $r_{x,i}$ i $r_{y,i}$ kraci $F_{x,i}$ i $F_{y,i}$ sila, respektivno, PW_i rezultanta sile pornog pritiska na podeljene blokove, $\bar{\varphi}_i$ prosečna vrednost ugla unutrašnjeg trenja duž klizne površine pojedinih blokova, $\bar{c}_i$ prosečna vrednost kohezije duž klizne površine pojedinih blokova. Faktor sigurnosti kosine F_s određuje se iterativno, redukujući parametre c i tgφ, tako da se dostigne vrednost faktora K_h (nula ili veća od nule).

Spencer-ova metoda zasniva se na graničnoj ravnoteži kosine, uspostavljanjem ravnoteže sila i momenata koji deluju na pojedine blokove [30]. Na slici 3 prikazana je podela tla na blokove za analizu stabilnosti kosine prema *Spencer*-ovoj metodi.

where $F_{x,i}$ and $F_{y,i}$ are components of the horizontal and vertical forces projections, $r_{x,i}$ and $r_{y,i}$ arms of the forces $F_{x,i}$ and $F_{y,i}$, respectively, of the PW_i resultant of the force of the pore pressure to the divided blocks, $\bar{\varphi}_i$ the average angle value of the internal friction along the sliding surface of the individual blocks, $\bar{c}_i$ the average cohesion value along the sliding surface of individual blocks. The slope safety factor F_s is determined by iteratively reducing the parameters c and tgφ, so as to reach the factor K_h value (zero or greater than zero).

The *Spencer* method is based on the limit equilibrium of the slope, by reaching the balance of forces and moments acting on individual blocks [30]. Figure 3 shows the division of the ground into blocks for slope stability analysis according to the *Spencer* method.



Slika 3. Podela tla na blokove za analizu stabilnosti kosine prema Spencer-ovoj metodi [30]
Figure 3. Division of the ground into blocks for slope stability analysis according to the Spencer method [30]

S ciljem postizanja rešenja problema granične ravnoteže kosine, koja je podeljena na blokove, uvedene su određene pretpostavke: ravni – kojima su podeljeni blokovi – ostaju vertikalne i tokom proračuna, linija dejstva sopstvene težine bloka W_i prolazi kroz centar *i*-tog segmenta klizne površi i predstavlja se tačkom *M*, normalna sila N_i deluje u centru *i*-tog segmenta klizne površi u tački *M* i ugao dejstva sile E_i, koja deluje između blokova, jeste konstantan za sve blokove i jednak je *δ*. Algoritam proračuna stabilnosti kosine prema *Spencer*-ovoj metodi zasniva se na izrazima:

In order to achieve a solution to the problem of the limit equilibrium of the slope, which is divided into blocks, certain assumptions have been made: the planes, which divide the blocks, remain vertical during the calculations as well, the line of action of the block's self weight W_i passes through the centre of the *i*-th segment of the sliding surface and it's represented as the point *M*, the normal force N_i acts in the centre of the *i*-th segment of the slide plane at the point *M* and the angle of action of the force E_i, which acts between the blocks, is constant for all the blocks and equals δ. The algorithm of the slope stability calculation according to the *Spencer* method is based on the following expressions:

$$N_i = N_i' + U_i \,, \tag{5}$$

$$T_i = \left(N_i - U_i\right)\mathrm{tg}\varphi_i + \frac{b_i}{\cos\alpha_i} = N_i'\mathrm{tg}\varphi_i + c_i\frac{b_i}{\cos\alpha_i} \,, \tag{6}$$

$$N_i' + U_i - W_i\cos\alpha_i + K_h W_i\sin\alpha_i + F_{y,i}\cos\alpha_i - F_{x,i}\sin\alpha_i + E_{i+1}\sin\left(\alpha_i - \delta_{i+1}\right) - E_i\sin\left(\alpha_i - \delta_i\right) = 0 \,, \tag{7}$$

$$N_i'\frac{\mathrm{tg}\varphi_i}{F_s} + \frac{c_i b_i}{F_s\cos\alpha_i} - W_i\sin\alpha_i - K_h W_i\cos\alpha_i + F_{y,i}\sin\alpha_i + F_{x,i}\cos\alpha_i - E_{i+1}\cos\left(\alpha_i - \delta_{i+1}\right) +$$

$$+ E_i\cos\left(\alpha_i - \delta_i\right) = 0 \,, \tag{8}$$

$$E_{i+1}\cos\delta_{i+1}\left(z_{i+1} - \frac{b_i}{2}\mathrm{tg}\alpha_i\right) - E_{i+1}\sin\delta_{i+1}\frac{b_i}{2} - E_i\cos\delta_i\left(z_i - \frac{b_i}{2}\mathrm{tg}\alpha_i\right) - E_i\sin\delta_i\frac{b_i}{2} +$$

$$+ M1_i - K_h W_i\left(y_M - y_{g,i}\right) = 0 \,, \tag{9}$$

gde je U_i rezultanta pornog pritiska na za *i*-ti segment klizne površi, $M1_i$ – momenat sila F_x i F_y oko tačke *M*. Izraz (5) predstavlja relaciju između efektivne i totalne vrednosti normalnih sila koje deluju duž klizne površi. Izraz (6) predstavlja relaciju između normalnih i smičućih sila segmenta klizne površi (*Mohr-Coulomb*-ovi uslovi). Preformulacijom izraza (7) i (8) dobija se:

where U_i is the resultant of the pore pressure for the *i*-th segment of the slide plane, $M1_i$ is the moment of forces F_x and F_y around the point *M*. The expression (5) represents the relation between the effective and the total value of the normal forces acting along the sliding surface. The expression (6) represents the relation between the normal and shear forces of the sliding surface segment (*Mohr-Coulomb* conditions). By reformulating the expressions (7) and (8), we get:

$$E_{i+1}=\frac{\left[\left(W_i-F_{y,i}\right)\cos\alpha_i-\left(K_hW_i-F_{x,i}\right)\sin\alpha_i-U_i+E_i\sin\left(\alpha_i-\delta_i\right)\right]\frac{tg\varphi_i}{F_s}+}{\sin\left(\alpha_i-\delta_{i+1}\right)\frac{tg\varphi_i}{F_s}+\cos\left(\alpha_i-\delta_{i+1}\right)}$$

$$\frac{+\frac{c_ib_i}{F_s\cos\alpha_i}-\left(W_i-F_{y,i}\right)\sin\alpha_i-\left(K_hW_i-F_{x,i}\right)\cos\alpha_i+E_i\cos\left(\alpha_i-\delta_i\right)}{}. \tag{10}$$

Primenom izraza (10) mogu se odrediti sve sile E_i koje deluju između blokova za date vrednosti δ_i i F_s. Preformulacijom izraza (9), dobija se:

By applying the expression (10), all the forces E_i acting between the blocks for the given values δ_i and F_s can be determined. By reformulating expression (9) we get:

$$z_{i+1}=\frac{\frac{b_i}{2}\left[E_{i+1}\left(\sin\delta_{i+1}-\cos\delta_{i+1}\mathrm{tg}\alpha_i\right)+E_i\left(\sin\delta_i-\cos\delta_i\mathrm{tg}\alpha_i\right)\right]+E_iz_i\cos\delta_i-M1_i+K_hW_i\left(y_M-y_{g,i}\right)}{E_{i+1}\left(\cos\delta_{i+1}\right)}. \tag{11}$$

Primenom izraza (11), mogu se odrediti svi kraci sile z za date vrednosti ugla δ_i. Faktor sigurnosti F_s određuje se primenom iterativnog algoritma: inicijalna vrednost za ugao δ jeste δ=0, faktor sigurnosti F_s, za datu vrednost ugla δ, određuje se prema izrazu (10), imajući u vidu to što je E_{n+1}=0 na kraju klizne površi, ugao δ se određuje iz izraza (11), koristeći vrednosti za silu E – koja je određena iz prethodnog koraka analize, pri čemu je vrednost z_{n+1}=0 i prethodna dva koraka analize iterativno se ponavljaju sve dok vrednost ugla δ, u dve uzastopne iteracije, ne postane jednaka. Da bi algoritam iteracija bio dovoljno stabilan, potrebno je intervenisati kako bi se otklonila nestabilna rešenja. Ove nestabilnosti javljaju se kada se u izrazima (10) i (11) pojave situacije deljenja s nulom. U izrazu (11) ovakva situacija može se pojaviti za vrednosti ugla $\delta=\pi/2$ ili $\delta=-\pi/2$, pa se rešenje mora tražiti za interval ugla $\delta=[-\pi/2;\pi/2]$. Deljenje s nulom u izrazu (10) pojavljuje se u slučaju:

By applying the expression (11) all the moment arms of the force z for the given values of the angle δ_i can be determined. The safety factor $F_s$is determined using an iterative algorithm: the initial value for the angle δ is δ=0, the safety factor F_s for the given value of the angle δ is determined according to the expression (10), taking into account that E_{n+1}=0 at the end of the sliding surface, the angle δ is determined from the expression (11) using the values for the force E, which is determined from the previous step of the analysis, where the value z_{n+1}=0 and the previous two steps of the analysis are repeated iteratively until the value of the angle δ, during two consecutive iterations, becomes equal. In order for the iteration algorithm to be stable enough, it is necessary to intervene with the aim of eliminating any unstable solutions. These instabilities occur when expressions (10) and (11) show the situation of the division by zero. In expression (11) such a situation can occur for the values of the angle $\delta=\pi/2$ or $\delta=-\pi/2$, so the solution should be sought for the interval of the angle $\delta=[-\pi/2;\pi/2]$. Division by zero in expression (10) appears in the case of:

$$F_s=\mathrm{tg}\varphi_i\mathrm{tg}\left(\delta_{i+1}-\alpha_i\right). \tag{12}$$

Radi sprečavanja nestabilnosti rešenja, potrebno je sprovesti proveru parametra m_α prema izrazu:

In order to prevent the solution instabilities, it is necessary to perform a parameter check m_α according to the expression:

$$m_\alpha=\cos\alpha_i+\frac{\sin\alpha_i\mathrm{tg}\varphi_i}{F_s}>0.2. \tag{13}$$

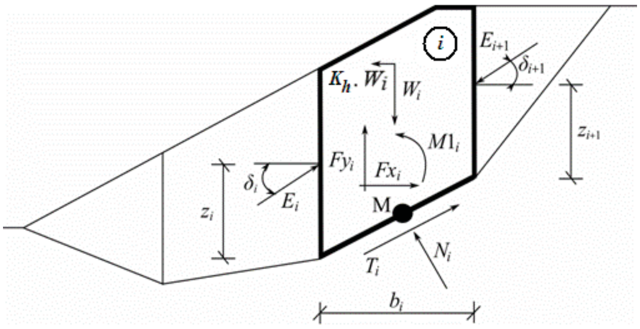
Pre nego što se započne sa iterativnom analizom, potrebno je pronaći najveću kritičnu vrednost $F_{s,min}$ koja zadovoljava prethodne uslove. Vrednosti faktora sigurnosti F_s koje su ispod ove kritične vrednosti $F_{s,min}$ pripadaju oblasti nestabilnog rešenja. Prva iteracija započinje s vrednošću faktora sigurnosti F_s koja je tek nešto veća od $F_{s,min}$, tako da su i preostale vrednosti faktora sigurnosti F_s – koje se određuju proračunom – uvek veće od $F_{s,min}$.

Janbu-ova metoda jeste procedura verifikacije

Before beginning iterative analysis, it is necessary to find the highest critical value of $F_{s,min}$ that satisfies the previous conditions. The values of the safety factors F_s below this critical value $F_{s,min}$ belong to the area of unstable solution. The first iteration starts with the value of the safety factor F_s, which is just slightly higher than $F_{s,min}$, so the remaining values of the safety factors F_s, which are determined by the calculation, are always higher than $F_{s,min}$.

The *Janbu*'s method is a procedure of verifying the stabilnosti granične ravnoteže kosina, a zasniva se na uspostavljanju ravnoteže sila i momenata koji deluju na pojedine blokove [19]. Na slici 4 prikazana je podela tla na blokove za analizu stabilnosti kosine prema *Janbu*-ovoj metodi.

stability of the slopes' limit equilibrium, and it is based on establishing the balance between forces and moments acting on individual blocks [19]. Figure 4 shows the division of the ground into blocks for slope stability analysis according to *Janbu*'s method.



Slika 4. Podela tla na blokove za analizu stabilnosti kosine prema Janbu-ovoj metodi [19]
Figure 4. Division of the ground into blocks for slope stability analysis according to the Janbu's method [19]

Radi postizanja rešenja problema granične ravnoteže kosine koja je podeljena na blokove, uvedene su određene pretpostavke: ravni – kojima su podeljeni blokovi – ostaju vertikalne i tokom proračuna, linija dejstva sopstvene težine bloka W_i prolazi kroz centar i-tog segmenta klizneərši i predstavlja se tačkom M, normalna sila N_i deluje u centru i-tog segmenta klizne površi u tački M i vertikalna pozicija z_i dejstva sile E_i, koja deluje između blokova, jednaka je nuli za krajnje tačke klizne površi. Izbor vertikalne pozicije z_i dejstva sile E_i ima značajan uticaj na dobijanje konvergentnog rešenja. Ukoliko se loše pretpostave vertikalne pozicije z_i, može nastupiti divergencija rešenja, uz prethodno znatno povećanje vremena proračuna. Vertikalne pozicije z_i dejstava sila E_i usvajaju se da su jednaki trećini visine blokova na koje je podeljena kosina. Ukoliko nastupi divergencija rešenja, potrebno je korigovati vrednosti z_i, tako što se one blago povećavaju kod blokova pasivne zone (kod nožice kosine) i blago smanjuju kod blokova aktivne zone (kod vrha kosine). Algoritam proračuna stabilnosti kosine prema *Janbu*-ovoj metodi zasniva se na izrazima:

In order to reach a solution to the problem of the limit equilibrium of the slope, which is divided into blocks, certain assumptions have been made: the planes which divide the blocks, remain vertical during the calculation as well, the line of action of the block's self weight W_i passes through the centre of the i-th segment of the sliding surface at the point M, the normal force N_i acts in the centre of the i-th segment of the slide plane at the point M and the vertical position z_i of the action of the force E_i, which acts between the blocks, is equal to zero for the end points of the sliding surface. The choice of the vertical position z_i of the effect of the force E_i has a significant influence on obtaining a convergent solution. If the vertical positions of z_i are inaccurately assumed, divergence of the solution can occur, with a significant increase in the calculation time. Vertical positions of z_i action of the forces E_i are assumed to be equal 1/3 of the blocks height, to which the slopes are divided. If there a divergence of the solution occurs, it is necessary to correct the z_i values, by slightly increasing them with the passive zone blocks (at the foot of the slope) and slightly decreasing them with the blocks of the active zone (at the top of the slope). The algorithm of the slope stability calculation according to the *Janbu*'s method is based on the expressions:

$$N_i = N_i' + U_i, \tag{14}$$

$$T_i = (N_i - U_i)\mathrm{tg}\varphi_i + \frac{b_i}{\cos\alpha_i} = N_i'\mathrm{tg}\varphi_i + c_i\frac{b_i}{\cos\alpha_i}, \tag{15}$$

$$N_i' + U_i - W_i\cos\alpha_i + K_h W_i\sin\alpha_i + F_{y,i}\cos\alpha_i - F_{x,i}\sin\alpha_i + E_{i+1}\sin(\alpha_i - \delta_{i+1}) - E_i\sin(\alpha_i - \delta_i) = 0, \tag{16}$$

$$N_i'\frac{\mathrm{tg}\varphi_i}{F_s} + \frac{c_i b_i}{F_s\cos\alpha_i} - W_i\sin\alpha_i - K_h W_i\cos\alpha_i + F_{y,i}\sin\alpha_i + F_{x,i}\cos\alpha_i - E_{i+1}\cos(\alpha_i - \delta_{i+1}) +$$

$$+ E_i\cos(\alpha_i - \delta_i) = 0, \tag{17}$$

$$E_{i+1}\cos\delta_{i+1}\left(z_{i+1}-\frac{b_i}{2}\,\mathrm{tg}\alpha_i\right)-E_{i+1}\sin\delta_{i+1}\,\frac{b_i}{2}-E_i\cos\delta_i\left(z_i-\frac{b_i}{2}\,\mathrm{tg}\alpha_i\right)-E_i\sin\delta_i\,\frac{b_i}{2}+$$

$$+M1_i-K_hW_i\left(y_M-y_{g,i}\right)=0\,. \qquad (18)$$

Preformulacijom izraza (16) i (17) dobija se:

Reformulation of the expressions (16) and (17) gives:

$$E_{i+1}=\frac{\dfrac{\left[\left(W_i-F_{y,i}\right)\cos\alpha_i-\left(K_hW_i-F_{x,i}\right)\sin\alpha_i-U_i+E_i\sin\left(\alpha_i-\delta_i\right)\right]\dfrac{tg\varphi_i}{F_s}+}{\sin\left(\alpha_i-\delta_{i+1}\right)\dfrac{tg\varphi_i}{F_s}+\cos\left(\alpha_i-\delta_{i+1}\right)}}{}$$

$$\frac{+\dfrac{c_ib_i}{F_s\cos\alpha_i}-\left(W_i-F_{y,i}\right)\sin\alpha_i-\left(K_hW_i-F_{x,i}\right)\cos\alpha_i+E_i\cos\left(\alpha_i-\delta_i\right)}{}\,. \qquad (19)$$

Preformulacijom izraza (18) dobija se:

Reformulation of the expression (18) gives:

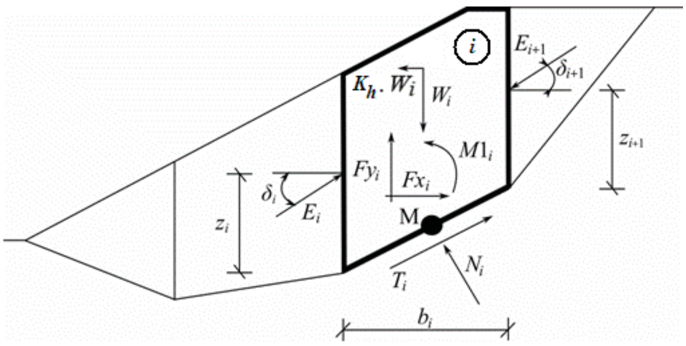
$$\delta_{i+1}=\mathrm{arctg}\left(\frac{2z_{i+1}}{b_i}+\mathrm{tg}\alpha_i\right)-\arcsin\frac{E_i\left(\cos\delta_i\left(z_i-\dfrac{b_i\mathrm{tg}\alpha_i}{2}\right)+\sin\delta_i\,\dfrac{b_i}{2}\right)-M1_i}{E_{i+1}\sqrt{\left(z_{i+1}+\dfrac{b_itg\alpha_i}{2}\right)^2+\left(\dfrac{b_i}{2}\right)^2}}\,. \qquad (20)$$

Faktor sigurnosti F_s određuje se primenom iterativnog algoritma: inicijalne vrednosti svih uglova su δ_i=0 i pozicije z_i su usvojene da su jednake trećini visine blokova, faktor sigurnosti F_s, za datu vrednost ugla δ, određuje se prema izrazu (19), uzimajući u obzir da je E_{n+1}=0 na kraju klizne površi, ugao δ se određuje iz izraza (20) koristeći vrednosti za silu E, koja je određena iz prethodnog koraka analize i prethodna dva koraka analize iterativno se ponavljaju, sve dok vrednost ugla δ u dve uzastopne iteracije ne postane jednaka. Otklanjanje nestabilnih rešenja sprovodi se isto kao i u slučaju *Spencer*-ove metode.

Morgenstern-Price-ova metoda verifikacije stabilnosti granične ravnoteže kosina zasniva se na sličnom principu kao i metode *Spencer*-a i *Janbu*-a [26], [36]. Na slici 5 prikazana je podela tla na blokove za analizu stabilnosti kosine prema *Morgenstern-Price*-ovoj metodi.

The safety factor $F_s$is determined using an iterative algorithm: the initial values of all angles are δ_i=0 and the positions z_i are assumed to be equal to 1/3 of the blocks' height, the safety factor F_s for the given angle δ value, is determined according to the expression (19), taking into account that E_{n+1}=0 at the end of the sliding surface, the angle δ is determined from the expression (20) using the values for the force E, which is determined from the previous step of the analysis, and the previous two steps of the analysis are iteratively repeated until the value of the angle δ in two consecutive iterations is equal. Removing any unstable solutions is conducted in the same way as with *Spencer*'s method.

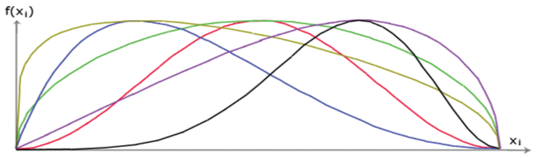
The *Morgenstern-Price*'s method for verifying the stability of the limit equilibrium of slopes is based on a principle similar to *Spencer*'s and *Janbu*'s methods [26], [36]. Figure 5 shows the division of the ground into blocks for the slope stability analysis according to the *Morgenstern-Price*'s method.



Slika 5. Podela tla na blokove za analizu stabilnosti kosine prema Morgenstern-Price-ovoj metodi [26]
Figure 5. Division of the soil into blocks for the slope stability analysis of according to the Morgenstern-Price's method [26]

S ciljem postizanja rešenja problema granične ravnoteže kosine koja je podeljena na blokove, uvedene su određene pretpostavke (slično *Spencer*-ovoj metodi): ravni, kojima su podeljeni blokovi, ostaju vertikalne i tokom proračuna, linija dejstva sopstvene težine bloka W_i prolazi kroz centar *i*-tog segmenta klizne površi i predstavlja se tačkom *M*, normalna sila N_i deluje u centru *i*-tog segmenta klizne površi u tački *M* i ugao dejstva sile E_i (koja deluje između blokova) je različit za sve blokove i jednak je δ=0 za krajnje tačke. Pretpostavka o vrednosti ugla δ_i uspostavlja se primenom polusinusne funkcije. Na slici 6 prikazan je spektar polusinusnih funkcija. Izbor oblika funkcije ima manjeg uticaja na kvalitet konačnog rešenja, ali optimalnim izborom oblika funkcije doprinosi se konvergenciji rešenja. Ugao δ_i određuje se multiplikacijom vrednosti polusinusne funkcije $f(x_i)$ i parametra λ.

In order to reach a solution to the problem of the limit equilibrium of a slope, which is divided into blocks, certain assumptions (similar to the *Spencer*'s method) have been made: the planes, which divide the blocks, remain vertical during the calculations as well, the line of action of the block's self weight W_i passes through the centre of the *i*-th segment of the sliding surface and it's represented as the point *M*, the normal force N_i acts in the centre of the *i*-th segment of the sliding surface at the point *M*, and the angle of action of the force E_i (acting between the blocks) is different for all the blocks and equals δ=0 for the end points. The assumption of the value of the angle δ_i is established by using the half-sine function. Figure 6 shows a spectrum of half-sine functions. The choice of the form of the function has less influence on the quality of the final solution, but with the choice of an appropriate form of the function, contributes to the convergence of the solution. The angle δ_i is determined by multiplying the value of the half-sine function $f(x_i)$ and the parameter λ.



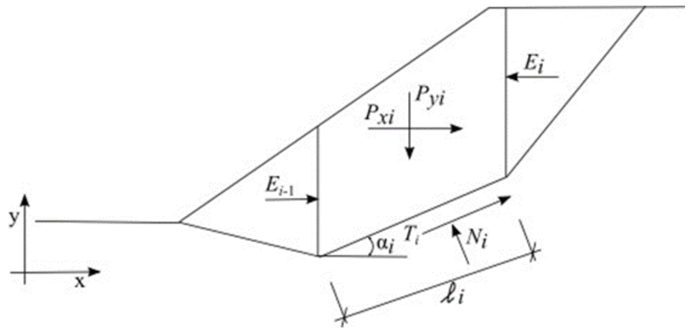
Slika 6. Polusinusna funkcija za pretpostavke o vrednosti ugla δ_i [26]
Figure 6. A half-sine function for assumptions about the value of the angle δ_i [26]

Algoritam proračuna stabilnosti kosine, prema *Morgenstern-Price*-ovoj metodi, zasniva se na izrazima koji su identični izrazima (5÷11) kod *Spencer*-ove metode. Faktor sigurnosti F_s određuje se primenom iterativnog algoritma: inicijalna vrednost uglova δ_i je $\delta_i=\lambda f(x_i)$, faktor sigurnosti F_s, za datu vrednost ugla δ, određuje se prema izrazu (10), uzimajući u obzir da je E_{n+1}=0 na kraju klizne površi,ugao δ se određuje iz izraza (11) koristeći vrednosti za silu *E*, koja je određena iz prethodnog koraka analize (z_{n+1}=0), pri čemu se vrednost polusinusne funkcije $f(x_i)$ zadržava kao konstantna kroz iteracije, a iterira se parametar λ i prethodna dva koraka analize iterativno se ponavljaju sve dok vrednost ugla δ u dve uzastopne iteracije ne postane jednaka. Kako bi se sprečila numerička nestabilnost rešenja, sprovode se kontrole prema izrazima (12) i (13).

Shahunyants-ova metoda verifikacije stabilnosti granične ravnoteže kosina zasniva se na sličnom principu kao i prethodne metode [31]. Na slici 7 prikazana je podela tla na blokove za analizu stabilnosti kosine prema *Shahunyants*-ovoj metodi. Radi postizanja rešenja problema granične ravnoteže kosine koja je podeljena na blokove, uvedene su određene pretpostavke: ravni, kojima su podeljeni blokovi, ostaju vertikalne tokom proračuna i ugao dejstva sile E_i, koja deluje između blokova, jednak je nuli (sile deluju horizontalno).

The algorithm of the slope stability calculation according to the *Morgenstern-Price*'s method is based on the expressions that are identical to expressions (5÷11) in the *Spencer*'s method. The safety factor F_s is determined by using an iterative algorithm: the initial value of the angles δ_i is $\delta_i=\lambda f(x_i)$, the safety factor F_s for the given value of the angle δ is determined according to the expression (10), taking into account that E_{n+1}=0 is at the end of the sliding surface, the angle δ is determined from the expression (11) using the values for the force *E*, which is determined from the previous step of the analysis (z_{n+1}=0), while the value of the half-sine function $f(x_i)$ is kept constant through iterations, and the parameter λ is iterated and the previous two steps of the analysis are iteratively repeated until the value of the angle δ is equal in two consecutive iterations. In order to prevent the numerical instability of the solution, controls are conducted according to the expressions (12) and (13).

The *Shahunyants*'s method for verifying the stability of the limit equilibrium of slopes is based on a similar principle as the previous methods [31]. Figure 7 shows the division of the ground into blocks for slope stability analysis according to the Shahunyants's method. In order to reach a solution to the problem of the limit equilibrium of the slope, which is divided into blocks, certain assumptions have been made: the planes, which divide the blocks, remain vertical during the calculation, and the angle of action of the force E_i, acting between the blocks, equals zero (the forces act horizontally).



Slika 7. Podela tla na blokove za analizu stabilnosti kosine prema Shahunyants-ovoj metodi [31]
Figure 7. Division of the ground into blocks for slope stability analysis according to the Shahunyants's method [31]

Algoritam proračuna stabilnosti kosine prema *Shahunyants*-ovoj metodi započinje transformacijom sila $P_{x,i}$ i $P_{y,i}$ u pravcu normale (*N*) i tangente (*T*) klizne površi:

The algorithm of the slope stability calculation according to the *Shahunyants*'s method begins with the transformation of the forces $P_{x,i}$ and $P_{y,i}$ in the direction of the normal (*N*) and the tangent (*T*) of the sliding surface:

$$P_{N,i} = P_{x,i}\sin\alpha_i + P_{y,i}\cos\alpha_i \, , \tag{21}$$

$$P_{Q,i} = P_{y,i}\sin\alpha_i - P_{x,i}\cos\alpha_i \, . \tag{22}$$

Sile koje deluju duž segmenata klizne površi proračunavaju se prema:

The forces acting along the sliding surface segments are calculated according to:

$$T_i = \left(N_i - U_i\right)\mathrm{tg}\varphi_i + c_i l_i \, . \tag{23}$$

Jednačina ravnoteže upravno na ravan segmenta klizne površi glasi:

The equation of equilibrium perpendicular to the plane of the sliding surface segment is:

$$N_i = P_{N,i} + E_{i-1}\sin\alpha_i - E_i\sin\alpha_i \, , \tag{24}$$

dok jednačina ravnoteže u ravni segmenta klizne površi glasi:

while the equation of equilibrium in the plane of the sliding surface segment is:

$$T_i = P_{Q,i} + E_i\cos\alpha_i - E_{i-1}\cos\alpha_i \, . \tag{25}$$

Uvođenjem izraza (23) u (25) dobija se:

By introducing the expression (23) into (25), we get:

$$\left(N_i - U_i\right)\mathrm{tg}\varphi_i + c_i l_i = P_{Q,i} + E_i\cos\alpha_i - E_{i-1}\cos\alpha_i \, , \tag{26}$$

dok se uvođenjem izraza (24) u (26) dobija:

whereas, by introducing the expression (24) into (26), we get:

$$\left(P_{N,i} + E_{i-1}\sin\alpha_i - E_i\sin\alpha_i - U_i\right)\mathrm{tg}\varphi_i + c_i l_i = P_{Q,i} + E_i\cos\alpha_i - E_{i-1}\cos\alpha_i \, . \tag{27}$$

Nakon sređivanja izraza (27), dobija se:

After arranging the expression (27), we get:

$$\left(P_{N,i} - U_i\right)\mathrm{tg}\varphi_i + \left(E_{i-1} - E_i\right)\sin\alpha_i\mathrm{tg}\varphi_i + c_i l_i = P_{Q,i} + \left(E_i - E_{i-1}\right)\cos\alpha_i \, , \tag{28}$$

odnosno:

i.e.:

$$\left(P_{N,i} - U_i\right)\mathrm{tg}\varphi_i + c_i l_i - P_{Q,i} = \left(E_i - E_{i-1}\right)\left(\cos\alpha_i + \sin\alpha_i\mathrm{tg}\varphi_i\right). \tag{29}$$

S obzirom na to što je:

Taking into the account the following:

$$\cos\alpha + \sin\alpha\,\mathrm{tg}\beta = \frac{\cos\alpha\cos\beta + \sin\alpha\sin\beta}{\cos\beta} = \frac{\cos(\alpha - \beta)}{\cos\beta} \, , \tag{30}$$

dobija se da je izraz (29):

we get that the expression (29) is:

$$\left(P_{N,i}-U_i\right)\mathrm{tg}\varphi_i+c_i l_i-P_{Q,i}=\left(E_i-E_{i-1}\right)\frac{\cos\left(\alpha_i-\varphi_i\right)}{\cos\varphi_i}, \tag{31}$$

a dodatnom modifikacijom izraza (31) dobija se:

and the additional modification of the expression (31) gives:

$$\left(P_{N,i}-U_i\right)\mathrm{tg}\varphi_i+c_i l_i-P_{Q,i}+E_{i-1}\frac{\cos\left(\alpha_i-\varphi_i\right)}{\cos\varphi_i}=E_i\frac{\cos\left(\alpha_i-\varphi_i\right)}{\cos\varphi_i}. \tag{32}$$

Primenom izraza (32) sile koje deluju između blokova E_i određuju se prema:

By applying the expression (32), the forces action between the blocks E_i are determined according to:

$$E_i=\frac{\left[\left(P_{N,i}-U_i\right)\mathrm{tg}\varphi_i+c_i l_i-P_{Q,i}\right]\cos\varphi_i}{\cos\left(\alpha_i-\varphi_i\right)}+E_{i-1}. \tag{33}$$

Sada se u proračun stabilnosti kosine uvodi faktor sigurnosti F_s, dok se $P_{Q,i}$ sile razlažu na sile koje doprinose klizanju $P_{Q,i,sd}$ (aktivne sile) i sile koje ne doprinose klizanju $P_{Q,i,ud}$ (stabilizujuće sile):

Now, the safety factor F_s is introduced into the slope stability calculation, while the $P_{Q,i}$ forces are broken down into the forces contributing to the sliding $P_{Q,i,sd}$ (active forces) and the forces that do not contribute to sliding $P_{Q,i,ud}$ (stabilizing forces):

$$E_i=\frac{\left[\left(P_{N,i}-U_i\right)\mathrm{tg}\varphi_i+c_i l_i-F_s P_{Q,i,sd}-P_{Q,i,ud}\right]\cos\varphi_i}{\cos\left(\alpha_i-\varphi_i\right)}+E_{i-1}. \tag{34}$$

$P_{Q,i}$ je pozitivno kada doprinosi klizanju kosine, a negativno kada ne doprinosi klizanju kosine, tako da se izraz (34) može pisati u formi:

$P_{Q,i}$ is positive when it contributes to the sliding of the slope, and negative when it does not contribute to the sliding of the slope, hence, the expression (34) can be written in the form:

$$E_i=\frac{\left[\left(P_{N,i}-U_i\right)\mathrm{tg}\varphi_i+c_i l_i-F_s P_{Q,i,sd}+\left|P_{Q,i,ud}\right|\right]\cos\varphi_i}{\cos\left(\alpha_i-\varphi_i\right)}+E_{i-1}. \tag{35}$$

Na kliznoj površi vrednost sile E_0 jednaka je nula, dok za E_1 važi:

On the sliding surface, the value of the force E_0 equals zero, whereas the following applies to E_1:

$$E_1=\frac{\left[\left(P_{N,1}-U_1\right)\mathrm{tg}\varphi_1+c_1 l_1-F_s P_{Q,1,sd}+\left|P_{Q,1,ud}\right|\right]\cos\varphi_1}{\cos\left(\alpha_1-\varphi_1\right)}, \tag{36}$$

a za E_2:

and to E_2:

$$E_2=\frac{\left[\left(P_{N,2}-U_2\right)\mathrm{tg}\varphi_2+c_2 l_2-F_s P_{Q,2,sd}+\left|P_{Q,2,ud}\right|\right]\cos\varphi_2}{\cos\left(\alpha_2-\varphi_2\right)}+ \\ +\frac{\left[\left(P_{N,1}-U_1\right)\mathrm{tg}\varphi_1+c_1 l_1-F_s P_{Q,1,sd}+\left|P_{Q,1,ud}\right|\right]\cos\varphi_1}{\cos\left(\alpha_1-\varphi_1\right)}. \tag{37}$$

Slično se mogu prikazati i izrazi za sve sile koje deluju između blokova, pri čemu je E_n=0:

The expressions for all the forces acting between the blocks can be presented in a similar way, where E_n=0:

$$E_n=\sum_{i=1}^{n}\left[\left(P_{N,i}-U_i\right)\mathrm{tg}\varphi_i+c_i l_i+\left|P_{Q,i,ud}\right|\right]\frac{\cos\varphi_i}{\cos\left(\alpha_i-\varphi_i\right)}+F_s\sum_{i=1}^{n}P_{Q,i,sd}\frac{\cos\varphi_i}{\cos\left(\alpha_i-\varphi_i\right)}=0, \tag{38}$$

tako da se iz ovog izraza može direktno prikazati faktor sigurnosti F_s u formi:

so that, from this expression, the safety factor F_s can be directly presented in the following form:

$$F_s = \frac{\sum_{i=1}^{n}\left[\left(P_{N,i} - U_i\right)\mathrm{tg}\varphi_i + c_i l_i + \left|P_{Q,i,ud}\right|\right]\frac{\cos\varphi_i}{\cos(\alpha_i - \varphi_i)}}{\sum_{i=1}^{n} P_{Q,i,sd}\frac{\cos\varphi_i}{\cos(\alpha_i - \varphi_i)}}. \quad (39)$$

Faktor sigurnosti F_s prema *Fellenius/Petterson*-ovoj metodi određuje se na osnovu izraza:

The safety factor F_s, according to the *Fellenius/Petterson*'s method, is determined on the basis of the expression:

$$F_s = \frac{1}{\sum_i W_i \sin\alpha_i}\sum_i\left[c_i l_i + \left(N_i - u_i l_i\right)\mathrm{tg}\varphi_i\right], \quad (40)$$

dok se prema *Bishop*-ovoj metodi određuje na osnovu izraza:

whereas, according to *Bishop*'s method, it is determined on the basis of the expression:

$$F_s = \frac{1}{\sum_i W_i \sin\alpha_i}\sum_i \frac{c_i b_i + \left(W_i - u_i b_i\right)\mathrm{tg}\varphi_i}{\cos\alpha_i + \frac{\mathrm{tg}\varphi_i \sin\alpha_i}{F_s}}. \quad (41)$$

3.3 Numeričke metode proračuna stabilnosti klizišta

Proračun stabilnosti klizišta numeričkim metodama zasniva se na metodama diskretizacije domena, kao što su:

- metoda konačnih elemenata (FEM – *Finite Element Method*);
- proširena metoda konačnih elemenata (XFEM – *eXtended Finite Element Method*);
- metoda graničnih elemenata (BEM – *Boundary Element Method*);
- metoda diskretnih elemenata (DEM – *Discrete Element Method*);
- metoda konačnih razlika (FDM – *Finite Difference Method*).

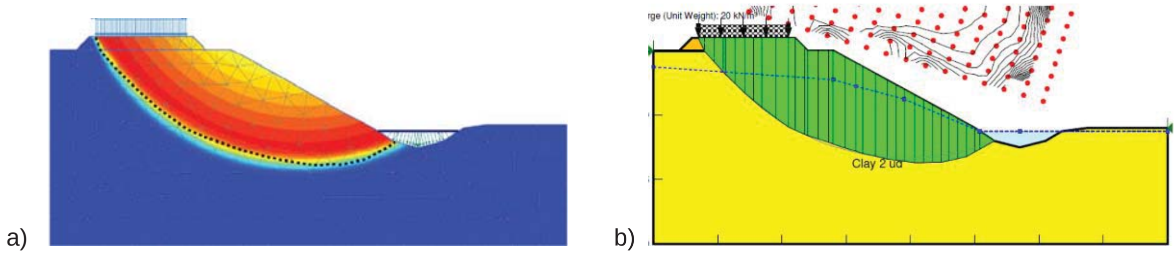
U ovim metodama, tlo se razmatra kao linearno-elastičan, elasto-plastičan i nelinearan materijal. Metoda konačnih elemenata (FEM) najčešće se upotrebljava za rešavanje problema numeričke analize stabilnosti kosina, tako da veliki broj softvera ima implementirane algoritme zasnovane na ovoj metodi. Na slici 8 prikazana je mreža konačnih elemenata diskretnog numeričkog modela kosine i skup tačaka dobijenih optimizacijom faktora sigurnosti kosine prema metodi konačnih elemenata (FEM). Kosina se modelira primenom površinskih konačnih elemenata sa integrisanom matematičkom formulacijom za analizu ravnog stanja deformacija (*plane strain*). Prilikom modeliranja i analize stabilnosti kosina, potrebno je imati u vidu dva bitna aspekta: diskretizaciju i aproksimaciju. Diskretizacija se odnosi na problem podele domena tla na konačne elemente dovoljno malih dimenzija za koje se moraju poštovati kriterijumi odnosa dijagonala i uglova četvorougaonog konačnog elementa ili odnosi stranica traouganog konačnog elementa. U oblasti kontakta tla sa elementima za plitko ili duboko fundiranje, koji se koriste prilikom sanacije klizišta, potrebno je izvršiti progušćenje mreže konačnih elemenata. Takođe, progušćenje se sprovodi i u zoni klizne površi, na mestima diskontinuiteta i otvora u tlu i slično.

3.3 Numerical methods of landslide stability calculations

Landslide stability calculation using numerical methods is based on methods of domain discretization, such as:

- *Finite Element Method* (FEM),
- *eXtended Finite Element Method* (XFEM),
- *Boundary Element Method* (BEM),
- *Discrete Element Method* (DEM),
- *Finite Difference Method* (FDM).

In these methods, the soil is considered as a linear-elastic, elasto-plastic and non-linear material. The *Finite Element Method* (FEM) is mostly used for solving the problem of numerical slope stability analysis, so a large number of software has implemented algorithms based on this method. Figure 8 shows the mesh of finite elements of the discrete numerical model of the slope and the set of points obtained by optimizing the slope safety factor according to the *Finite Element Method* (FEM). The slope is modelled by using surface finite elements with an integrated mathematical formulation for the analysis of the plane strain. When modelling and analyzing slope stability, two important aspects need to be taken into account: discretization and approximation. Discretization refers to the problem of the ground domain division into finite elements of sufficiently small dimensions for which the criteria of the relation between the diagonal and the angles of the quadrangle finite element or the relations of the sides of the triangle finite element must be respected. In the area of the contact between the ground and the elements for shallow or deep foundation, which are used during the landslide repair, it is necessary to increase the density of the mesh of finite elements. In addition, the increase in density realized in the sliding surface area as well, at discontinuity points and in the openings in the ground, and the like.

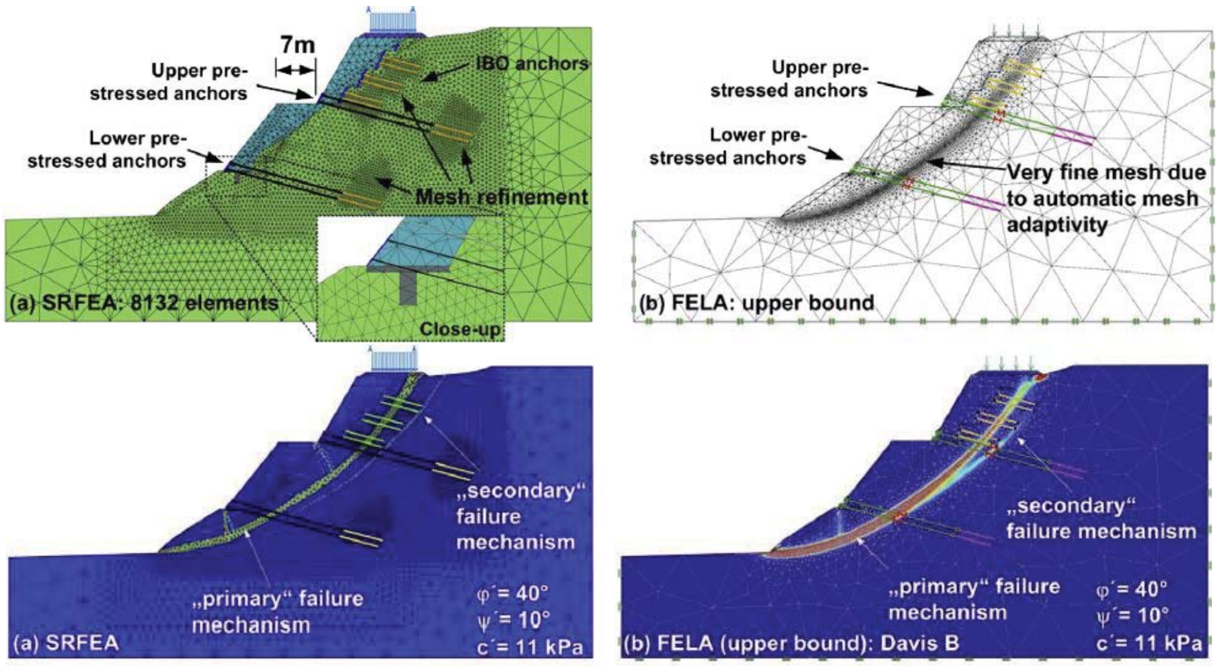


Slika 8. 2D numerički model kosine: a) mreža konačnih elemenata diskretnog numeričkog modela kosine prema metodi konačnih elemenata (FEM); b) skup tačaka dobijenih optimizacijom faktora sigurnosti kosine prema metodi konačnih elemenata (FEM)[12]

Figure 8. 2D numerical model of a slope: a) a finite elements mesh of the discrete numerical model of a slope according to the Finite Element Method (FEM); b) a set of points obtained by optimizing the slope safety factor according to the Finite Element Method (FEM) [12]

Uspostavljanje veze osnovnih konačnih elemenata koji formiraju domen tla, s progušćenom mrežom konačnih elemenata, sprovodi se primenom prelaznih elemenata. Kao prelazni elementi, najčešće se primenjuju trougaoni konačni elementi. Veoma bitan aspekt jeste i uspostavljanje kompatibilnosti čvorova konačnih elemenata, analizom konformnosti/nekonformnosti, posebno kod prelaznih konačnih elemenata, pri čemu se ne sme dozvoliti da određeni čvorovi, u kombinaciji osnovnih i prelaznih konačnih elemenata, ostanu nepovezani ili parcijalno povezani. Na slici 9 prikazani su 2D numerički modeli kosina, s generisanim mrežama konačnih elemenata i progušćenjima po selektovanim domenima.

Establishing a connection between the basic finite elements, which form the domain of the ground, with the increased density mesh of finite elements is carried out by using transition elements. As transition elements, the most commonly used are triangular finite elements. A very important aspect is also establishing the compatibility of finite elements nodes through conformity/non-conformity analysis, especially with transition finite elements, whereby it should not be allowed for certain nodes, in combination of basic and transition finite elements, to be left unconnected or partially connected. Figure 9 shows the 2D numerical slope models with generated finite element mesh and increased density over selected domains.

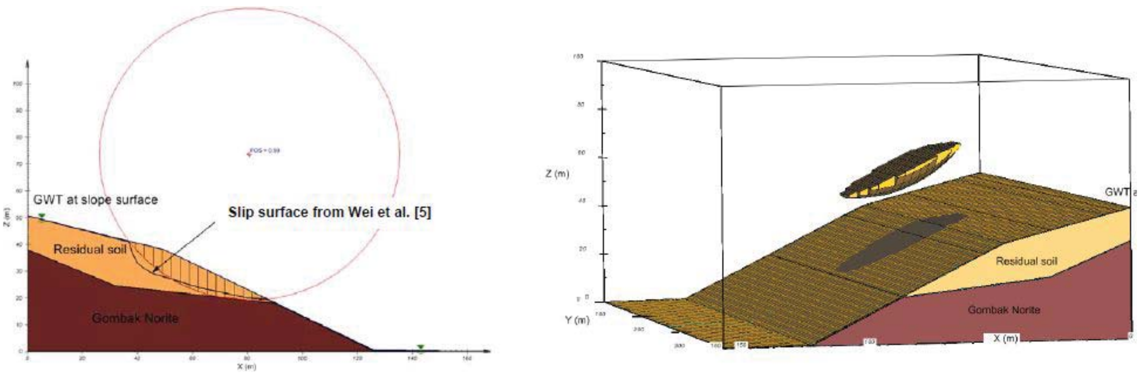


Slika 9. 2D numerički modeli kosina s generisanim mrežama konačnih elemenata i progušćenjima po selektovanim domenima [32]

Figure 9. 2D numerical slope models with generated finite element mesh and increased density over selected domains [32]

U odnosu na 2D model kosine, koji se i najviše koristi u praktične svrhe, primenom 3D modela kosine mogu se modelirati kompleksniji geometrijski modeli s prostorno složenijom i promenljivijom geologijom na manjem prostoru. Na slici 10 prikazani su 2D i 3D numerički modeli kosine, sa izdvojenim prikazom klizne mase tla i prostornim modelom klizne površi. Za modeliranje 3D modela kosina koriste se prizmatični (*solid*) ili tetra-edarski konačni elementi, pri čemu modeliranje domena tla prostornim konačnim elementima zahteva znatnije hardverske kapacitete. Kod prizmatičnih konačnih elemenata, primenjuje se minimalno 2x2x2 numerička integracija preko *Gaussian*-ovih kvadratura [8].

Compared to the 2D model of the slope, which is the one mostly used for practical purposes, by using the 3D model of the slope, more complex geometric models can be modelled, with a spatially more complex and significantly more variable geology in a smaller area. Figure 10 shows the 2D and 3D numerical models of the slope with a separate representation of the sliding mass of the soil and the spatial model of the sliding surface. For the modelling of the 3D model of slopes, solid or tetrahedral finite elements are used, whereby modelling of the ground domain by spatial finite elements requires significantly higher hardware capacities. With prismatic finite elements, a minimum of 2x2x2 numerical integration is applied over *Gaussian* quadratures [8].

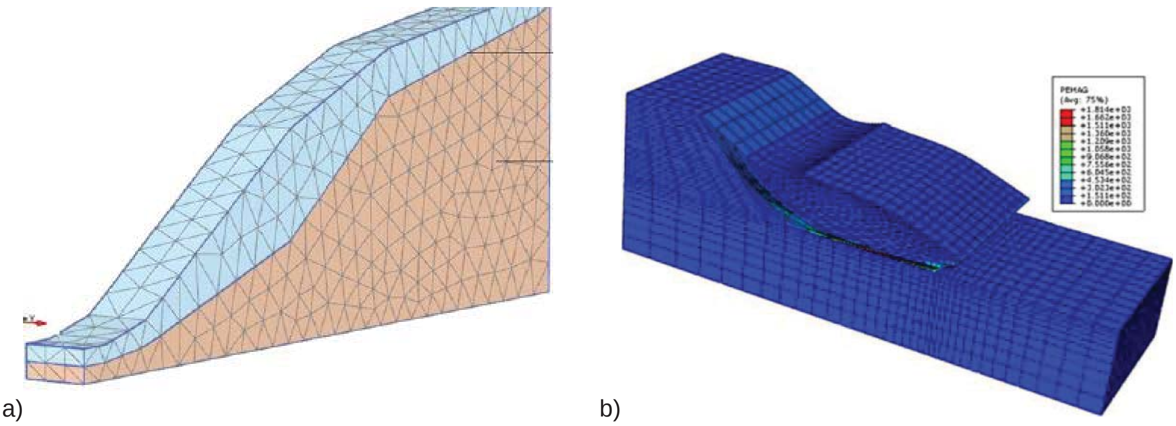


Slika 10. 2D i 3D numerički modeli kosine sa izdvojenim prikazom klizne mase tla i prostornim modelom klizne površi [23]

Figure 10. 2D and 3D numerical models of the slope with a separate representation of the sliding mass of the soil and a spatial model of the sliding surface [23]

Na slici 11 prikazani su 3D numerički modeli kosina – formirani od tetraedarskih i prizmatičnih konačnih elemenata, dok su na slici 12 prikazani 3D numerički modeli kosina formirani od prizmatičnih konačnih elemenata koji za osnovu imaju trougao, kvadrat i četvorougao s različitim unutrašnjim uglovima.

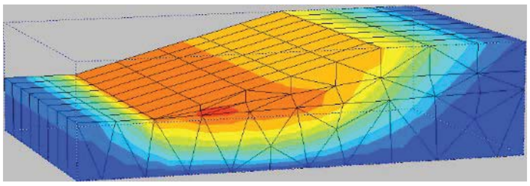
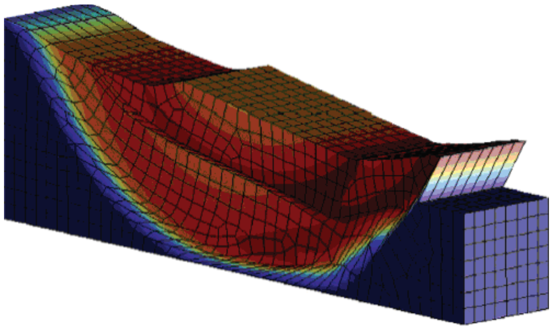
Figure 11 shows 3D numerical models of slopes formed from tetrahedral and solid finite elements, while Figure 12 shows 3D numerical models of slopes formed from solid finite elements, which have the base in the shape of a triangle, square and quadrangle with different inner corners.



a) b)

Slika 11. 3D numerički modeli kosina formirani od: a) tetraedarskih konačnih elemenata [33]; b) prizmatičnih konačnih elemenata [14]

Figure 11. 3D numerical models of slopes formed from: a) tetrahedral finite elements [33], b) solid finite elements [14]

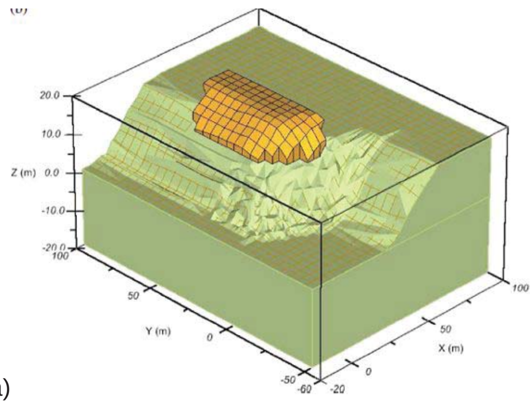
a)  b) 

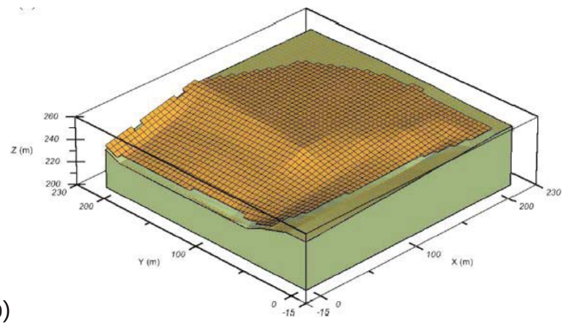
Slika 12. 3D numerički modeli kosina formirani od prizmatičnih konačnih elemenata koji za osnovu imaju: a) trougao [1]; b) kvadrat i četvorougao s različitim unutrašnjim uglovima [4]

Figure 12. 3D numerical models of slopes formed from solid finite elements that have the base in the shape of a: a) triangle [1], b) square and quadrandgle with different inner angles [4]

U određenim slučajevima, kada je domen tla znatnih dimenzija i kompleksnije geometrije, mreža konačnih elemenata 3D modela kosine može imati i nekoliko miliona konačnih elemenata, pa se u tim slučajevima najčešće primenjuje tehnika paralelnog procesiranja. Dodatno se kod ovakvih problema optimizuje mreža konačnih elemenata i numeracija čvorova elemenata, s obzirom na to što se optimizacijom numeracije čvorova konačnih elemenata redukuje širina trake matrice krutosti sistema i članovi matrice krutosti sistema grupišu oko dijagonale. Na slici 13 prikazani su 3D numerički modeli kosina nešto složenije geometrije sa izdvojenom kliznom masom tla. Modeliranje klizne površi – u analizi stabilnosti 3D modela kosina – može se sprovesti, kao što je već prezentovano, primenom 3D prostornih konačnih elemenata ili čak primenom 2D površinskih konačnih elementa.

In certain cases, when the ground domain is of considerable dimensions and a slightly complex geometry, the finite elements mesh of the 3D model of the slope can even have a several million finite elements, so in these cases, the most commonly used is parallel processing technique. With this type of problems, the mesh of finite elements and the numbering of the nodes of the elements are additionally optimized, since optimizing the numbering of finite element nodes reduces the bandwidth of the system stiffness matrix and concentrates the members of the system stiffness matrix around the diagonal. Figure 13 shows 3D numerical slopes models of a slightly complex geometry with the separate sliding mass of soil. Modelling the sliding surface, when analyzing the stability of 3D slopes models, can be carried out, as it has already been presented, by using 3D spatial finite elements or even 2D surface finite elements.

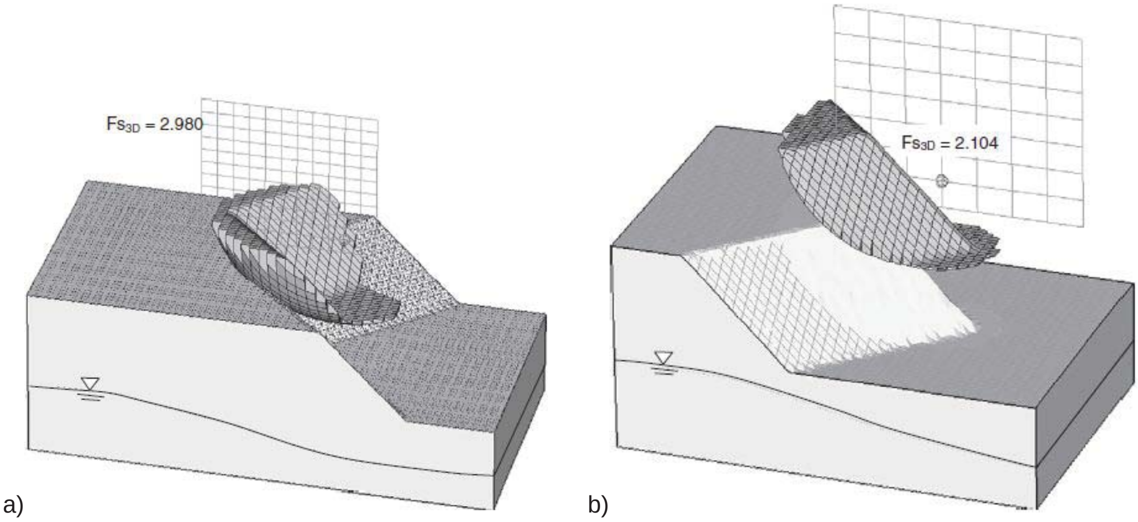
a) 

b) 

Slika 13. 3D numerički modeli kosina složenije geometrije s prikazanom izdvojenom kliznom masom tla [35]
Figure 13. 3D numerical models of slopes of a more complex geometry with the sliding mass of the soil separately shown [35]

Na slici 14 prikazani su 3D numerički modeli kosina nešto složenije geometrije, s prikazanom izdvojenom kliznom masom tla i položajima proračunatih tačaka faktora sigurnosti, dobijenih optimizacijom za konkavnu i konveksnu kliznu površ. Konkavna klizna površ formirana je iz 3D prostornih konačnih elemenata, dok je konveksna klizna površ formirana kombinacijom 3D prostornih i 2D površinskih konačnih elemenata.

Figure 14 shows the 3D numerical models of the slopes of a slightly complex geometry with separately shown sliding mass of the soil and the locations of the calculated points of the safety factors, obtained through optimization for the concave and convex sliding surface. The concave sliding surface is formed from 3D spatial finite elements, while the convex sliding surface is formed by combining 3D spatial and 2D surface finite elements.



a) b)

Slika 14. 3D numerički modeli kosina složenije geometrije s prikazanom izdvojenom kliznom masom tla i položajima proračunatih tačaka faktora sigurnosti, dobijenih optimizacijom: a) konkavna klizna površ; b) konveksna klizna površ [35]

Figure 14. 3D numerical models of slopes of a slightly complex geometry with separately shown sliding mass of the soil and the positions of the calculated safety factor points, obtained by optimization: a) concave sliding surface, b) convex sliding surface [35]

Modeliranje omekšanja i diskontinuiteta u tlu sprovodi se korekcijom parametara konstitutivnog modela ponašanja tla i eliminacijom veze određenih konačnih elemenata ili čak redukcijom određenog broja konačnih elemenata koji se nalaze u posebnoj zoni progušćenja mreže konačnih elemenata. Aproksimacija se odnosi na izbor optimalnog tipa konačnog elementa kojim se efikasno modelira polje pomeranja tla u modelu kosine. U ovom slučaju, postoji niz razvijenih tipova konačnih elemenata kod kojih se nepoznate određuju putem sila, pomeranja ili kombinovano (mešovito). Za interpolacione funkcije koristi se izoparametarska formulacija, pri čemu su čvorovi za proračun numeričkih integracija rapoređeni u uglovima, u unutrašnjosti i/ili po konturi konačnog elementa. Takođe, aspekt aproksimacije odnosi se na numeričko modeliranje konturnih i prelaznih uslova, modeliranje ponašanja materijala i modeliranje dejstava – opterećenja.

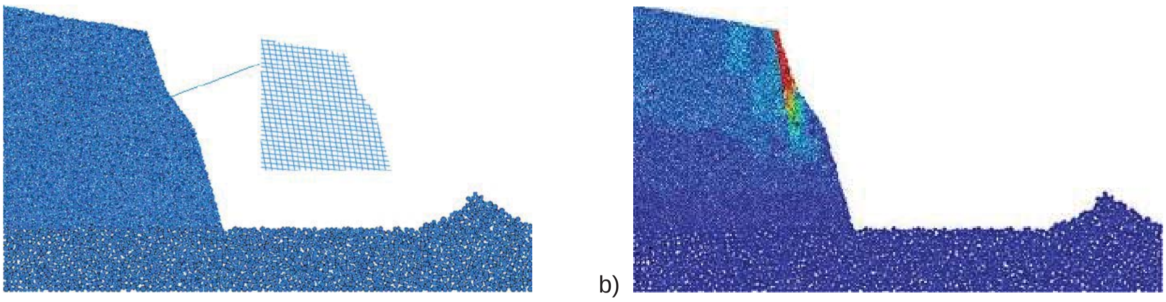
Proširena metoda konačnih elemenata (XFEM), za razliku od metode konačnih elemenata (FEM), ima mogućnost primene poboljšane nelinearne analize i proračuna postnelinearnog ponašanja sistema. Takođe, kod ove metode, prilikom formiranja klizišta, može se modelirati razvoj: prslina, pukotina i raseda u tlu. Prsline u tlu, u opštem slučaju, modeliraju se kao razmazane, dok se kod visokozahtevnih problema formiranja klizišta primenjuju algoritmi modeliranja diskretnih prslina. Model diskretnih prslina u tlu zahteva implementaciju algoritama mehanike kontakta, dok se model razmazanih prslina u tlu rešava nelinearnom analizom trajektorija ekstremnih vrednosti glavnih napona u tlu. Metoda graničnih elemenata (BEM) ima značajnu primenu u geotehnici, budući da se primenom ove metode brže dobijaju rešenja, u odnosu na metodu konačnih elemenata (FEM), pri čemu je i nivo kvaliteta konačnog rešenja zadovoljavajući. S obzirom na to što postoji nekoliko algoritama u okviru metode graničnih elemenata (BEM), oni se – u najvećem broju slučajeva –

Modelling of the softening and discontinuity in the soil is carried out by correcting the parameters of the constitutive model of soil behaviour and eliminating the connection of certain finite elements or even the reducing of a number of finite elements, which are located in a special zone of refined finite element mesh. The approximation refers to the choice of the optimal type of the finite element through which the field of soil displacement in the slope model is effectively modelled. In this case, there is a number of developed finite elements types in which unknowns are determined by: force, displacement or combined (mixed). For interpolation functions, an isoparametric formulation is used, while the nodes for the numerical integration calculation are mapped: in the angles, in the interior and/or on the contour of the final element. Also, the aspect of approximation refers to: numerical modelling of contour and transition conditions, modelling of material behaviour and modelling of effects - loads.

The *eXtended Finite Element Method* (XFEM), compared to the *Finite Element Method* (FEM), offers the possibility of applying an improved nonlinear analysis and the post-non-linear system behaviour calculation. Also, with this method, during the formation of the landslide, it is possible to modelled the development of: cracks, gaps and splits in the soil. In general, cracks in the soil are modelled as smeared, while with the highly demanding problems of landslide formation, the modelling algorithms for discrete cracks are applied. The model of discrete cracks in the ground requires the implementation of algorithms of contact mechanics, while the model of smeared cracks in the soil is solved by nonlinear analysis of the main stress in the soil for extreme values trajectory. The *Boundary Elements Method* (BEM) has a significant application in geo-technics, since the application of this method gives solutions faster than the *Finite Elements Method* (FEM), while the quality of the final solution is also satisfactory.

zasnivaju na diskretizaciji granične oblasti (kontura) graničnim elementima. Unutrašnjost oblasti najčešće se ne diskretizuje, pa ovakve metode pripadaju grupi bezmrežnih metoda. Metoda diskretnih elemenata (DEM) zasniva se na razmatranju ravnotežnog stanja pojedinačno za svaki konačni element. U poređenju s metodom konačnih elemenata (FEM), gde se ravnotežno stanje razmatra na globalnom nivou preko kompletnog numeričkog modela, kod metode diskretnih elemenata (DEM) jednačine kretanja definišu se posebno za svaki konačni element, tako da se mogu pratiti međusobno nezavisna polja pomeranja konačnih elemenata. Na slici 15 prikazan je 2D numerički model kosine prema metodi diskretnih elemenata (DEM) sa identifikovanom zonom iniciranja klizišta.

Since there are several algorithms within the *Boundary Elements Method* (BEM), they are mostly based on the discretization of the boundary area (contours) by the boundary elements. In most cases, the intrinsic domain is not discretized, so such methods belong to the group of mesh free methods. The *Discrete Element Method* (DEM) is based on the analysis of the equilibrium state for each finite element individually. In comparison to the *Finite Element Method* (FEM), where the equilibrium state is considered globally, through a complete numerical model, with the *Discrete Element Method* (DEM), the motion equations are defined for each finite element individually, so that the independent fields of finite elements movement can be traced. Figure 15 shows 2D numerical model of the slope according to the *Discrete Elements Method* (DEM) with identified landslide initiation zone.



a) b)

Slika 15. 2D numerički model kosine: a) numerički model kosine prema metodi diskretnih elemenata (DEM); b) identifikacija zone iniciranja klizišta prema metodi diskretnih elemenata (DEM)[22]

Figure 15. 2D numerical model of the slope: a) numerical model of the slope according to the Discrete Elements Method (DEM), b) identification of the landslide initiation zone according to the Discrete Elements Method (DEM) [22]

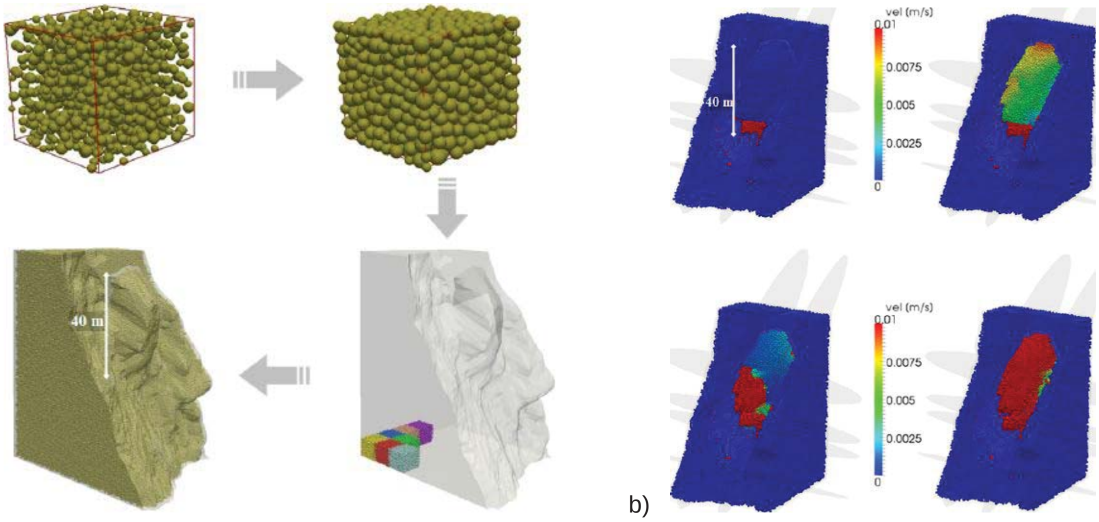
Primenom ove metode, može se pratiti inkrementalni razvoj klizišta, tako da se kao konačna vrednost proračuna dobija spektar faktora sigurnosti. Takođe, ova metoda primenjuje se i za 3D modeliranje složenih formi kosina, pri čemu je razvijen niz algoritama za topologiju i kompaktnost elementa kojima se formira 3D model kosine. Na slici 16 prikazan je postupak formiranja 3D numeričkog modela kosine prema metodi diskretnih elemenata (DEM) i odgovarajuće inkrementalne proračunske faze.

Da bi se ovakav algoritam efikasno primenio u praksi, međusobne veze konačnih elemenata modeliraju se kontaktnim elementima s mogućnošću implementacije različitih nelinearnih ponašanja. Kod kontaktnih elemenata, definišu se komponente krutosti pri pritisku, a naponi zatezanja se takođe mogu definisati ili čak eliminisati. Prilikom modeliranja kontakta dveju tačaka modela, javljaju se dva stanja: aktivno (kontakt je uspostavljen uz učešće određene krutosti) i neaktivno (kontakt nije uspostavljen uz učešće male krutosti ili bez uvođenja efekata krutosti). Da bi se efikasno modelirali efekti interakcije kontaktnih elemenata, potrebno je primeniti geometrijski nelinearnu inkrementalno-iterativnu analizu. Usled nelinearnog ponašanja kontaktnog elementa, gde promenu stanja može pratiti velika promena krutosti, mogu se javiti ozbiljne teškoće u obezbeđenju konvergencije nelinearnog rešenja. U tom smislu, može biti povoljnije koristiti proceduru kontrole inkrementalnog priraštaja pomeranja, nego proceduru

Through application of this method, the incremental development of the landslide can be traced, so that the spectrum of the safety factors is obtained as the final value of the calculation. Moreover, this method is also applied for 3D modelling of complex slope shapes, where a series of algorithms is developed for the topology and compactness of the elements which form the 3D slope model. Figure 16 shows the process of forming the 3D numerical slope model according to the *Discrete Elements Method* (DEM) and the corresponding incremental calculation phases.

For this algorithm to be effectively applied in practice, the connections between the finite elements are modelled by the contact elements with the possibility of implementing different nonlinear behaviours. The contact elements define the stiffness components under the pressure, and the tensile stresses can also be defined or even eliminated. When modelling the contact between two points of the model, two states occur: active (the contact is established with the involvement of certain stiffness) and inactive (the contact is not established with the involvement of little stiffness or without the introduction of stiffness effects). In order to efficiently model the effects of contact elements interaction, it is necessary to apply the geometric nonlinear incremental-iterative analysis. Due to the non-linear behaviour of the contact element, where the change of the state can be followed by a major change in stiffness, serious difficulties can arise in ensuring the convergence of the kontrole inkrementalnog priraštaja silama.

nonlinear solution. In that sense, it may be more beneficial to use the procedure for controlling the incremental increase of displacements, rather than the procedure for controlling the incremental increase of forces.



a) b)

Slika 16. 3D numerički model kosine: a) postupak formiranja 3D numeričkog modela kosine prema metodi diskretnih elemenata (DEM); b) inkrementalne proračunske faze [3]

Figure 16. 3D numerical slope model: a) the procedure of formation of the 3D numerical slope model according to the Discrete Elements Method (DEM), b) incremental calculation phases [3]

Uvođenje mehanike kontakta u analizu razvoja velikih plastičnih deformacija i kretanja mase tla klizišta sprovodi se i kod proširene metode konačnih elemenata (XFEM), slično kao i kod metode diskretnih elemenata (DEM). U samoj formulaciji problema smatra se da – pri inkrementalnim proračunskim fazama – nastupa takva promena geometrije zone kontakta, da inicijalnoj generisanoj mreži konačnih elemenata odgovara konfiguracija mreže konačnih elemenata za bilo koju inkrementalnu situaciju. Ovim se eliminiše upotreba dodatnih algoritama za pretraživanje povoljne konfiguracije u povezivanju čvorova mreže u *i*-toj inkrementalnoj analizi ili čak primena adaptivne metode za korekciju mreže konačnih elemenata sistema [34].

Numeričke inkrementalno-iterativne (nelinearne) analize stabilnosti klizišta zasnivaju se na formulaciji nelinearnog problema sistemom nelinearnih algebarskih jednačina oblika [2], [5]:

The introduction of the contact mechanics in the analysis of the development of large plastic deformations and the displacement of the landslide soil mass is also carried out with the *eXtended Finite Element Method* (XFEM), similar to the *Discrete Element Method* (DEM). In the formulation of the problem itself, it is considered that during incremental calculation phases occurs such a change in the geometry of the contact zone, that the initial generated mesh of finite elements is corresponding to the configuration of the mesh of finite elements for any incremental situation. This eliminates the use of additional algorithms for search for a favourable configuration in connecting the mesh nodes in *i*-th incremental analysis, or even the use of an adaptive method for correcting the mesh of finite elements of the system [34].

Numerical incremental-iterative (nonlinear) landslide stability analyses are based on the formulation of a non-linear problem through a system of non-linear algebraic equations of the form [2], [5]:

$$[K]\{u\}+\{F\}=0, \tag{42}$$

odnosno:

i.e.:

$$\{P\}+\{F\}=0, \tag{43}$$

gde su {*u*} nepoznati parametri pomeranja, {*F*} generalisani spoljašnji uticaji (opterećenja) u čvorovima sistema. Jednačine problema (42) umesto za ukupno opterećenje, rešavaju se za niz posebnih inkrementalnih opterećenja. U okviru svakog inkrementa, pretpostavlja se da je sistem jednačina linearan. Na taj način, rešenje

where {*u*} is the unknown displacement parameters, {*F*} generalized external effects (loads) in the system nodes. The equations of the problem (42) instead of for the total load, are solved for a series of specific incremental loads. Within each increment, it is assumed that the equation system is linear. In that way, the solution of a nelinearnog problema dobija se kao zbir niza linearnih (inkrementalnih) rešenja. Nelinearan problem može da se prikaže izrazom:

nonlinear problem is obtained as the sum of a series of linear (incremental) solutions. A non-linear problem can be represented by the expression:

$$[K_t]\{\Delta u\}+\lambda\{F\}=0\,, \tag{44}$$

odnosno:

i.e.:

$$\{P\}+\lambda\{F\}=0\,, \tag{45}$$

gde je $\{P\}$ vektor unutrašnjih generalisanih sila modela koje su funkcija vektora generalisanih pomeranja $\{u\}$, λ parametar inkrementalnog opterećenja (odnos inkrementalnog i kompletnog opterećenja). U skladu s konceptom inkrementalnog rešenja jeste:

where $\{P\}$ is the vector of the internal generalized model forces, which are the function of the generalized displacement vector $\{u\}$, $\{\lambda\}$ the incremental loading parameter (the ratio of incremental and total load). In accordance with the concept of incremental solution, we have:

$$\begin{aligned}\{\Delta u\}_i &= -[K_t]^{-1}\Delta\lambda_i\{F\}=-[K_t]_i^{-1}\{\Delta F\}_i\\ \{\Delta u\}_i &= \{u\}_{i+1}-\{u\}_i\\ \Delta\lambda_i &= \lambda_{i+1}-\lambda_i\\ \{\Delta F\}_i &= \{F\}_{i+1}-\{F\}_i=\Delta\lambda_i\{F\}\end{aligned}\;. \tag{46}$$

Iz izraza (46) određuju se inkrementi vektora pomeranja za inkremente opterećenja i tangentnu matricu krutosti modela klizišta, koja se formuliše za referentno stanje na početku inkrementa. Referentnom stanju na početku prvog inkrementa odgovara linearna matrica krutosti klizišta (inicijalna matrica krutosti). Opšti *i*-ti korak inkrementalnog postupka obuhvata: formiranje tangentne matrice krutosti $[K_t]_i$ numeričkog modela klizišta, određivanje inkremenata vektora opterećenja $\{\Delta F\}_i$ numeričkog modela, određivanje inkremenata vektora generalisanih pomeranja $\{\Delta u\}_i$ rešavanjem sistema linearnih algebarskih jednačina za tangentnu matricu krutosti, određivanje inkremenata uticaja u konačnim elementima (deformacije, naponi), i određivanje ukupne vrednosti generalisanih pomeranja inkrementalnim (kumulativnim) sabiranjem. Pomeranja posle *m*-tog inkrementa određena su izrazom:

From the expression (46), the increments of the displacement vector for loading increments of the load and the tangent stiffness matrix of the landslide model stiffness are determined, which is formulated for the reference state at the beginning of the increment. The reference state at the beginning of the first increment corresponds to the linear matrix of the landslide stiffness (initial stiffness matrix). The general *i*-th step of the incremental procedure includes: the formation of a tangent stiffness matrix $[K_t]_i$ of the numerical landslide model, determining the load vector increment $\{\Delta F\}_i$ of the numerical model, determining the vector of generalized displacements increments $\{\Delta u\}_i$ by solving the system of linear algebraic equations for the tangent stiffness matrix, determining the increments of the impact in the finite elements (deformations, tensions), and determining the total value of generalized displacements by incremental (cumulative) addition. Displacements after the *m*-th increment are defined by the expression:

$$\{u\}_m=\{u\}_0+\sum_{i=1}^{m}\{\Delta u\}_i\;. \tag{47}$$

Razlog za pojavu greške inkrementalnog rešenja jeste sprovedena linearizacija u okviru inkrementa. Veličina greške može da se odredi iz uslova ravnoteže na kraju inkrementa. Kao posledica linearizacije, javljaju se neuravnotežena (rezidualna) opterećenja koja su mera odstupanja inkrementalnog rešenja od tačnog. Vektor rezidualnog opterećenja može se prikazati kao odstupanje od ravnoteže:

The reason behind the occurrence of the incremental solution error is the linearization conducted within the framework of the increment. The error dimensions can be determined from the balance conditions at the end of the increment. As the linearization consequence, unbalanced (residual) loads occur, that are the measure of deviation of the incremental solution from the exact one. The residual load vector can be represented as a deviation from balance:

$$\{\Delta R\}_i=\{\Delta F\}_i-[K_t]_{i+1}\{\Delta u\}_i\,. \tag{48}$$

Korekcija greške postiže se dodavanjem rezidualnog opterećenja na spoljašnje opterećenje u sledećem inkrementu:

Error correction is achieved by adding the residual load to the external load in the following increment:

$$\{\Delta F\}_{i+1}^{R}=\{\Delta F\}_{i+1}+\{\Delta R\}_i\,. \tag{49}$$

Najbolji rezultati postižu se ako se kombinuje inkrementalni i iterativni postupak. U prvoj iteraciji, pojavljuju se rezidualna opterećenja zbog neispunjavanja uslova ravnoteže. Ako se naredne iteracije realizuju samo s rezidualnim opterećenjima, uz korekciju tangentne matrice krutosti, postupak može da konvergira uz minimiziranje rezidualnog opterećenja. Pri formulisanju iterativne metode, polazi se od izraza za razvoj u *Taylor*-ov red vektora rezidualnih sila u okolini pomeranja $\{u\}_j$:

The best results are achieved if the incremental and iterative processes are combined. In the first iteration, residual loads appear due to unfulfilled balance conditions. If the following iterations are realized only with residual loads, with the correction of the tangent stiffness matrix, the process can converge, with the minimization of the residual load. When formulating the iterative method, it is started with the expression for development in the *Taylor* series of the residual forces vector in the vicinity of the displacement $\{u\}_j$:

$$\{R\}_{j+1} = \{R\}_j + \frac{d\{R\}_j}{d\{u\}_j}\{\varDelta u\}_j . \tag{50}$$

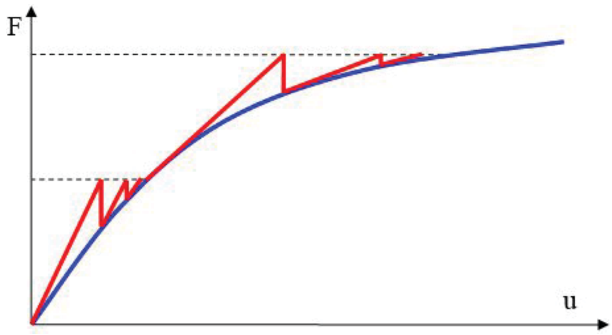
Iz uslova da rezidualno opterećenje ispunjava uslove ravnoteže $\{R\}_{j+1}=0$, važi:

From the condition that the residual load meets the balance conditions $\{R\}_{j+1}=0$, follows:

$$\{\varDelta u\}_j = -[K_t]^{-1}\{R\}_j . \tag{51}$$

Poslednja dva izraza predstavljaju osnovu iterativne metode. Kombinacijom inkrementalne i iterativne metode dobija se *Newton-Raphson*-ova inkrementalno-iterativna metoda (slika 17).

The last two expressions represent the basis of the iterative method. By combining the incremental and iterative methods, *Newton-Raphson*'s incremental-iterative method is obtained (Figure 17).



Slika 17.Newton-Raphson-ova inkrementalno-iterativna metoda [2], [5]
Figure 17. Newton-Raphson's incremental-iterative method [2], [5]

Numeričke inkrementalno-iterativne (nelinearne) analize stabilnosti klizišta – u kojima se primenjuje numerička integracija u vremenskom domenu – zasnivaju se na formulaciji nelinearnog problema kroz diferencijalne jednačine kretanja sistema s više stepeni slobode u matričnom obliku:

Numerical incremental-iterative (non-linear) landslide stability analyses, in which numerical integration in the time domain is applied, are based on the formulation of a nonlinear problem through the differential equations of the motion of the system with several degrees of freedom in the matrix form:

$$[M]\{a\}+[C]\{v\}+[K]\{d\}=\{Q\}. \tag{52}$$

S obzirom na to što se uzimaju u obzir potpuni razvoj i geometrijske i materijalne nelinearnosti, ovakva metoda u literaturi zove se i potpuna nelinearna dinamička analiza (NDA – *Nonlinear Dynamic Analysis*). Rešavanje jednačina (52) sprovodi se numeričkom integracijom korak po korak (*step by step*) *Hilber-Hughes-Taylor*-ovim (HHT) postupkom u modifikovanom obliku [13]:

Since the full development and geometric and material non-linearities are taken into account, this method is also referred to in the literature as the complete *Nonlinear Dynamic Analysis* (NDA). Solving the equations (52) is carried out through step-by-step numerical integration by *Hilber-Hughes-Taylor* (HHT) method in a modified form [13]:

$$[M]\{a\}_{i+1}+(1+\alpha)[C]\{v\}_{i+1}-\alpha[C]\{v\}_i+(1+\alpha)[K]\{d\}_{i+1}-\alpha[K]\{d\}_i=\{Q\}_{i+\alpha}, \tag{53}$$

a za trenutak vremena:

and for the moment of time:

$$t_{i+1}=t_i+\varDelta t , \tag{54}$$

gde je [*M*] matrica masa, {*a*} vektor ubrzanja, [*C*] matrica prigušenja, {*v*} vektor brzine, [*K*] matrica krutosti, {*d*} vektor pomeranja, {*Q*} vektor spoljašnjih generalisanih sila. Vektori pomeranja i brzine izražavaju se prema:

where [*M*] is the mass matrix, {*a*} acceleration vector, [*C*] damping matrix, {*v*} velocity vector, [*K*] stiffness matrix, {*d*} displacement vector, {*Q*} vector of external generalized forces. The displacement and velocity vectors are expressed according to:

$$\{d\}_{i+1} = \{d\}_i + \Delta t\{v\}_i + \frac{\Delta t^2}{2}\left[(1-2\beta)\{a\}_i + 2\beta\{a\}_{i+1}\right], \tag{55}$$

$$\{v\}_{i+1} = \{v\}_i + \Delta t\left[(1-\gamma)\{a\}_i + \gamma\{a\}_{i+1}\right], \tag{56}$$

dok za vektor spoljašnjih generalisanih sila važi:

while to the vector of external generalized forces applies:

$$\{Q\}_{i+\alpha} = \{Q\}(t_{i+\alpha}), \tag{57}$$

gde je:

where:

$$t_{i+\alpha} = (1+\alpha)t_{i+1} - \alpha t_i = t_{i+1} + \alpha\Delta t\,. \tag{58}$$

HHT postupak postaje bezuslovno stabilan ukoliko su parametri *α*, *β* i *γ* izabrani u skladu s relacijama:

The HHT method becomes unconditionally stable if the parameters *α*, *β* and *γ* are selected in accordance with the relations:

$$\alpha \in \left[-\frac{1}{3}, 0\right], \quad \beta = \frac{1}{4}(1-\alpha)^2, \quad \gamma = \frac{1}{2} - \alpha\,. \tag{59}$$

Vektori brzine $\{v\}_{i+1}$ i ubrzanja $\{a\}_{i+1}$ u trenutku t_{i+1} izražavaju se preko vektora pomeranja na kraju intervala $\{d\}_{i+1}$:

The velocity vector $\{v\}_{i+1}$ and the acceleration vector $\{a\}_{i+1}$ at the moment t_{i+1} are expressed by the displacement vector at the end of the interval $\{d\}_{i+1}$:

$$\{v\}_{i+1} = \frac{\gamma}{\beta\Delta t}\left(\{d\}_{i+1} - \{d\}_i\right) - \left(\frac{\gamma}{\beta} - 1\right)\{v\}_i - \Delta t\left(\frac{\gamma}{2\beta} - 1\right)\{a\}_i, \tag{60}$$

$$\{a\}_{i+1} = \frac{\gamma}{\beta\Delta t^2}\left(\{d\}_{i+1} - \{d\}_i\right) - \frac{1}{\beta\Delta t}\{v\}_i - \left(\frac{1}{2\beta} - 1\right)\{a\}_i\,. \tag{61}$$

Unošenjem ovih izraza u jednačinu (53), dobija se ekvivalentna jednačina ravnoteže:

Including these expressions into the equation (53) gives the equivalent equation of equilibrium:

$$[K]^*\{d\}_{i+1} = \{Q\}^*_{i+\alpha}, \tag{62}$$

gde je:

where:

$$[K]^* = (1+\alpha)[K] + \frac{1}{\beta\Delta t^2}[M] + (1+\alpha)\frac{\gamma}{\beta\Delta t}[C], \tag{63}$$

$$\begin{aligned}\{Q\}^*_{i+\alpha} = {} & \{Q\}_{i+\alpha} + [M]\left[\frac{1}{\beta\Delta t^2}\{d\}_i + \frac{1}{\beta\Delta t}\{v\}_i + \left(\frac{1}{2\beta} - 1\right)\{a\}_i\right] + \\ & + [C]\left\{(1+\alpha)\frac{\gamma}{\beta\Delta t}\{d\}_i + \left[(1+\alpha)\frac{\gamma}{\beta} - 1\right]\{v\}_i + \Delta t(1+\alpha)\left(\frac{\gamma}{2\beta} - 1\right)\{a\}_i\right\} + \alpha[K]\{d\}_i\,.\end{aligned} \tag{64}$$

Ukoliko se vrednosti parametara *α*, *β* i *γ* usvoje da su:

If the following values are accepted for parameters *α*, *β* and *γ*:

$$\alpha = -\frac{1}{3}, \quad \beta = \frac{4}{9}, \quad \gamma = \frac{5}{6}, \tag{65}$$

tada su efektivna matrica krutosti i vektor efektivnog opterećenja:

then the effective stiffness matrix and the effective load vector are:

$$[K]^{*}=\frac{2}{3}[K]+\frac{9}{4\Delta t^{2}}[M]+\frac{5}{4\Delta t}[C], \tag{66}$$

$$\{Q\}_{i+\alpha}^{*}=\{Q\}_{i+\alpha}+[M]\left(\frac{9}{4\Delta t^{2}}\{d\}_{i}+\frac{9}{4\Delta t}\{v\}_{i}+\frac{1}{8}\{a\}_{i}\right)+$$
$$+[C]\left(\frac{5}{4\Delta t}\{d\}_{i}+\frac{1}{4}\{v\}_{i}-\frac{1}{24}\Delta t\{a\}_{i}\right)-\frac{1}{3}[K]\{d\}_{i}, \tag{67}$$

gde je:

where:

$$t_{i+\alpha}=t_{i+1}-\frac{1}{3}\Delta t=t_{i}+\frac{2}{3}\Delta t, \tag{68}$$

odnosno:

i.e.:

$$\{Q\}_{i+\alpha}=\{Q\}\left(t_{i}+\frac{2}{3}\Delta t\right). \tag{69}$$

Sa određenim pomeranjima na kraju posmatranog intervala, rešavanjem jednačina (62), brzine i ubrzanja na kraju intervala dobijaju se prema izrazima:

With certain shifts at the end of the observed time interval by solving the equations (62), velocity and acceleration at the end of the time interval are obtained according to the following expressions:

$$\{v\}_{i+1}=\frac{15}{8\Delta t}\left(\{d\}_{i+1}-\{d\}_{i}\right)-\frac{7}{8}\{v\}_{i}+\frac{1}{16}\Delta t\{a\}_{i}, \tag{70}$$

$$\{a\}_{i+1}=\frac{9}{4\Delta t^{2}}\left(\{d\}_{i+1}-\{d\}_{i}\right)-\frac{9}{4\Delta t}\{v\}_{i}-\frac{1}{8}\{a\}_{i}. \tag{71}$$

Pre započinjanja algoritma korak po korak, potrebno je da se početno ubrzanje sistema odredi iz diferencijalne jednačine kretanja prema:

Before starting the step-by-step algorithm, it is necessary that the initial acceleration of the system is determined from the differential equation of motion according to:

$$\{a\}_{0}=[M]^{-1}\left(\{Q\}_{0}-[C]\{v\}_{0}-[K]\{d\}_{0}\right). \tag{72}$$

Korekcija matrice krutosti sistema sprovodi se posle svakog apliciranog koraka vremena, a prema prethodno prezentovanoj *Newton-Raphson*-ovoj metodi. Primenom NDA analize sa HHT postupkom i NR metodom za proračun 2D i 3D modela klizišta, dobijaju se najpouzdanija rešenja za procenu nelinearnog odgovora sistema. Primenom ovakve metode, moguće je razmatrati uticaj dinamičnosti povećanja nivoa podzemne i površinske vode, a takođe i dejstvo zemljotresa inkrementalno skalirajući akcelerogram. Odgovor sistema (klizišta) predstavlja se kao funkcija promene faktora sigurnosti F_s u vremenu, a ne samo kao jedinstvena (diskretna) vrednost.

The correction of the system stiffness matrix is carried out after each applied time step, and according to the previously presented *Newton-Raphson*'s method. Using the NDA analysis with the HHT method and the NR method for calculating the 2D and 3D landslide models, the most reliable solutions for estimating the nonlinear system response are obtained. Using this method allows us to consider the influence of the level of underground and surface water increase dynamics, as well as the effect of the earthquake, incrementally scaling the accelerometer. System (landslide) response is represented as the function of change of the safety factor F_s in time, and not only as a unique (discrete) value.

3.4 Kompleksno 3D geometrijsko modeliranje i numeričke metode proračuna stabilnosti klizišta

Standardni pristup u modeliranju terena i klizišta – inkorporiranog u terenu – zasniva se na korišćenju tehnike 2D prezentacije primenom situacionog plana i vertikalnih poprečnih preseka. Na osnovu definisanih tipova slojeva tla po dubini i njihovih fizičko-mehaničkih

3.4 Complex 3D geometric modelling and numerical methods for landslides stability calculations

The standard approach to modelling of the terrain and landslide, incorporated in the terrain, is based on the usage of the 2D presentation technique by applying a situational plan and vertical cross sections. Based on the defined types of soil layers according to depth and their karakteristika, sprovodi se analitički i/ili numerički proračun stabilnosti kosina. U slučaju prostorno složenijeg modela terena i kompleksnije geometrije klizišta, pitanje 2D modeliranja i pouzdanosti odgovarajućih analiza može biti diskutabilno. Međutim, i u situacijama kada se pouzdano može tvrditi da je tehnika 2D prezentacije terena i klizišta, primenom situacionog plana i vertikalnih poprečnih preseka pouzdana, ostaju otvorena neka pitanja – da li se može dodatno poboljšati prezentacija terena i klizišta u skladu sa savremenim informacionim tehnologijama i da li se može pouzdano odrediti zapremina tla koja formira klizište. Odgovori na ova pitanja mogu se pronaći u 3D vizuelizaciji terena i klizišta, pri čemu se kao najsofisticiranije rešenje, primenom 4D vizuelizacije (3D + dinamičke simulacije) može predstaviti problem sanacije klizišta, od inicijalnog stanja, preko faznih rešenja, pa sve do finalnog rešenja. 3D modeliranje terena i klizišta koristi se za geometrijsku prezentaciju i numeričku analizu primenom površi i *solida*. Geometrijska 3D prezentacija, u najvećem broju slučajeva, ima veći stepen vizuelizacije konačnog rešenja, dok je cilj numeričke 3D analize da se primenom površi i *solida* modelira teren i klizište, tako da svaki geometrijsko-numerički element ima u sebi integrisanu i matematičku formulaciju problema. Podrazumeva se da se i prilikom numeričkog modeliranja terena i klizišta može dodatno postići realističan efekat geometrijske prezentacije, međutim u ovakvim situacijama dodatno se povećava vreme proračuna, tako da se – u veoma složenim modelima i s veoma velikim brojem konačnih elemenata – proračun svodi na primenu tehnike paralelnog procesiranja. Međutim, geometrijsko 3D modeliranje za prezentaciju terena i klizišta dosta je korisnije za proračune zapremine tla, s obzirom na to što se modeliranjem klizišta kao *solida* može veoma brzo odrediti odgovarajuća zapremina, čak i u situacijama veoma složenih *solid* modela. Postupak kompleksnog 3D modeliranja terena i klizišta zasniva se na prethodnoj identifikaciji većeg broja kliznih ravni za odgovarajući broj inženjersko-geoloških profila, njihovom integracijom sa 2D situacionim planom klizišta i konstrukcijom 3D modela klizišta u softveru za geometrijsku prezentaciju (CAD). Za integrisane klizne ravni formira se klizna površ u prostoru, dok se za modelirano klizište u prostoru formira *solid* model klizišta. Modeliranje klizne površi u prostoru zasniva se na primeni kompleksne zakrivljene površi koja formira mrežu četvorouglova, dok se *solid* model klizišta generiše primenom primitiva i tehnika za editovanje primitiva: ekstrudiranje, sečenje, proširenje, ujedinjenje, ekstrakcija, intersekcija i slično. Na slici 18 prikazani su generisani kompleksni geometrijsko-numerički 3D modeli terena za analizu stabilnosti klizišta.

Generalno razmatrajući modeliranje površi u prostoru može se sprovesti primenom matematičkih funkcija, mapiranja i diskretnih vrednosti. Najviše se koristi tehnika mapiranja terena s rasterskom mrežom (ortogonalna, poluortogonalna, radijalna i zakrivljena) za formiranje mape terena, ali je primena diskretnih vrednosti i formiranje polilinija, površi i *solida* u prednosti, pa se za ovakvu grafiku koristi termin vektorska grafika. Izohipse terena i klizne površi, u opštem slučaju, predstavljaju se primenom polilinija i splajnova. Da bi se geometrijski i matematički modelirao skup tačaka koji formira jednu kliznu površ u 2D koordinatnom sistemu, potrebno je

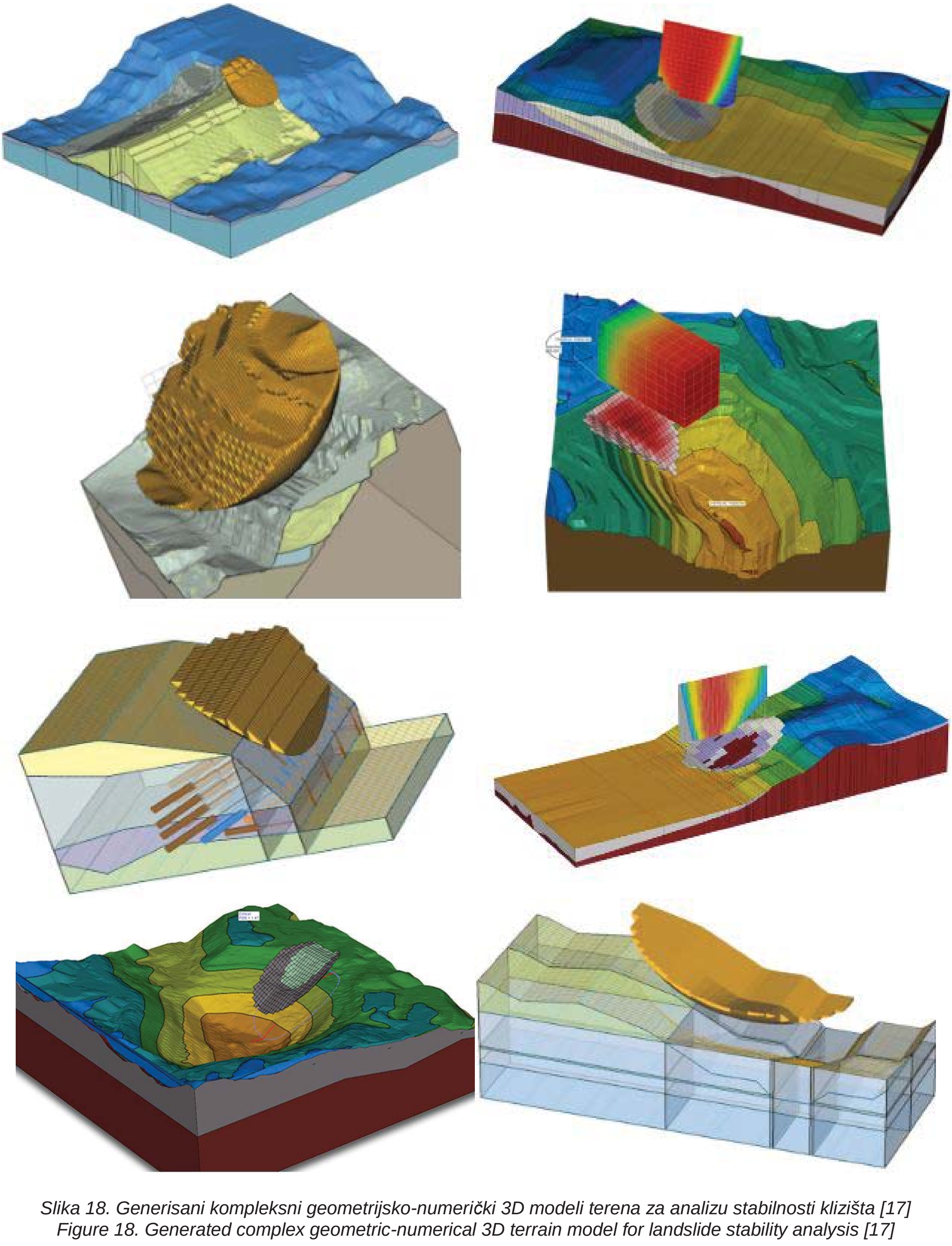
physico-mechanical characteristics, an analytical and/or numerical calculation of the slope stability is carried out. In the case of a spatially slightly complex terrain model and slightly complex landslide geometry, the question of 2D modelling and the reliability of the corresponding analyses can be debatable. However, even in situations where it can be reliably asserted that the 2D presentation of the terrain and landslide by using the situational plan and vertical cross-sections is reliable, the following questions remain open: can the presentation of the terrain and the landslide be further improved in accordance with modern information technology and whether the volume of the soil forming the landslide can be reliably determined? The solution to these issues can be found in 3D visualization of terrain and landslide, whereby the most sophisticated solution, by using 4D visualization (3D + dynamic simulation), can present the problem of landslide sanation, from the initial state, through phase solutions to the final solution. 3D modelling of the terrain and landslide is used for geometric presentation and numerical analysis through using surfaces and solids. Geometric 3D presentation, in most cases, has a greater degree of visualization of the final solution, while numerical 3D analysis aims to use the surfaces and solids to model the terrain and landslide, so that each geometric-numerical element also has in itself an integrated mathematical formulation of the problem. It is presumed that the realistic effect of the geometric presentation can be additionally achieved in numerical modelling of the terrain and landslide, however, in these situations the time of the calculation is further increased, so that, in very complex models and with a very large number of finite elements, the calculation is reduced to the application of the parallel processing technique. However, geometric 3D modelling for the presentation of terrain and landslides is much more useful for soil volume calculations, since modelling the landslide as a solid can quickly determine the appropriate volume, even in situations of very complex solid models. The process of complex 3D modelling the terrain and landslide is based on: the previous identification of a larger number of sliding planes for the corresponding number of engineering-geological profiles, integration of these with the 2D situational plan of the landslide and the construction of the landslide 3D model in the geometric presentation software (CAD). For the integrated sliding planes, a sliding surface is formed in space, while for the modelled landslide in space a solid landslide model is formed. The modelling of the sliding surfaces in space is based on the application of complex curved surface that forms a grid of quadrangles, while the solid model of the landslide is generated using primitives and techniques for editing primitives: extrusion, cutting, expanding, unifying, extraction, intersection, and the like. Figure 18 shows the generated complex geometric-numerical 3D terrain models for landslide stability analysis.

In general, modelling the surface in space can be conducted by using mathematical functions, mapping, and discrete values. The technique most widely used is terrain mapping with a raster mesh (orthogonal, semi-orthogonal, radial and curved) to form a map of the terrain, but the application of discrete values and the formation of polylines, surfaces and solids has more benefits, so the term used for such a graphic is vector sprovesti interpolaciju. Jednostavniji modeli interpolacija zasnivaju se na primeni matematičkih funkcija u zatvorenom obliku. Međutim, interpolacija većeg broja tačaka – koje formiraju jednu kliznu površ u 2D koordinatnom sistemu – zasniva se na primeni parametarskih funkcija, gde rešenje nije definisano u zatvorenom obliku, već u skupu funkcija. Povezanost ovih funkcija uspostavlja se uslovima ekvivalencije tangente za krive s leve i desne strane u svakoj tački interpolacije. Na taj način, dobija se glatka interpolirana kriva, pa se među najboljim parametarskim funkcijama pokazala primena *splajna*.

U slučaju 3D modela terena i klizne ravni, tačnije klizne površi, oni se u prostoru modeliraju primenom NURBS krivih (*non-uniform rational basis spline*). NURBS krive definisane su kontrolnim čvorovima i vektorom čvora. U opštem slučaju, NURBS krive i i odgovarajuće površi jesu generalizacija B-*splajnova* i *Bezier*-ovih krivih i površi. Kontrolni čvorovi definišu oblik površi, u konkretnom slučaju klizne površi, dok vektor čvora određuje gde i kako površ dodiruje kontrolne čvorove. Međutim, i prilikom primene NURBS površi može se pojaviti problem u interpolaciji, ukoliko se za određene kontrolne čvorove – koji su diskretne vrednosti skupa kliznih površi – adekvatno ne izaberu parametri interpolacije. Mogu se dobiti isuviše velika odstupanja u interpolaciji, tako da 3D model terena i klizišta može biti aproksimiran slično kao što se primenjuje princip u regresionim analizama, bilo da su one linearnog ili nelinearnog tipa. Minimiziranje prethodnog problema postiže se progušćenjem mreže konačnih elemenata, uvođenjem novih međuelemenata. U opštem slučaju najpouzdanija, ali i isto tako i vizuelno grublja rešenja postižu se primenom četvorouglova čiji čvorovi direktno povezuju diskretne čvorove (linearna interpolacija) terena i klizišta. Rafiniranost mreže postiže se interpolacijom trouglovima. Kao što je već prethodno napisano, prezentacija terena sprovodi se, zapravo, primenom žičanog (*wireframe*) modela površi sa dodavanjem 3D površi, dok se modeliranje klizišta sprovodi primenom *solida* (3D geometrijsko telo). Diferencijacija klizišta u odnosu na ostale delove terena može se sprovesti izdvajanjem i prikazom samo klizišta, nezavisno od terena, s mogućnošću 4D kontinualne translacije i rotacije u prostoru, i renderovanjem, tako da se terenu poveća transparentnost, u odnosu na klizište.

graphics. The terrain isohypse and sliding surfaces, in general, are represented using polylines and splines. In order to geometrically and mathematically model the set of points that forms a single sliding surface in the 2D coordinate system, interpolation is required. Those simpler interpolation models are based on the application of mathematical functions in closed form. However, interpolation of a large number of points, that form a single sliding surface in 2D coordinate system, is based on the application of parametric functions, where the solution is not defined in a closed form, but in a set of functions. The connection of these functions is established by the conditions of the tangent equivalence for curves on the left and right at each point of interpolation. This way, a smooth interpolated curve is obtained, so the application of the spline has turned out to be among the best parametric functions.

In the case of 3D terrain model and sliding plane, more precisely the sliding surface, they are modelled in the space using NURBS curves (*Non-Uniform Rational Basis Spline*). NURBS curves are defined by the control nodes and the node vector. In general, NURBS curves and the corresponding surfaces are the generalization of B-splines and *Bezier*'s curves and surfaces. The control nodes define the shape of the surface, in particular, the sliding surface, while the node vector determines where and how the surface touches the control nodes. However, even with the application of NURBS surfaces, a problem may arise in interpolation, if the adequate interpolation parameters are not selected for certain control nodes, and which are discrete values of a set of sliding planes. Excessive interpolation deviations can occur so that the 3D terrain and landslide model can be approximated in a similar manner as the principle in regression analysis applies, whether they are linear or nonlinear. Minimizing the previous problem is achieved by increase in the density of the mesh of finite elements through the introduction of new inter elements. In general, the most reliable, but also visually rougher solutions are achieved by applying quadrangles whose nodes directly connect discrete nodes (linear interpolation) of the terrain and landslide. The mesh refinement is achieved by interpolation by triangles. As previously mentioned, the presentation of the terrain is carried out, in fact, by using a wireframe plane model with the addition of 3D planes, while the landslide modelling is carried out by using a solid (3D geometric body). The differentiation of the landslide in relation to other parts of the terrain can be carried out by allocation and display of landslide only, irrespective of the terrain, with the possibility of 4D continuous translation and rotation in space, and rendering, so that the terrain transparency is increased in relation to the landslide.



Slika 18. Generisani kompleksni geometrijsko-numerički 3D modeli terena za analizu stabilnosti klizišta [17]
Figure 18. Generated complex geometric-numerical 3D terrain model for landslide stability analysis [17]

4 ZAVRŠNE NAPOMENE

Primenom sprovedene sistematizacije analitičkih i numeričkih metoda proračuna stabilnosti klizišta, može se efikasno razmotriti koji tip metode se može primeniti u fazama preliminarnih i finalnih analiza za naučna istraživanja i stručne projekte. Autori su napravili sopstvenu sistematizaciju metoda proračuna stabilnosti klizišta, s tim što pojedine metode mogu pripadati i prelaznim kategorijama. Posebno je to slučaj kod onih metoda koje se zasnivaju na direktnoj analizi stabilnosti za odgovarajuću kliznu površ i kod metoda koje koriste iteracije kliznih površi primenom optimizacionih algoritama.

Ključni problemi u modeliranju i numeričkoj analizi klizišta današnjice mogli bi se prikazati iz nekoliko aspekata:

– generalizacija nedovoljnog broja uzorkovanja i dobijanja odgovarajućih kvalitetnih laboratorijskih ispitivanja fizičko-mehaničkih karakteristika tla i konstitutivnih modela ponašanja tla za kompletno klizište;

– primena geometrijsko-numeričke prezentacije klizišta putem 3D modela (u određenim situacijama, mogu se dobiti i viši faktori sigurnosti usled zaklinjavanja klizišta pri klizanju tla);

– potreba da se dodatno unapredi metodologija verifikacije stabilnosti klizišta na osnovu matematičkih modela i analiza inkrementalnog pomeranja klizišta, monitoringom deformacija, a ne sila i momenata;

– implementiranje tehnike paralelnog procesiranja u praktične svrhe (povećanje hardverskih kapaciteta – višejezgarnim procesiranjem i resursa – skladištenjem memorije).

4 FINAL REMARKS

By applying the conducted systematization of analytical and numerical methods of landslide stability calculation it can effectively be considered which type of method can be applied in the phases of preliminary and final analyzes for scientific research and professional projects. The authors have made their own systematization of the methods of landslide stability calculation, but some methods can also belong to transition categories. This is especially the case with those methods that are based on a direct stability analysis for the corresponding sliding surface and for methods using sliding surface iterations by applying optimization algorithms.

Key problems in modelling and numerical analysis of nowadays landslides could be presented through several aspects:

– generalization of insufficient number of sampling and obtaining appropriate quality laboratory tests of physical-mechanical characteristics of soil and constitutive models of soil behaviour for a complete landslide,

– the application of the geometric-numerical presentation of the landslide through 3D models (in certain situations, higher safety factors can be obtained due to the wedging of the landslide during the soil sliding),

– it is necessary to further improve the methodology of landslide stability verification based on mathematical models and analysis of incremental displacement of the landslide, by monitoring the deformations, but not the forces and moments,

– implementing parallel processing techniques for practical purposes (increasing: hardware capacities through multi-core processing and resources through storage of the memory).

5 LITERATURA REFERENCE

[1] Albataineh N.: Slope Stability Analysis Using 2D and 3D Methods, University of Akron, 2016.

[2] Bathe K.: Finite Element Procedures, Prentice Hall, 1037p, Upper Saddle River, USA, 1996.

[3] Bonilla Sierra V.: De la Photogrammetrie a la Modelisation 3D: Evaluation Quantitative du Risque d'Eboulement Rocheux, Universite Grenoble Alpes, Docteur de l'Universite Grenoble Alpes, 2006.

[4] Chen X., Wub Y., Yu Y., Liu J., Frank X, Ren J.: A Two-Grid Search Scheme for Large-Scale 3-D Finite Element Analyses of Slope Stability, Computers and Geotechnics, Vol. 62, 2014, pp. 203-2015.

[5] Crisfield M.: Non-Linear Finite Element Analysis of Solids and Structures, Volume 2: Essentials, John Wiley & Sons, 345p, New York, USA, 2000.

[6] Dai F., Lee C., Ngai Y.: Landslide Risk Assessment and Management: An Overview, Engineering Geology, Vol. 64, No. 1, 2002, pp. 65–87.

[7] EN 1997-1:2004, Eurocode 7: Geotechnical Design – Part 1: General Rules, Brussels, Belgium, 2004.

[8] Fellipa C.: Advanced Finite Element Methods, University of Colorado, Boulder, 2007.

[9] Fredlund D.: Analytical Methods for Slope Stability Analysis, State of the Art, The 4th International Symposium on Landslides, Toronto, Canada, 1984, pp. 229-250.

[10] GEO 5, User's Guide, Fine Ltd., 2016.

[11] Geološka terminologija i nomenklatura VIII-2, Inženjerska geologija, Zavod za regionalnu geologiju i paleontologiju Rudarsko-geološkog fakulteta, Univerzitet u Beogradu, Beograd, Srbija, 1978.

[12] Gustafsson J., Lindstrom M.: Applicability of Optimised Slip Surfaces: Evaluation of a Software's Optimisation Function for Generating Composite Slip Surfaces, Applied on Stability Analysis of Clay Slopes, Chalmers University of Technology, Gothenburg, Sweden, 2014.

[13] Hilber H., Hughes T., Taylor R.: Improved Numerical Dissipation for Time Integration Algorithms in Structural Dynamics, Earthquake Engineering and Structural Dynamics, Vol. 5, No. 3, pp. 283-292, 1977.

[14] Ho I-H.: Parametric Studies of Slope Stability Analyses Using Three-Dimensional Finite Element Technique: Geometric Effect, Journal of Geoengineering, Vol. 9, No. 1, 2014, pp. 33-43.
[15] http://geoliss.mre.gov.rs/beware/form/guest_page.php
[16] http://landslides.usgs.gov/learn/majorls.php
[17] https://www.soilvision.com/products/svoffice5/svslope
[18] https://www.studyblue.com/notes/note/n/earth-science-final-exam/deck/4839996
[19] Janbu N.: Slope Stability Computations in Embankment Dam Engineering, R. Hirschfeld and S. Poulos, eds., John Wiley and Sons, New York, USA, 1973, pp. 47-86.
[20] Kainthola A., Verma D., Thareja R., Singh T.: A Review on Numerical Slope Stability Analysis, International Journal of Science, Engineering and Technology Research (IJSETR), Vol. 2, No. 6, 2013, pp. 1315-1320.
[21] Kaur A., Sharma R.: Slope Stability Analysis Techniques: A Review, International Journal of Engineering Applied Sciences and Technology, Vol. 1, No. 4, 2016, pp. 52-57.
[22] Kong Y., Chen P., Yu H.: Analysis of Rock High-Slope Stability Based on a Particle Flow Code Strength Reduction Method, Electronic Journal of Geotechnical Engineering, Vol. 20, 2015, pp. 13421-13430.
[23] Leong E., Rahardjo H.: Two and Three-Dimensional Slope Stability Reanalyses of Bukit Batok Slope, Computers and Geotechnics, Vol. 42, pp. 81-88, 2012.
[24] Maksimović M.: Mehanika tla, Čigoja štampa, Beograd, Srbija, 2001.
[25] Memić M., Folć R., Ibrahimović A.: Numerical Modeling and Slope Reparation Methods in an Altered and Unstable Serpentine Rock Mass, Building Materials and Structures, Vol. 55, No. 4, 2012, pp. 23-45.
[26] Morgenstern N., Price V.: The Analysis of the Stability of General Slip Surfaces. Géotechnique, Vol. 15, No. 1, 1965, pp.79-93.
[27] Pereira T, Robaina A., Peiter M., Braga F., Rosso R.: Performance of Analysis Methods of Slope Stability for Different Geotechnical Classes Soil on Earth Dams, Journal of the Brazilian Association of Agricultural Engineering, Vol. 36, No. 6, 2016, pp.1027-1036.
[28] Sarma S.: Stability Analysis of Embankments and Slopes, Géotechnique, Vol. 23, No. 3, 1973, pp. 423-433.
[29] Schuster R.: The 25 Most Catastrophic Landslides of the 20th Century, in Chacon, Irigaray and Fernandez (eds.), Landslides, Proc. Of the 8th International Conf. & Field Trip on Landslides, Granada, Spain, Rotterdam: Balkema, 1996.
[30] Spencer E.: A Method of Analysis of the Stability of Embankments Assuming Parallel Inter-Slice Forces, Géotechnique, Vol. 17, No. 1, 1967, pp. 11-26.
[31] Шахунянц Г.:Железнодорожный путь: учеб. для вузов ж.-д. трансп. /– 3-е изд., перераб. и доп. – М. : Транспорт, 1987.
[32] Tschuchnigg F., Schweiger H., Sloan S.: Slope Stability Analysis by Means of Finite Element Limit Analysis and Finite Element Strength Reduction Techniques, Part II: Back Analyses of a Case History, Computers and Geotechnics, Vol. 70 , 2015, pp. 178–189.
[33] Usluogullari O., Temugan A., Duman E.: Comparison of Slope Stabilization Methods by Threedimensional Finite Element Analysis, Natural Hazards, Vol. 81, No. 2, 2016, pp. 1027-1050.
[34] Wriggers P.: Computational Contact Mechanics, Springer-Verlag, New York, USA, 2006.
[35] Zhang L., Fredlund M., Fredlund D., Lub H., Wilson G.: The Influence of the Unsaturated Soil Zone on 2-D and 3-D Slope Stability Analyses, Engineering Geology, Vol. 193, 2015, pp. 374–383.
[36] Zhu D., Lee C., Qian Q., Chen G.: A Concise Algorithm for Computing the Factor of Safety Using the Morgenstern-Price Method, Canadian Geotechnical Journal, Vol. 42, No. 1, 2005, 272-278.

REZIME

SISTEMATIZACIJA ANALITIČKIH I NUMERIČKIH METODA PRORAČUNA STABILNOSTI KLIZIŠTA

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Na osnovu analize mnogih naučnih radova, autori su dali prikaz sopstvene originalne sistematizacije analitičkih i numeričkih metoda proračuna stabilnosti klizišta, pri čemu mnoge od njih tek treba dodatno da se unaprede, implementiraju i testiraju na kompleksnim 3D modelima klizišta. Metode proračuna stabilnosti klizišta klasifikovane su u pet grupa: analitičke jednokoračne, analitičke višekoračne (iteracije kliznih površi), numeričke višekoračne (iteracije kliznih površi), numeričke inkrementalno-iterativne (nelinearne) analize i numeričke inkrementalno-iterativne (nelinearne) analize, uz primenu numeričke integracije u vremenskom domenu. Primenom sprovedene sistematizacije metoda proračuna stabilnosti klizišta, može se vrlo efikasno razmotriti koji je tip metode optimalan za analizu klizišta i koji tip metode je potrebno koristiti u fazi preliminarnih i finalnih analiza za naučna istraživanja i stručne projekte.

Ključne reči: klizište, sistematizacija, analitičke metode, numeričke metode, 2D i 3D modeliranje

SUMMARY

THE SYSTEMATIZATION OF ANALYTICAL AND NUMERICAL METHODS OF LANDSLIDE STABILITY CALCULATION

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According to the analysis of a large number of scientific papers, the authors of the paper presented their own original systematization of the analytical and numerical methods of landslide stability calculation, with a large part of them still to be further improved, implemented and tested on complex 3D landslide models. Methods for calculating the stability of the landslide are classified into five groups: analytical single-step, analytical multi-step (iterations of sliding surfaces), numerical multi-step (iterations of sliding surfaces), numerical incremental-iterative (nonlinear) analysis and numerical incremental-iterative (nonlinear) analysis, applying numerical integration in the time domain. By using the systematization method of calculating the stability of the landslide it can be very effective to consider which type of method is optimal for landslide analysis and which type of method should be considered in the phase of preliminary and final analysis for scientific research and expert projects.

Keywords: landslide, systematization, analytical methods, numerical methods, 2D and 3D modelling

PRIMER ZAŠTITE DUBOKE TEMELJNE JAME I SUSEDNIH OBJEKATA U SLOŽENIM URBANIM I GEOTEHNIČKIM USLOVIMA

EXAMPLE OF PROTECTION OF DEEP FOUNDATION PIT IN COMPLEX URBAN AND GEOTECHNICAL CONDITIONS

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1 UVOD

Buduči stambeno-poslovni objekat „Pupinova palata” nalazi se na uglu ulice Narodnih heroja i Bulevara Mihajla Pupina, u strogom centru Novog Sada. Objekat je moderno arhitektonsko rešenje, u kojem su objedinjeni poslovni i uslužni sadržaji koje predvi´a stroga urbana sredina grada, klasiđne stambene jedinice, elitni stambeni deo na najvišem spratu, sa zelenim krovom (penthaus), trospratna podzemna garaža sa 400 parking mesta za vlasnike objekta, s direktnom lift-vezom do svakog sprata i javna podzemna garaža za posetioce.

Ukupna površina objekta jeste oko 43.000 m^2. Objekat ima razu´enu spratnost – koja iznosi P+4, P+5, P+7, P+8 do P+12 na delu najviše kule – s tim što u nivou prizemlja ima i dva atrijuma za dodatnu komunikaciju, kao i vezu s kružnim požarnim putem.

S obzirom na to što je lokacija u strogom centru, okružena je starim postoječim objektima đija se spratnost kreče od P+1 do P+3, a konstruktivni sistem je zidani, s pruskom tavanicom, bez serklaža, dok samo manji deo objekata ima podrum. Gra´evinska linija novog objekta – u skladu s lokacijskim uslovima – nalazi se neposredno na liniji gabarita postoječih objekata, što omogučava maksimalno iskoriščenje postoječe površine gra´evinske parcele.

1 INTRODUCTION

The future residential and commercial building "Pupin's Palace" is located on the corner of Narodnih heroja street and boulevard Mihajlo Pupin in the very centre of Novi Sad. The building is a modern architectural solution that combines business and service facilities that are planned by a strict urban city centre, classic residential units, in elite residential area on the highest floor with a green roof (pent house), a three-story underground garage with 400 parking spaces for the owners of the facility with direct elevator-link to each floor and a public underground garage for visitors.

The total area of the building is about 43,000m^2. The building has different floors, which are 4, 5, 7, 8 to 12 on the part of the highest tower, and two atriums at the ground level for additional communication, and a connection with a circular fire route.

Since the location is in the centre of the city, it is surrounded by old existing buildings, the floor of which is from 1 to 3, and the constructive system is masonry, with a Prussian ceiling, without stiffness rib, and only a small part of the building has a basement. The construction line of the new facility is in line with the location conditions directly on the line of the existing buildings, which allows the max utilization of the existing surface.

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Slika 1. Izgled budućeg objekta „Pupinova palata" u Novom Sadu
Figure 1. The appearance of the future "Pupin's Palace" building in Novi Sad

Pre početka radova, svi susedni objekti detaljno su pregledani i snimljeni, u prisustvu stručnog sudskog veštaka, utvrđena su sva oštećenja u vidu prslina i pukotina, te je urađen poseban elaborat postojećeg stanja objekata.

Na predmetnoj lokaciji, pre početka radova, urađena su i detaljna geomehanička ispitivanja, koja su obuhvatila sondažne bušotine i oglede statičke penetracije (CPTu). Na osnovu terensko-laboratorijskih ispitivanja, utvrđeno je da profil terena predstavlja aluvion reke Dunav, koji izgrađuje čist pesak s tankim horizontalnim proslojcima zaglinjenog peska, u čijoj se podini pojavljuje peskovit sitnozrnasti do srednjezrnasti šljunak. Na dubini od 22–23 m pojavljuje se laporovita glina.

Teren je na lokaciji u blagom padu ka jugoistoku od 78.8 do 77.6 m nadmorske visine. U hidrogeološkom pogledu, sloj peska u direktnoj je hidrauličkoj vezi s Dunavom, tako da se dubina podzemne vode – u zavisnosti od vodostaja reke – kreće od minimalnih cca 1.0 m do maksimalnih cca 6.0 m. U toku istražnih radova (jun 2015), dubina podzemne vode bila je između 2.2-3.4 m od postojeće površine terena, odnosno na koti cca 75.3-75.5 metara nadmorske visine.

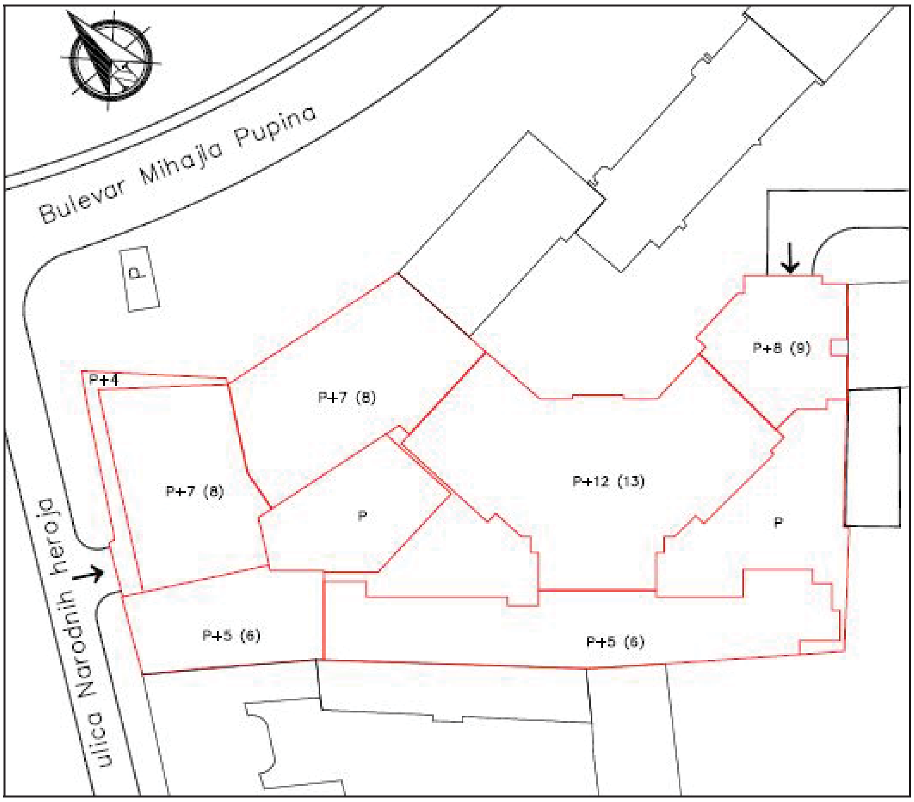
Gabaritna površina budućeg objekta jeste 4.100 m^2, a gabaritni obim – oko 301 m^1. Kao što je na Slici 2. evidentno, gabarit objekta okružen je postojećim objektima koji su neposredno na građevinskoj liniji, kao i vrlo frekvetnim saobraćajnicama. U takvom, vrlo nepovoljnom okruženju, koje se ogleda u starim susednim objektima, relativno visokom nivou podzemne vode, visokoj vodopropusnosti tla i minimalnom rastojanju krajnjih naspramnih strana zidova podzemne etaže od cca 36 m, trebalo je izvesti temeljnu jamu dubine 9 metara.

All adjacent objects were carefully inspected and recorded in the presence of a professional forensic expert, before the beginning of the work, by identifying all damages in the form of cracks, and a special study of the existing condition of the objects was made.

At the site, detailed geomechanical tests, including borings and static penetration (CPTu), were made before the beginning of the work. On the basis of field-laboratory tests, it was determined that the terrain profile represents the alluvium of the Danube river, which build clean sand with thin horizontal clayey-sand, at which bottom the sandy fine to medium-sized pebbles appear. At a depth of about 22-23 m, a clayey marl appears.

The terrain is a gentle slope towards southeast from 78.8-77.6m. In hydrogeological terms, the sand layer is in direct hydraulic connection with the Danube, so the depth of ground-water depending on the water level of the river, ranges from a min. 1.0m to a max. of 6.0m. During the geotechnical works (June 2015), the depth of the groundwater was between 2.2-3.4m from the existing surface of the terrain, or at sea level of approx. 75.3-75.5 m.

The basis area of the future object is 4,100m^2, and the circuit is about 301m^1. As shown in Figure 2, the facility's dimensions are surrounded by existing facilities that are directly on the construction line and busy street with heavy traffic. In such a very unfavourable environment, which is reflected in the old neighbouring buildings, a relatively high level of groundwater, high soil permeability and a min. distance of the final opposite sides of the walls of the underground floor of approx. 36m, a pit of 9m depth was to be performed.



Slika 2. Situacioni prikaz položaja objekta „Pupinova palata” u Novom Sadu
Figure 2. Display of the position of the "Pupin's Palace" building in Novi Sad

Analizirajući moguće varijante rešenja, u svim slučajevima zaključak jeste da je najbolje AB dijafragme spustiti do laporovite gline i time izbeći jak priliv podzemne vode i formiranje dubokog depresionog levka oko temeljne jame. Dijafragma je do dubine 15 m konstruktivna, a ispod toga je nearmirana i služi kao svojevrsna protivfiltraciona zavesa. Time se povećava površina dijafragmi za cca 2.100 m^2, što znači veću količinu i cenu iskopa i kontraktorskog betona, u iznosu od cca 190.000 evra. Međutim, realna dobit višestruko nadmašuje ovaj gubitak.

Prva dobit od izrade dublje dijafragme jeste to što se izbegava izrada 8-9 bunara bušenih do sloja laporovite gline, s pojedinačnim kapacitetom oko 20–25 l/s, koji zahtevaju složen potisni vod, dvadesetčetvoročasovni nadzor, stalnu pripravnost dizel agregata velike snage, prihvat i do 200 l/s podzemne vode iz bunara u sistem javne kanalizacije koja nema mogućnost prijema te količine vode u slučaju velike kiše, kao i troškove naknade JKP-a oko 40 evra dnevno po bunaru. Za procenjeno trajanje radova od minimum osam meseci, dok se s težinom konstrukcije ne savlada uzgon podzemne vode, samo troškovi naknade za JKP jesu oko 90.000 evra. Ako se na to još dodaju troškovi izrade osam-devet bunara s potisnim cevovodima, stalni nadzor i najam dizel agregata, iznos raste do oko 140.000 evra.

Nadalje, izradom dublje dijafragme, izbegava se formiranje širokog depresionog levka, koje bi nastalo intenzivnim dugotrajnim crpljenjem podzemne vode bunarima unutar temeljne jame. Takav depresioni levak, pri postojećem nivou podzemne vode, imao bi dubinu

Analyzing the possible variants of the solution, in all cases, it was concluded that the best diaphragm wall are to be lowered to the marl, thereby avoiding a strong inflow of groundwater and forming a deep depression funnel around the foundation pit. The diaphragm wall is constructive to the depth of 15m, and below it non-reinforced and serves as a kind of anti-filtration curtain. This increases the surface of the diaphragm wall by approx. 2,100m^2, which means a higher quantity and price of excavation and concrete, in the amount of approx. 190,000€. However, real profits outweigh this loss altogether.

The first gain from making the deeper diaphragm wall is that the generation of 8-9 wells drilled to a layer of marble clay with an individual capacity of about 20-25 l/s, requiring a complex pressure line, 24h control, constant standby power of high-power diesel units, 200 l/s of groundwater from the wells into a public sewage system that does not have the possibility of receiving this amount of water in case of heavy rainfall, and the cost of compensation for “JKP vodovod i kanalizacija” about 40€/day/well. For an estimated duration of works of at least 8 months, while the weight of the construction is not overgrown the uplift pressure, only the cost of the compensation is about 90,000€. If there are additional costs for the construction of 8-9 wells with pressure pipelines, constant monitoring and leasing of diesel engines, the amount increases to around 140,000€.

Secondly, by creating a deeper diaphragm wall, the formation of a wide depression funnel is avoided by intensive long-term drainage of groundwater with wells oko 6 m, što bi prema proračunima izazvalo sleganja (zbog porasta efektivnih napona u tlu do 60 kPa) koja iznose cca 4-5 cm nesposredno oko dijafragme, a postepeno bi se smanjivala udaljavajući od nje. Svi susedni objekti bili bi zahvaćeni depresionom zonom širine 15-30 m, u kojoj bi trpeli neravnomerna sleganja koja bi – zbog njihovog kostruktivnog sklopa – izazvala vidna oštećenja u vidu cm-pukotina u zidovima, što je neprihvatljivo.

Izrada bunara unutar temeljne jame ima i dodatan zahtev za posebnu obradu otvora u temeljnoj ploči debljine 80 cm, za prolaz bunarskih cevi, koje se po prestanku potrebe za crpljenjem vode moraju posebnim postupkom hermetički zatvoriti. To je dodatni neizbežni deo koji se mora raditi vrlo pažljivo, uz postepeno isključivanje/uključivanje rada pojedinih bunara, sa stalnom regulacijom proticaja, kako bi se sprečilo nadiranje podzemne vode.

Nakon što je odlučeno da se urade duboke dijafragme, analizirana su konstruktivna rešenja razupiranja. Poseban problem predstavljala je velika širina temeljne jame, koja je na najužem delu široka oko 36 metara. Ponuđene su tri varijante rešenja. Prvo rešenje bila je tzv. Top-Down metoda. To je sistem koji je odavno poznat, a koristi se u urbanim sredinama, na malom prostoru, kada je potrebna brza gradnja. Sistem podrazumeva istovremenu gradnju i podzemnih i nadzemnih etaža. Investitor – koji je ujedno bio i izvođač radova – procenio je da je ova metoda vrlo složena i skupa, jer dodatno zahteva izradu više stotina dubokih šipova velikog prečnika – kao oslonaca stubova objekta, podzemni iskop i transport, angažovanje obimne radne snage na malom prostoru i vrlo složenu organizaciju gradilišta.

Kao sledeće rešenje, detaljno je analizirana varijanta otkopa temeljne jame s paralelnom izradom prethodno napregnutih sidara koje bi obezbeđivale stabilnost dijafragme pri velikim bočnim pritiscima tla i podzemne vode. Imajući u vidu aluvijalne sedimente kao sredinu u kojoj bi se formirala sidrišna zona, računski kapacitet sidara bio je relativno mali, oko 250-300 kN, što je zahtevalo vrlo velik broj sidara (oko 600 komada), postavljenih u dva nivoa; cena bi bila oko 500.000 evra. Mada je investitor bio spreman na tu investiciju, budući da se dobija široka, sigurna i otvorena temeljna jama bez razupirača i privremenih bermi, od ove varijante odustalo se zbog zakonskih prepreka. Naime, prema srpskom zakonu, izradom ankera ulazi se u privatno vlasništvo suseda, od kojih investitor mora dobiti overenu pismenu saglasnost, a što je - s obzirom na već postojeće odnose sa susedima - procenjeno kao nemoguće.

Na kraju, investitor se odlučio na varijantu faznog iskopa temeljne jame uz fazno razupiranje dijafragme horizontalnim čeličnim razupiračima, pri čemu se kao obostrani oslonci razupirača koriste dijafragme, a kao dodatni jednostrani oslonci - već izvedeni podzemni delovi objekta. Generalno, usvojeno je pet (V) faza iskopa s razupiranjem, koji su u periodu od 6-7 meseci omogućili da se kompletira temeljna ploča na poslednjoj fazi, pri čemu je na prvoj fazi objekat završen približno do nivoa petog sprata.

Za potrebe crpljenja zarobljene podzemne vode unutar gabarita dijafragme, urađena su dva bunara kapaciteta od oko 15 l/s. Ovi bunari su napravljeni i

inside the foundation pit. Such a depressed funnel would have a depth of about 6m at the existing groundwater level, which would have led to settlements according to the calculations (due to an increase in the effective voltage in the soil to 60kPa), which would be about 4-5cm near to the diaphragm wall and gradually decrease away from it. All adjacent objects would be affected by a depression zone of 15-30m wide, in which they would suffer uneven settlements that would cause visible damages in the form of cm-cracks in the walls due to their poor building structure, which is unacceptable.

The construction of the well inside the foundation pit has additional requirements for the special treatment of openings in the 80cm thick foundation slab for the passage of well tubes which, after the need for water extraction, must be sealed by a special procedure. This is an additional inevitable part, which must be done very carefully, with the gradual shutdown-activation of the work of individual wells, with constant flow control in order to prevent the overflow of groundwater.

After deciding to do deep diaphragm wall, constructive solutions of bracing were analyzed. A special problem was the large width of the foundation pit, which is at the narrowest part about 36m. Three variants of the solution are offered. The first solution was the so-called. method "Top-Down". It is a system that has been known for a long time, and is used in urban part, in a small area, when rapid construction is required. The system implies the simultaneous construction of both underground and above ground floors. The investor, who was also the contractor, estimated that this method is very complex and costly, as it additionally requires the production of hundreds of deep piles of large diameter as supports of the columns, underground excavation and transport, engaging large workforce in a small space and very complex organization of construction sites.

As the next solution, in detailed are analyzed the variant of the excavation of the foundation pit with parallel production of pre-stressed anchors, which would ensure the stability of the diaphragm wall at large lateral soil and water pressures. Beyond a weak alluvial sediments as the anchor zone, the calculated capacity of the anchors was relatively small, about 250-300kN, which required a very large number of it, about 600, which would be placed in two levels, and the price it was around 500,000€. Although the investor was ready for this investment, which will give a wide, safe and open pit without braces and berms, this variant was abandoned due to legal obstacles. Namely, according to the Serbian law, the anchor enters the private ownership of the neighbours, of which the investor must obtain a certified written consent, which, given the mutual relations is estimated as impossible.

In the end, the investor decided on a variant of the phase excavation of the foundation pit with horizontal steel braces, relying on the diaphragm wall, and on the already constructed underground parts of the building. In general, a five (5) phase of excavation was adopted, which in the period of about 6-7 months enabled the completion of the foundation slab at the last stage, with the first stage being completed approximately to the level of 5 floors.

For the needs of the pumping of captured groundwater inside the circuit of diaphragm wall, two wells with capacity of about 15 l/s were made. These aktivirani neposredno po završetku dijafragme. Prema proračunu, nivo podzemne vode unutar temeljne jame trebalo je biti snižen na 50 cm ispod dna budućeg iskopa za jedanaest dana. Međutim, merenjem nivoa na dva kontrolna piezometra, utvrđeno je da sniženje ne napreduje po planiranoj dinamici. Analizom rezultata i dopunskim hidrauličkim proračunom, zaključeno je da postoji konstantan doticaj podzemne vode u količini oko 7 l/s, što usporava snižavanje nivoa u temeljnoj jami. Takođe, zaključeno je da je razlog za to bio u procurivanju na spojevima kampade dijafragme, što je bilo očekivano s obzirom na ukupan broj spojeva od oko 120, s pojedinačnom dužinom ispod podzemne vode oko 16 m, što je ukupno oko 2 km spojeva. Nakon sniženja podzemne vode na planirani nivo za oko 22 dana, nastavljeno je crpljenje u iznosu od 4-5 l/s do zvršetka kompletne temeljne ploče.

wells were created and activated immediately after the end of the diaphragm wall. According to the budget, the groundwater level within the foundation pit was to be reduced to 50cm below the bottom of the future pit in 11days. However, by measuring the level on two control piezometers it was established that the reduction is unlikely to progress according to the planned dynamics. By analyzing the results and the additional hydraulic calculation, it was concluded that there is a constant inflow of groundwater in the amount of about 7 l/s, which slows down the level reduction. It was concluded that the reason was in the flow leakage on the joints of the diaphragm wall, which was expected due to the total number of joints of about 120, with an individual joint length below the ground water of about 16m, which is a total of about 2km of joints. After lowering the groundwater to the planned level in about 22 days, it continued with pumping in the amount of 4-5 l/s to the end of the completion of the slab.

2 GEOTEHNIČKI USLOVI NA LOKACIJI

Šire područje oko lokacije pripada krajnjem južnom delu velike bačke ravnice koja se prostire od državne granice na severu do Fruške gore na jugu. Morfologija terena je tipično ravničarska, blago zatalasana, aluvijalno fluvijalnog porekla. Nadmorska visina lokacije, u blagom padu prema jugoistoku, kreće se između 78.8 do 77.6 m nadmorske visine. Predmetna lokacija jeste neuređen parking prostor (Slika 3), u strogom centru Novog Sada, koji svojim izgledom godinama narušava gradsko jezgro.

2 GEOTECHNICAL CONDITIONS AT THE LOCATION

The wider area around the site belongs to the far southern part of the big bačka plain which extends from the state border in the north to Fruška Gora in the south. The morphology of the terrain is typically a flat, slightly stratified, alluvial fluvial origin. The altitude of the site, slightly down to the southeast, ranges from 78.8-77.6m. The location is an unregulated parking space (Fig 3), in the strict centre of Novi Sad, disturbing the city core for years.

Slika 3. Izgled parcele za objekat „Pupinova palata” u Novom Sadu
Figure 3. View of the parcel for the "Pupin’s Palace" building in Novi Sad

Primarna morfologija izmenjena je antropogenim uticajem prilikom urbanizacije, što potvrđuje sloj antropogenog nasipa promenljive debljine, utvrđenog istražnim bušenjem, dok dublje slojeve izgrađuju kvartarni sedimenti (OGK Srbije: List 34-100 Novi Sad), holocene starosti preko sedimenata neogena (pliocen).

Kvartarni sedimenti javljaju se u faciji starača (am), kao i u faciji korita i povodnja (alp), koje izgrađuju najmlađi deo aluvijalne ravni Dunava, u čijem sastavu preovlađuju organogeno-barski pesak, razni alevriti i alevritske gline. U faciji korita preovlađuju srednjezrnasti do krupnozrni šljunak i sivi srednjezrni pesak, dok faciju povodnja predstavljaju žuti, liskunski, alevritski pesak i

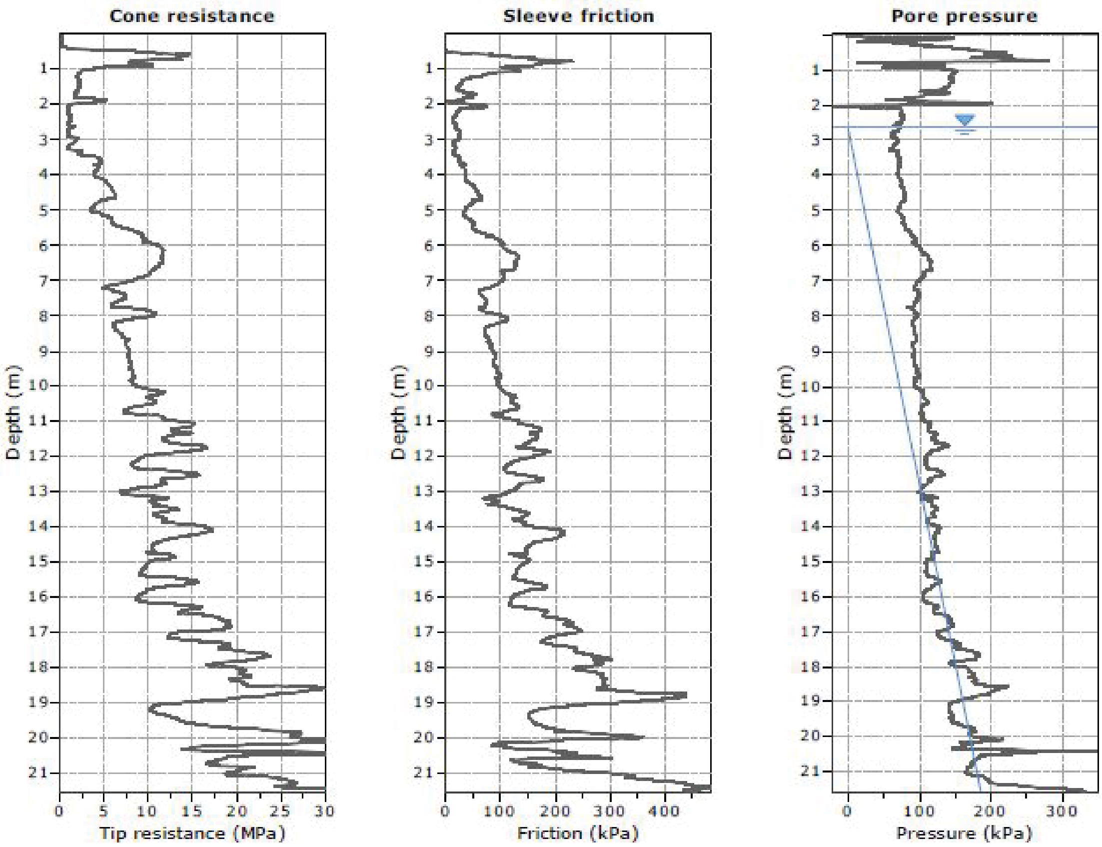
Primary morphology has been changed by influence in the urbanization process, which is confirmed by the layer of building dump of variable thickness, determined by investigative drilling, while deeper layers are constructed by quaternary sediments (OGK Srbije: List 34-100 Novi Sad), Holocene ages through sedimentary Neogene’s (Pliocene).

Quaternary sediments occur in the facies of the aging (am), as well as in the facies of the wrecks (alp), which build the youngest part of the alluvial plane of the Danube, which consists of organogenic-sand, various alevrites and alevritic clays. The facies of bed prevail medium to large-scale gravel and gray medium-sized peskoviti alevriti.

Neogeni sedimenti pružaju se u podini šljunka facije korita. Javljaju se u vidu laporovitih glina pliocene starosti. Laporovite gline su žute do tamnosmeđe boje, prašinastog sastava, s tankim proslojcima i sočivima peska, vrlo niske vodopropusnosti.

Na predmetnoj lokaciji, osim sondažnih bušotina, urađeno je i više penetracionih ispitivanja (CPTu), tačnije ukupno pet, a tipičan penetracioni profil prikazan je na Slici 4.

sand, while the facies of the basin is represented by yellow, marshy, alevritic sand and sandy alevrites. Neogene sediments are provided in the underground gravel facies of bed. They appear in the form of clayey marl of Pliocene age. Clayey marl are yellow to dark brown colour, very low permeable, silty composition, with thin sediments and sand lenses. At the site, besides drill holes, a number of penetration tests (CPT) were performed, a total 5 pieces, and a typical penetration profile is shown in Fig. 4.



Slika 4. Tipičan penetracioni profil (CPTu4) na predmetnoj lokaciji
Figure 4. A typical penetration profile (CPTu4) at the site

Koristeći rezultate statičke penetracije (CPTu), mogu se korelisati neki parametri tla, kao što su vodopropusnost, broj udaraca N_{60} iz standardnog penetracionog ogleda (SPT), Young-ov modul elastičnosti za nivo mobilizovane deformacije od 10^{-3} (Robertson, 2009), relativna zbijenost, ugao smičuće čvrstoće (Kulhawy & Mayne, 1990) i drugo (Slika 5).

Rezultati laboratorijskih ispitivanja i korelacije, na osnovu CPTu, korišćeni su u okviru projekta zaštite temeljne jame, za definisanje naponsko-deformacijske karakteristike tla za numeričku simulaciju, u softverskom paketu „GeoStudio". Na osnovu istražnih bušotina i laboratorijskih ispitivanja, na datoj lokaciji mogu se izdvojiti sledeći litološki članovi:

1) Nasip (n), izgrađen pretežno od prašinasto-peskovitog materijala (ML,SF), pomešanog s čvrstim građevinskim otpadom. Zapreminska težina sloja jeste $\gamma \approx 19.5$ kN/m^3, kohezija – $c' \approx 5.0$ kPa, a ugao smičuće

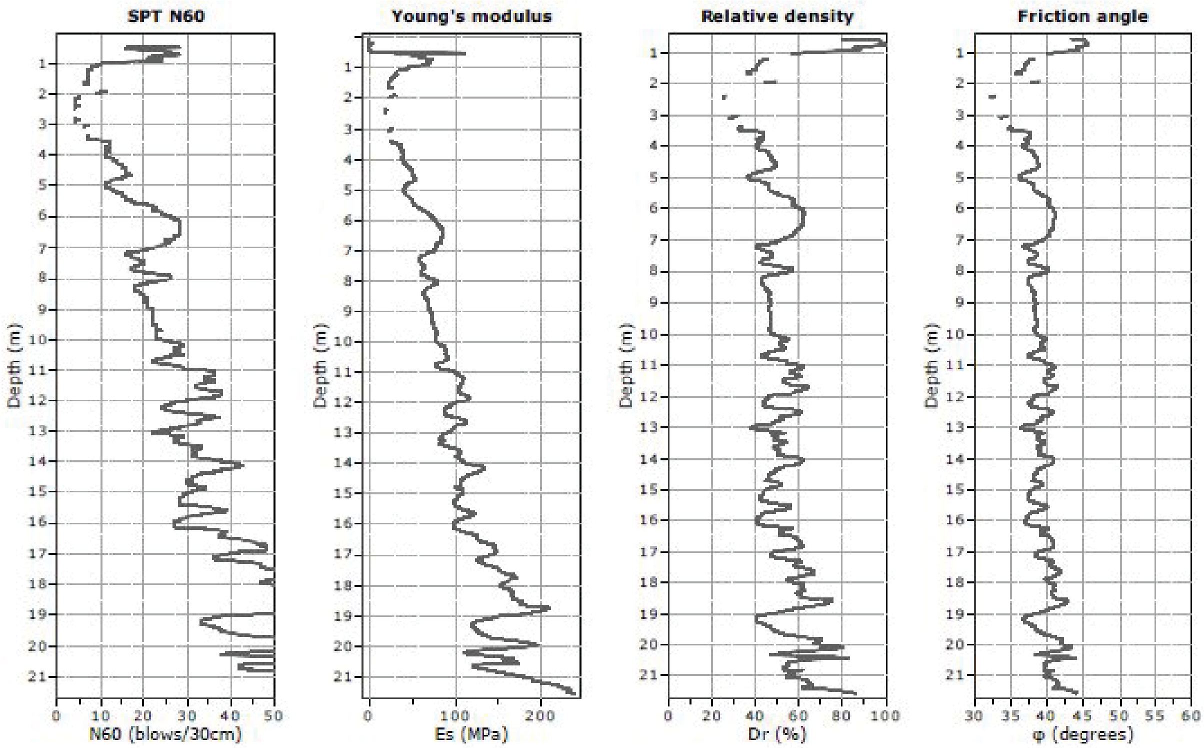
Using the results of static penetration (CPTu), some soil parameters can be correlated, for example, permeability, number of impacts N60 from the standard penetration test (SPT), Young's modulus of elasticity at the 10^{-3} level of mobilized deformation (Robertson, 2009), relative compactness, angle of shear strength (Kulhawy & Mayne, 1990) and others (Fig. 5.)

The results of laboratory tests and correlations based on CPT's were used within the framework of the project for the protection of the foundation pit, for defining the stress-deformation characteristics of the soil, for numerical simulation in the software GeoStudio. Based on borings and laboratory tests, the following lithological members are identified:

1) Building dump (n), built mainly of dust-sandy material (ML, SF), which is mixed with solid construction waste. The bulk density of the layer is $\gamma \approx 19.5$ kN/m^3, the cohesion is $c' \approx 5.0$ kPa, the angle of shear strength

čvrstoće – $\phi' \approx 25^0$. Podina sloja je između 2–3 m od površine terena.

$\phi' \approx 25^0$. The floor of the layer is between 2-3m from the ground surface.



Slika 5. Tipičan rezultat korelacije nekih parametara tla na osnovu CPTu
Figure 5. A typical result of the correlation of some soil parameters based on CPT

2) Prašina (ML–CL) peskovita, malo zaglinjena, niskoplastična, meke konzistencije, tamnosive boje. Zapreminska težina sloja jeste $\gamma \approx 19.5$ kN/m^3, kohezija – $c' \approx 2.0$ kPa, a ugao smičuće čvrstoće – $\phi' \approx 23^0$. Podina sloja je na dubini oko 4.5–5 m od površine terena.

3) Pesak (SF–SC) sitnozrnast do srednjezrnast, sive do sivožute boje, u povlatnom delu prašinastiji, s povećanjem dubine – čistiji i bolje zbijen, u intervalima s primesama oksida gvožđa. U sloju su uočljiva sočiva šljunka, peščara i glinovite prašine meke konzistencije. Zapreminska težina sloja jeste $\gamma \approx 20.0$kN/m^3, kohezija – $c' \approx 1.0$ kPa, a ugao smičuće čvrstoće – $\phi' \approx 32^0$. Podina sloja je na dubini između 15–19 m od površine terena.

4) Šljunak (GF, GW) peskovit, srednjezrnast do sitnozrnast, sive do sivoplave boje, u podinskom delu više peskovit, s proslojcima peskovite prašine crne boje i ostacima treseta. Zapreminska težina sloja jeste $\gamma \approx 21.0$ kN/m^3, kohezija – $c' \approx 0$ kPa, a ugao smičuće čvrstoće – $\phi' \approx 33^0$. Debljina sloja je između 1.7–3 m.

5) Prašina (ML-CL) peskovita, malo zaglinjena, niskoplastična, meke do srednjeplastične konzistencije, tamnosive boje, s primesama oksida gvožđa. Zapreminska težina sloja jeste $\gamma \approx 20.5$ kN/m^3, kohezija – $c' \approx 7.0$ kPa, a ugao smičuće čvrstoće – $\phi' \approx 28^0$. Sloj je utvrđen na dubini između 21.7–23 m od površine terena.

2) Silt (ML-CL) sandy, slightly clayey, low-plastic, soft consistency, dark gray. The bulk density of the layer is $\gamma \approx 19.5$ kN/m^3, the cohesion is $c' \approx 2.0$kPa, the angle of shear strength $\phi' \approx 23^0$. The bottom of the layer is at ≈4.5-5 m from the ground surface.

3) Sand (SF-SC) fine to medium-sized, gray to gray-yellow, in the floor part more silty, and deeply cleaner and more dense, at intervals with admixtures of iron oxide. In the layer there are gravel, sandstone and clayey silt of soft consistency. The bulk density of the layer is $\gamma \approx 20.0$ kN/m^3, the cohesion is $c' \approx 1.0$ kPa, the angle of shear strength is $\phi' \approx 32^0$. The bottom of the layer is at a depth of 15-19 m from the surface of the ground.

4) Gravel (GF, GW) sandy, medium to fine grains, gray to gray-blue, at the bottom more sandy, with sediments of black sandy silt and peat remains. The bulk density of the layer is $\gamma \approx 21.0$ kN/m^3, the cohesion is $c' \approx 0$, the angle of shear strength $\phi' \approx 33^0$. The thickness of the layer is between 1.7-3m.

5) Silt (ML-CL) sandy, low clayey, low plastic, soft to medium-plastic consistency, dark gray, with admixtures of iron oxide. The bulk density of the layer is $\gamma \approx 20.5$ kN/m^3, the cohesion is $c' \approx = 7.0$ kPa, the angle of shear strength $\phi' \approx 28^0$. The layer is determined at a depth between 21.7-23m from the surface of the terrain.

6) Glina (CI-CH) srednjeplastična do visokoplastična, tvrdoplastične konzistencije, žute do tamnosmeđe boje, s primesom oksida mangana i gvožđa i $CaCO_3$ u vidu praha i konkrecija cm-dimenzija. Zapreminska težina sloja jeste $\gamma \approx 20.5 kN/m^3$, kohezija – $c' \approx 75.0$ kPa, a ugao smičuće čvrstoće – $\phi' \approx 18^0$. Sloj se prostire do dna istražnog bušenja, do 35 m od površine terena.

Parametri deformabilnosti slojeva za numeričku simulaciju usvojeni su na osnovu rezultata terensko-laboratorijskih ispitivanja. Imajući u vidu nelinearnu vezu između parametra deformabilnosti i efektivnog napona, korišćena je sledeća jednačina (Duncan, 1980):

Clayey marl (CI-CH) medium to high plasticity, hard consistency, yellow to dark brown with the admixture of manganese and iron oxide and $CaCO_3$ in the form of powder and grains of cm-dimensions. The layer bulk density is $\gamma \approx 20.5 kN/m^3$, cohesion c'=75.0 kPa. The layer extends to the bottom of the drillings, up to 35m from the ground surface.

The deformability parameters of the layers for numerical simulation were estimated on the results of field-laboratory testing. Bearing in mind the nonlinear relationship between the deformability parameter and the effective stress, the following equation is used (Duncan, 1980):

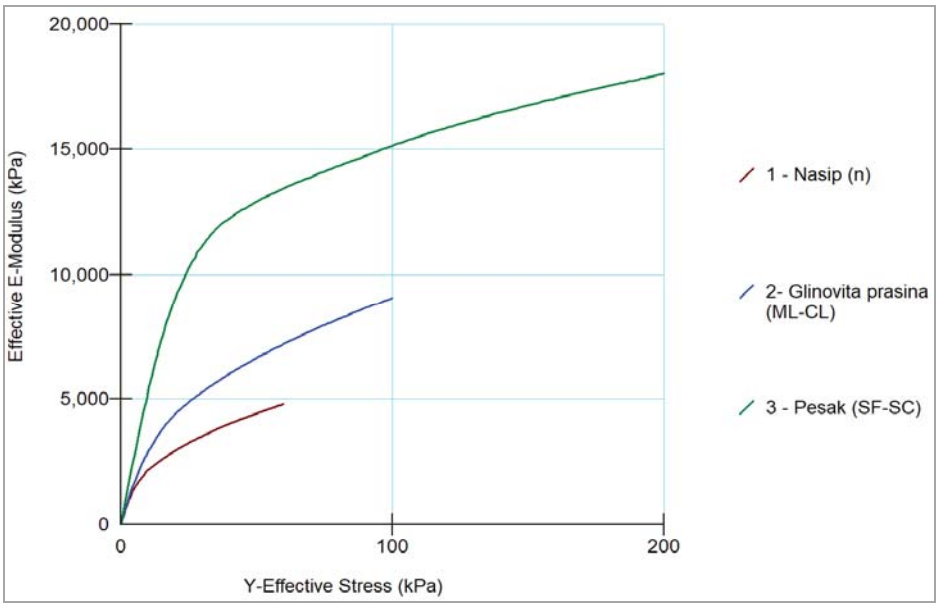
$$E = Kp_a \left(\frac{k_0 \sigma'_v}{p_a} \right)^N , \quad p_a = 100\, kPa$$

Ulazni parametri za prikazanu jednačinu dati su u tabeli 1, a grafički prikaz Young-ovog modula elastičnosti u funkciji efektivnog vertikalnog napona – na Slici 6.

The input parameters for the equation are given in Table 1, and the graphic representation of the Young Modulus Module in the function of effective vertical stress is shown in Fig. 6.

Tabela 1. Parametri deformabilnosti slojeva
Table 1. Parameters of deformability of layers

	(n)	(ML-CL)	(SF-SC)	(CI-CH)
E (MPa)	-	-	-	30.0
K	60.0	90.0	150.0	-
N	0.45	0.45	0.25	-
k_0	1.0	1.0	1.0	-
ν	0.33	0.30	0.33	0.33



Slika 6. Young-ov modul elastičnosti slojeva u funkciji efektivnog napona
Figure 6. Young's modulus of elasticity of layers in the function of effective stress

Nivo podzemne vode na lokaciji, tokom ispitivanja (jun 2015), bio je na dubini između 2.2–3.4 m od površine terena koji je u blagom padu ka jugoistoku, od 78.8 do 77.6 m nadmorske visine. Nivo podzemne vode je promenljiv, pa je za potrebe praćenja nivoa ugrađen jedan piezometar ∅50 mm, do dubine od 20 m. Najveći deo kvartarnih sedimenata zastupaju peskak i šljunak, koji se odlikuju međuzrnskom poroznošću i čine hidrogeološke rezervoare. U tim sedimentima formirana je slobodna izdan, čija je gornja granica u direktnoj hidrauličkoj vezi s Dunavom, pa se dubina podzemne vode – zavisno od nivoa Dunava – kreće od min. cca 1.0 m do max. cca 6.0 m. U podini ove izdani pojavljuju se sedimenti neogena, koji predstavljaju hidrogeološki izolator, a izgrađeni su od veoma malo propusnih laporovitih glina.

3 PRORAČUN ZAŠTITNE KONSTRUKCIJE OD AB DIJAFRAGMI I RAZUPIRAČA

Na osnovu analize različitih varijanti iskopa i razupiranja, odlučeno je da se – kao najpovoljnije rešenje – primene fazni iskopi temeljne jame, uz fazno razupiranje dijafragme.

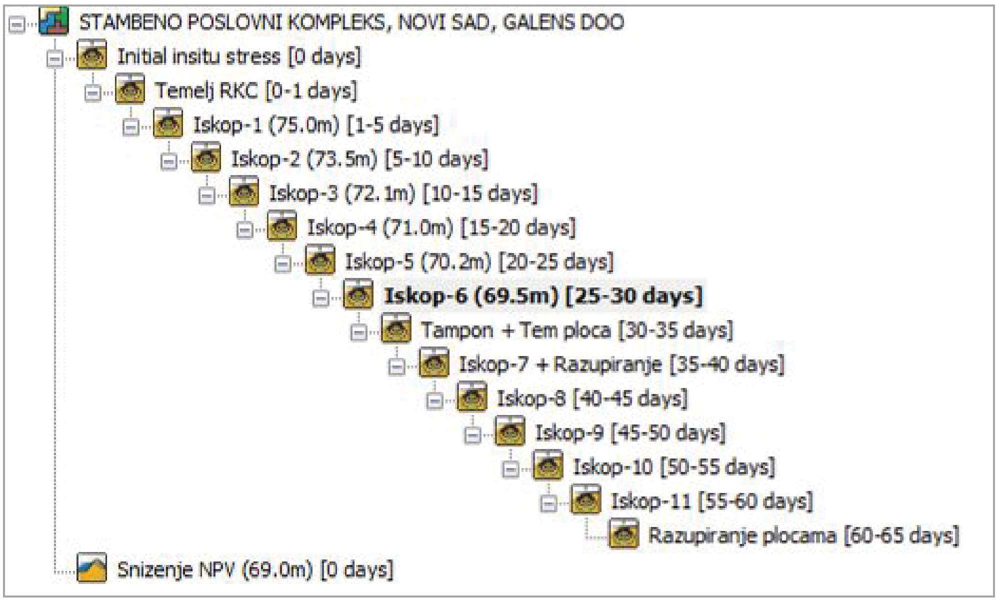
Ceo proces iskopa i razupiranja pojednostavljeno je modeliran u geotehničkom softveru „GeoStudio” za ravansko stanje deformacija. Imajući u vidu veliku vodopropusnost slojeva, korišćena je analiza sa efektivnim parametrima u dreniranim uslovima. Analiziran je kritičan presek u kojem se nalazi postojeći zidani objekat koji vrši dodatno horizontalno opterećenje na AB dijafragmu. Program koristi metodu konačnih elemenata i inkrementalno-iterativni postupak za rešavanje sistema nelinearnih algebarskih jednačina. U modelu postoji ukupno 14 inkremenata, od kojih „Initial Insitu Stress” predstavlja inicijalno naponsko za definisanje nelinearnih parametara deformabilnosti u funkciji napona, dok se u „Temelj RKC” unosi uticaj postojećeg objekta.

The groundwater level at the site (June 2015) was at a depth of 2.2-3.4m from the ground surface, which is slightly inclined to the southeast from 78.8-77.6m. The ground-water level is variable, so a ∅50mm piezometer up to a 20m depth is installed to monitoring purposes. Most of the quaternary sediments are represented by sand and gravel, which are distinguished by integranular porosity and make up hydrogeological reservoirs. In these sediments, a free issue is formed, whose upper limit is in direct hydraulic connection with the Danube, so the depth of the groundwater depending on the Danube level ranges from min. 1.0m to max. 6.0m. In the bottom of this issue, sediments of Neogene's representing a hydrogeological isolator are present, built from very low permeable clayey marl.

3 THE DESIGN OF THE RC DIAPHGRAM WALLS AND STRUTS

Based on the analysis of various variants of excavation and bracing, it was decided to use the phase digging and bracing of the diaphragm wall as the most favourable solution.

The entire process of digging-bracing is simplified and modelled in the geotechnical software "GeoStudio", for plane deformation state. Considering the high permeability of the layers, the analysis with effective parameters in drained conditions was used. A critical cross-section in which the existing masonry is located, which performs additional horizontal load on the AB diaphragm wall, has been analyzed. The program uses the finite element method and an incremental-iterative procedure for solving the system of nonlinear algebraic which the "Initial Insitu Stress" represents an initial stress for the definition of non-linear deformability equations. The model has a total of 14 increments, of parameters in the function of stress, while the "RKC"



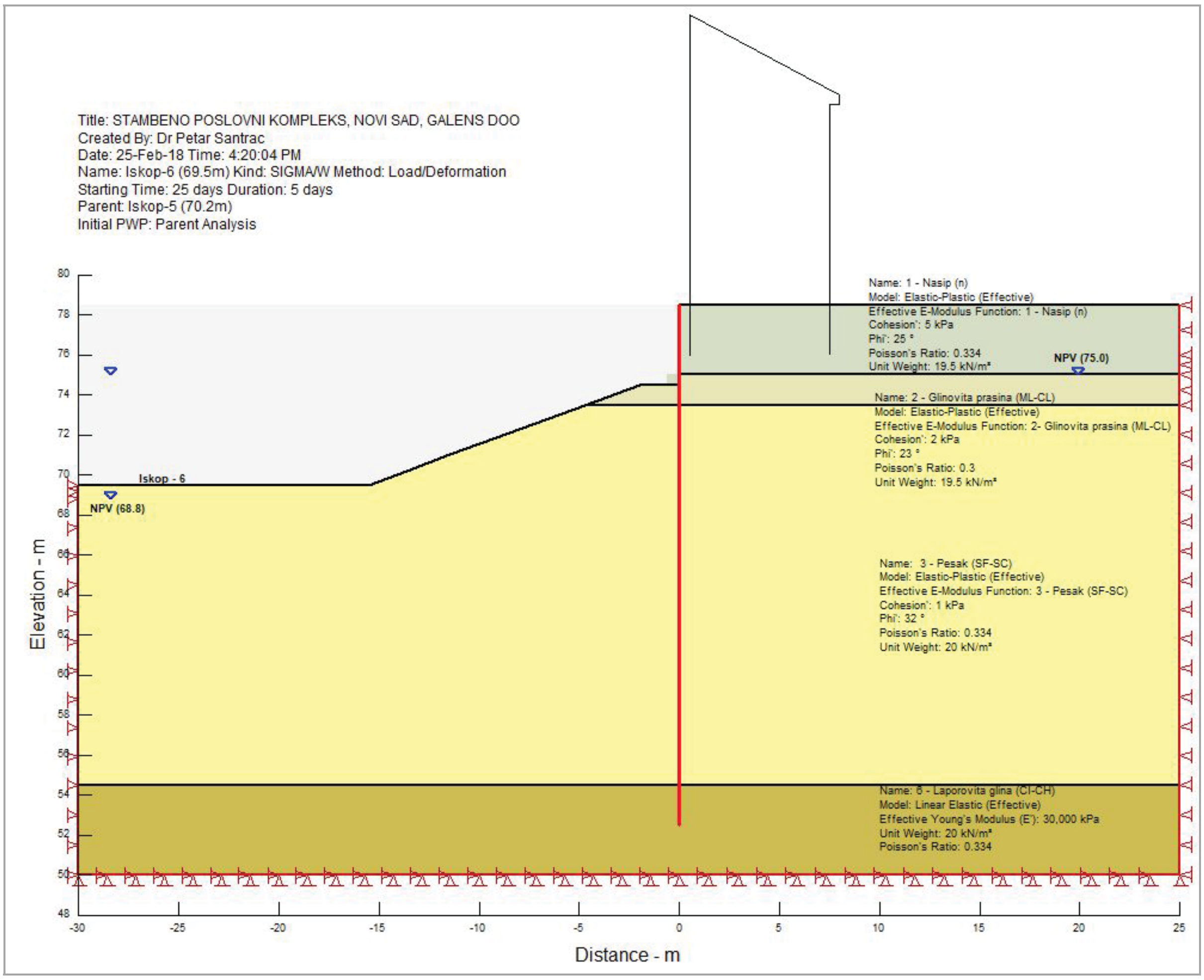
Slika 7. Pojedinačna stanja (inkrementi) definisani u numeričkom modelu za „GeoStudio”
Figure 7. Partial states (increments) defined in numerical model for the GeoStudio

Po završetku AB dijafragme, prvo se radi širok iskop (cca 4.100 m^2) do dubine od 4 m, što je približno oko 0.5 m iznad trenutnog nivoa podzemne vode, kako bi se formirao radni plato na koti 75 m. U numeričkom modelu, to je „Iskop-1”.

S radnog platoa urađena su dva bunara, do sloja laporovite gline, kapaciteta 15l/s, za crpljenje i snižavanje nivoa podzemne vode unutar temeljne jame. Nakon što je nivo podzemne vode u roku od približno tri nedelje snižen na oko 0.5 m ispod dna konačnog iskopa, pristupilo se iskopu do kote 69.5 m, s privremenom bermom uz dijafragmu. U numeričkom modelu, ovo stanje jeste „Iskop-6”, a predočeno je na Slici 8.

enter the influence of the existing object. At the end of the diaphragm wall execution, a wide excavation (approx. 4,100m^2) is completed to a depth of 4.0m, which is about 0.5m above the current groundwater level, in order to form a working plateau a level of 75.0m. In the numerical model, this increment is ″Iskop-1”.

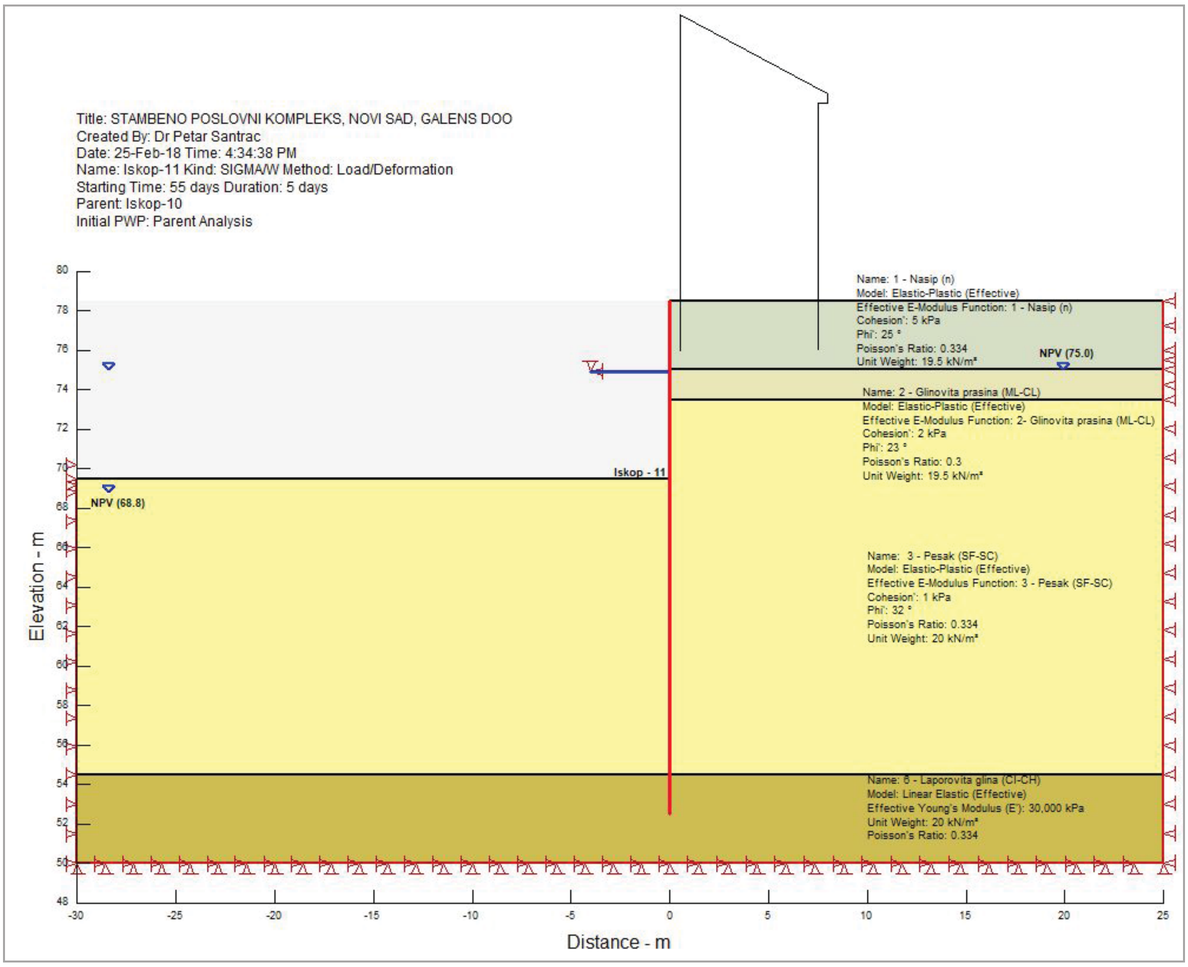
From the working plateau, 2 wells were made, to a layer of clayey marl, with capacity of 15 l/s, for pumping and lowering the level of groundwater inside the foundation pit. After its lowering to about 0.5m below the bottom of the final excavation, within approximately 3 weeks, a digging up to the level of 69.5m, with a temporary berm in front of diaphragm wall, has begun. In the numerical model, this state is "Iskop-6", and is shown in Fig. 8.



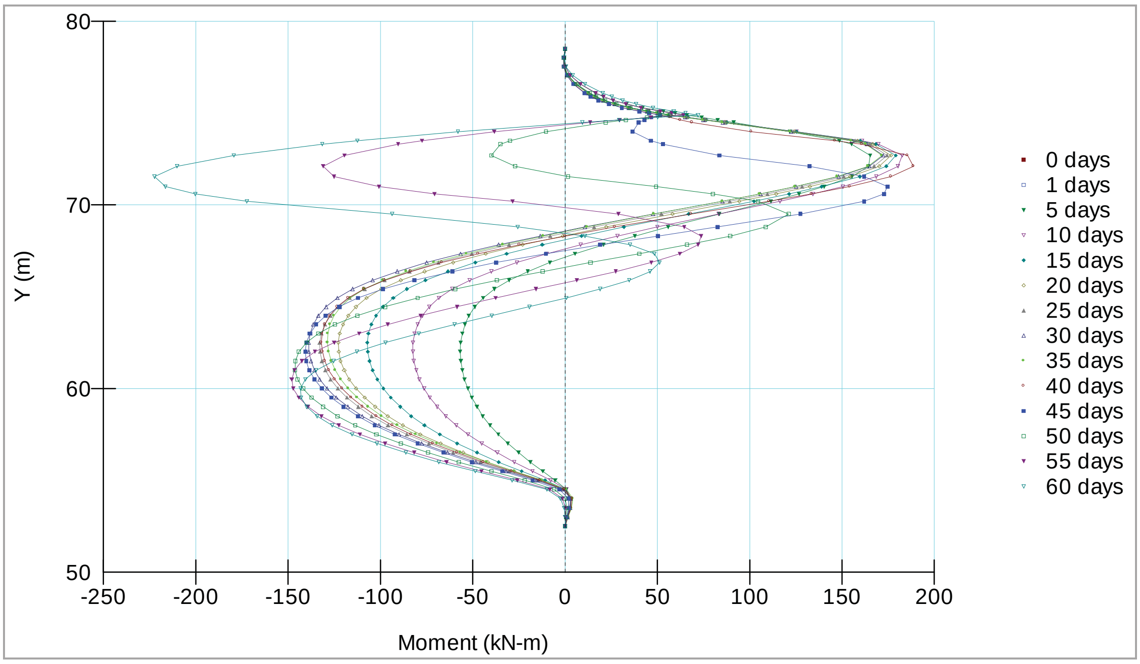
Slika 8. Privremeno stanje s bermom, nakon iskopa do dna temeljne jame
Figure 8. Temporary condition with berm after excavation to the bottom of the pit

Snižen nivo podzemne vode i privremena berma omogućuju izradu dela temeljne ploče, na kojoj se gradi podzemni deo konstrukcije. Uklanjanje berme radi se postupno, nakon postavljanja razupirača između dijafragmi i izgrađenih elemenata konstrukcije (stubovi i ploče) podzemnog dela objekta. Privremeno stanje, nakon postavljanja čeličnog razupirača i potpunog uklanjanja berme, prikazano je na slici 9.

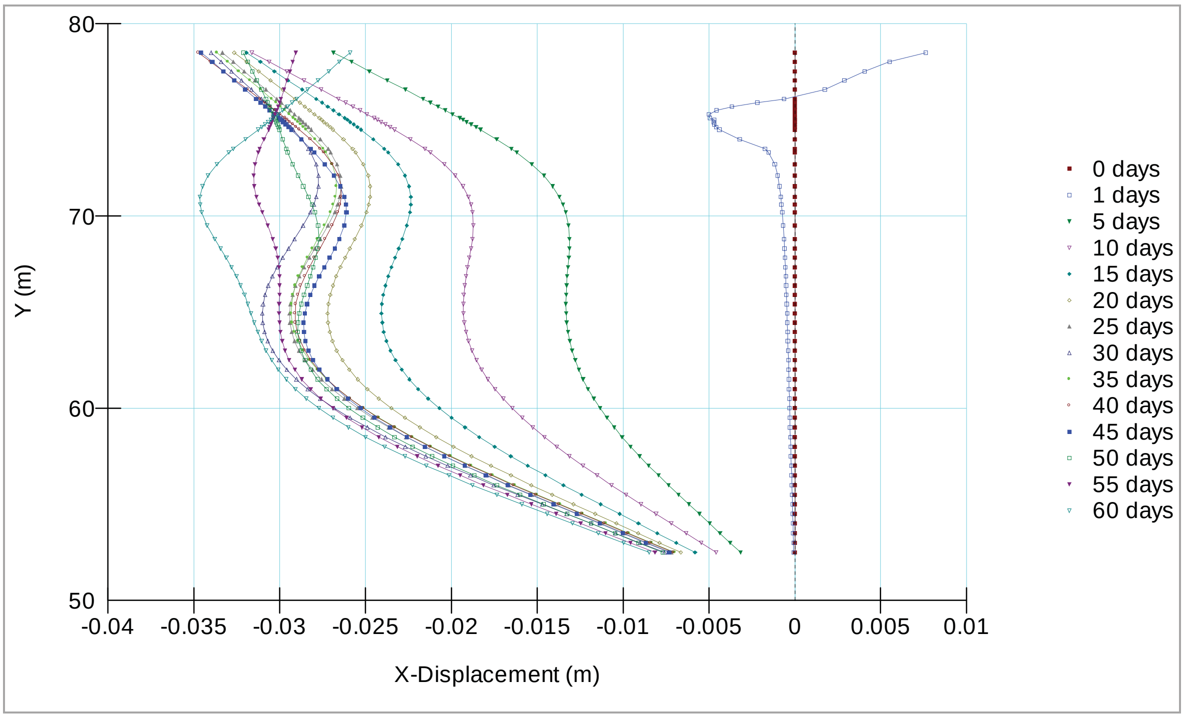
The lower groundwater level and the temporary berm allow the execution of the part of the foundation slab. The berm is removed gradually after the placement of the struts between the diaphragm wall and between the diaphragm wall and the built elements of the structure (columns and slabs) of the underground part of the building. The temporary state after installation of the struts and complete removal of the berm is shown in Fig. 9.



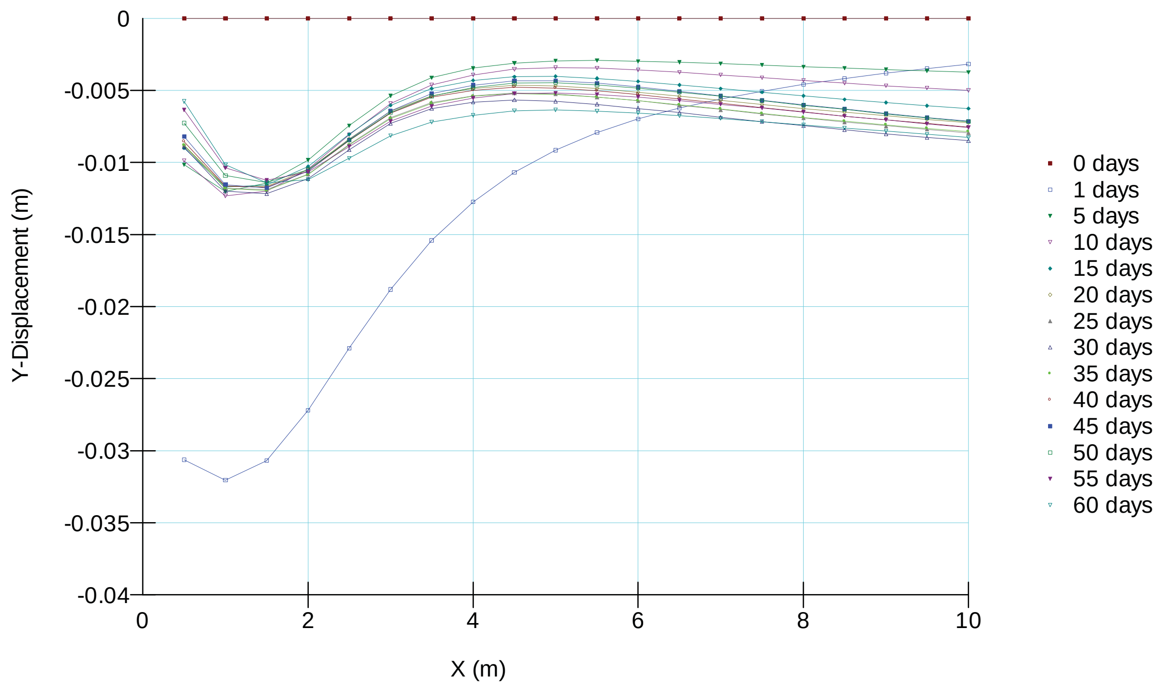
Slika 9. Privremeno stanje po završetku iskopa, s razupiračem i uklonjenom bermom
Figure 9. Temporary condition with strut after completion of the excavation



Slika 10. Momenti savijanja u dijafragmi tokom iskopa i razupiranja
Figure 10. Bending moments in the d-wall during excavation and bracing



Slika 11. Horizontalno pomeranje dijafragme tokom iskopa i razupiranja
Figure 11. Horizontal displacement of the d-wall during excavation and bracing



Slika 12. Sleganje zaleđa dijafragme tokom iskopa i razupiranja
Figure 12. Settlements behind the d-wall during excavation and bracing

Razupiranje se radi čeličnim cevima ∅600 mm, koje se vare na čelične HOP-podvlake koje su zavarene za ubušene ankere ∅22 u dijafragmu. Na mestu gde je temeljna jama najšira (36m), umesto čelične cevi postavljen je razupirač od čelične rešetke.

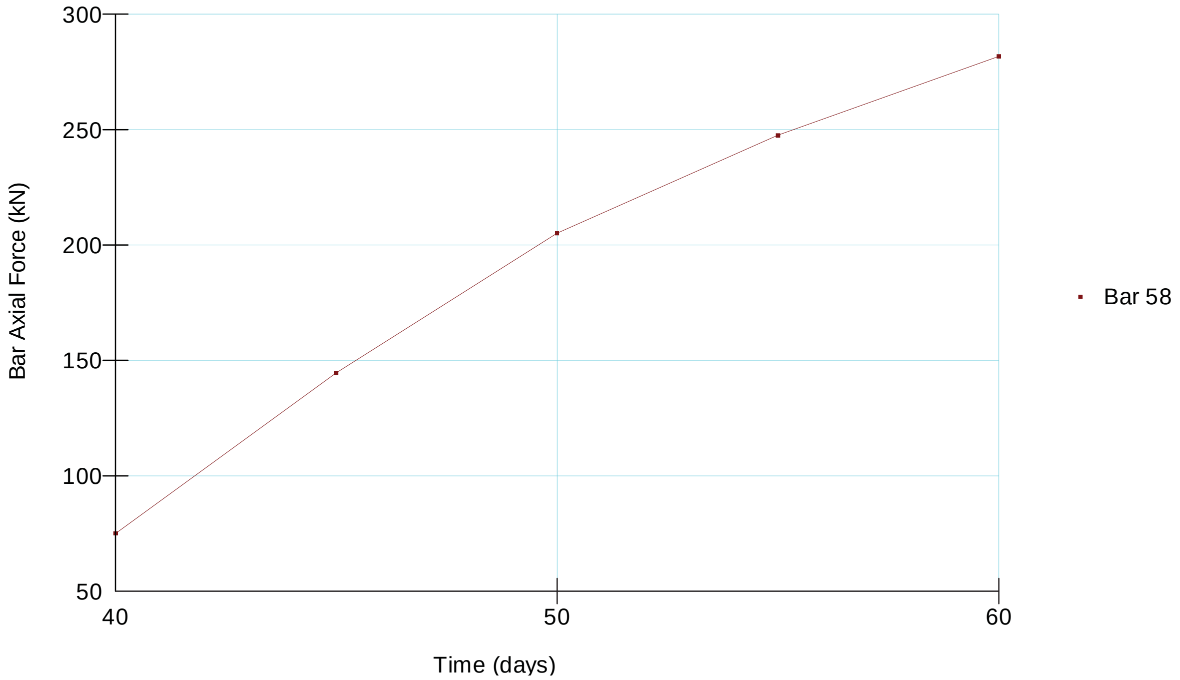
The bracing is done with ∅600mm steel tubes, which are applied to steel beam underlays welded to battered ∅22mm anchors in the diaphragm wall. At the point where the width of the pit is the largest, up to 36m, instead of pipe, the steel trust grid is laid out. The

Proračunom su određeni svi relevantni podaci za dimenzionisanje i procenu stanja – kao što su momenti savijanja u dijafragmi i pomeranje dijafragme, sila u razupiraču, pomeranje i sleganje terena ispod postojećeg objekta u zaleđu dijafragme i tome slično.

Maksimalni računski moment savijanja jeste 180 kNm u zaleđu, a 230 kNm u pročelju dijafragme. Maksimalno horizontalno računsko pomeranje dijafragme je 34 mm, a najveće pomeranje izmereno geoedetskim instrumentom jeste oko 40 mm. Maksimalno računsko sleganje zaleđa zida je oko 32 mm, dok je geodetski izmereno maksimalno sleganje oko 21 mm. Maksimalna računska sila u razupiraču jeste 280 kN/m.

calculation in GeoStudio determines all relevant data for dimensioning and assessment of the condition, such as the bending moments and the displacement of the diaphragm wall, the force in the strut, the displacements and settlements of the terrain beneath the existing object in the back of the diaphragm wall, and others.

The maximum bending moment is 180kNm in the back of d-wall and 230kNm in front of it. The maximum horizontal computed movement of the d-wall is 3.4cm, and the maximum movement measured by the geodesic survey is about 4cm. The maximum settlement behind the d-wall is about 3.2cm, while the measured movement occurred by geodesic survey if about 2.1cm. The maximum estimated force in the strut is 280 kN/m.



Slika 13. Sila u razupiraču tokom iskopa
Figure 13. Strut axial force during the excavation

4 KRATAK PRIKAZ IZVOĐENJA RADOVA

Iskop temeljne jame rađen je fazno, u vertikalnom i horizontalnom smislu. Vertikalna faznost prikazana je u prethodno opisanom računskom modelu. Horizontalna faznost bila je neophodna zbog toga što su se razupirači oslanjali ne samo na dijafragme međusobno, već i na izvedene delove podzemne konstrukcije. Načelno, bilo je pet (V) horizontalnih faza iskopa s razupiranjem, koje su - u periodu od oko 6-7 meseci – omogućile da se kompletira temeljna ploča na poslednjoj fazi, pri čemu je na prvoj fazi objekat završen približno do nivoa 5. sprata.

4 SHORT DESCRIPTION OF THE WORKS

The excavation of the foundation pit has been carried out in phases, in a vertical and horizontal sense. Vertical phases is shown through the previously described computational model. Horizontal phases was necessary because the struts relieved not only between the diaphragm wall, but also on the derived parts of the underground structure. In principle, there were five (V) horizontal excavation phases, which in the period of about 6-7 months, enabled the foundation slab to be completed at the last stage, with the first stage being completed approximately to the level of 5 floors.

Slika 14. Razupirači – oslonjeni samo na dijafragme u I fazi (10.05.2017)
Figure 14. The struts relying on RC diaphragm wall in phase-I (10.05.2017)

Slika 15. Razupirači i armatura temeljne ploče u I fazi (21.05.2017)
Figure 15. The reinforcements of the foundation slab in phase-I (21.05.2017)

Prva faza obuhvatila je krajnji severoistočni deo građevinske parcele, na kojoj se pojavljuje stambena jedinica spratnosti P+8(9). To je najuži deo lokacije, koji je omogućio da se izvrši razupiranje dijafragme o dijafragmu, preko čeličnog razupirača (najduže – 27.5 m). Radi ravnomernijeg prijema sile s podvlake, razupiraču su dodati bočni kosi kraci. Na slikama su vidljiva mestimična procurivanja vode na spojevima dijafragmi, koji se injektiraju epoksidnim smolama pre postavljanja hidroizolacije. Takođe, uz dijafragme, uočava se povijena hidroizolacija iz temeljne ploče.

The first phase included the far north-eastern part of the building plot, where the housing unit P+8 appears. It is the closest part of the site, which enabled the diaphragm wall to be pulled out, through a steel pipe, with the largest length up to 27.5m. For the evener reception of the force from the walls, the lateral pieces are added to the pipes. In the pictures, there are visible spots of water leaks on the diaphragm wall joints, which are injected with epoxy resins prior to the installation of waterproofing. Also, along the walls, bended waterproofing from the slab is visible. The waterproofing

Hidroizolacija se lepi za betonsku površinu, a nastavlja se na preklop, uz specijalni lepak. Na prednjem delu slike 16 vidi se razupiranje dijafragme o stub izgrađenog dela konstrukcije.

is adhered to the concrete surface and continues to the transition with a special adhesives. The struts between diaphragm wall and built column of the structure appear at the lower front of Fig. 16.

Slika 16. Razupirači oslonjeni na dijafragme i delove konstrukcije u II fazi (21.06.2017)
Figure 16. Struts relying on d-wall and on the part of construction in phase-II (21.06.2017)

U fazi III – zbog velike širine temeljne jame – odlučeno je da se razupiranje izvrši čeličnom četvoropojasnom rešetkom od starog krana koji je dlimično nadograđen (slika 17.). Na istoj slici, u levom uglu je detalj razupiranja o postojeći stub konstrukcije, dok se u desnom uglu nazire deo neuklonjene peščane berme.

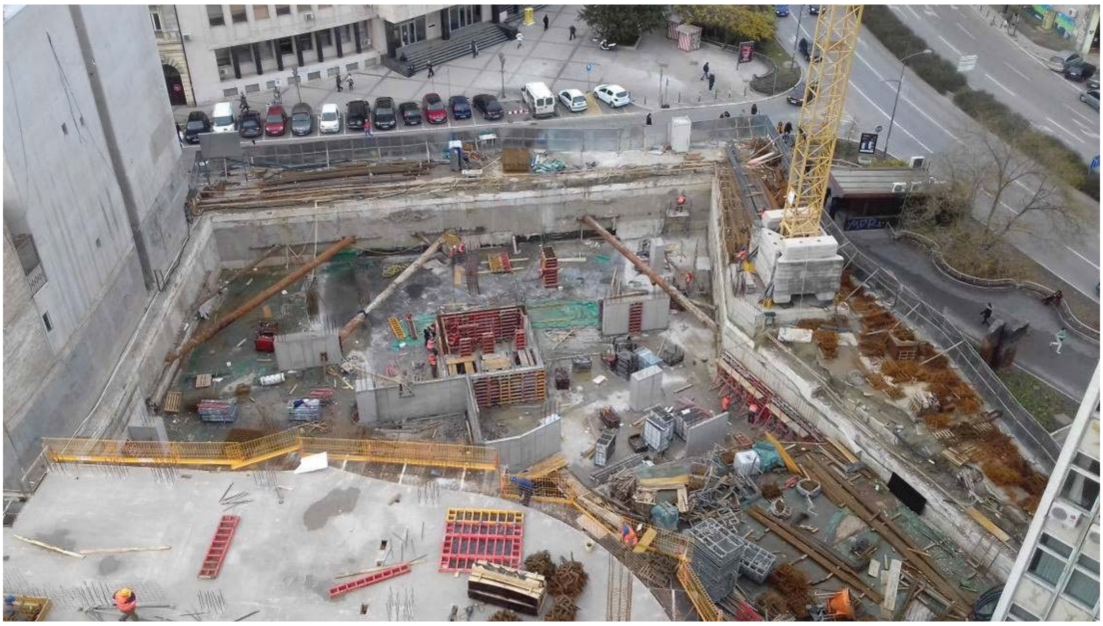
In phase III, due to the great width of the foundation pit, it was decided to brace it with steel four-lane grid from the old crane, which was gradually upgraded (Fig. 17). In the same picture, in the left corner is the detail of the strut between the column and the wall, while in the right-hand corner there is a part of the un removed sand berm.

Slika 17. Razupiranje četvoropojasnom čeličnom rešetkom od 36 m u III fazi (08.07.2017)
Figure 17. Strutting with 36m long steel grid at phase-III (08.07.2017)

U donjem levom delu slike 17. vide se nadzemni deo bunarske konstrukcije i potisni cevovod od alkatin cevi ∅100 mm. Iskop u temeljnoj jami rađen je sve vreme manjim bagerima, smeštenim na dnu temeljne jame, koji su materijal prebacivali do kašike bagera na višem nivou, koji je potom pesak tovario u kamione koji su išli na izlaz u ulici Narodnih heroja.

The above-ground part of the well structure and the alkaline ∅100mm pipeline appear in the lower left part of Fig. 17. The excavation in the foundation pit was carried out all the time with smaller excavators that were at the bottom of the underground pit. The material was transported by the bucket of the excavator on a higher level where the sand was loaded into trucks and moved to the exit in the Narodnih heroja street.

Slika 18. Završni zemljani radovi i razupiranje u IV I V fazi (03.11.2017)
Figure 18. Final earthworks and strutting at phase IV-V (03.11.2017)



Slika 19. Radovi na nivou 2 u IV i V fazi (12.12.2017)
Figure 19. Works at level-2 at phase IV-V (12.12.2017)

Radovi na iskopu ispod radnog platoa, na koti 75.0 m, započeti su krajem aprila 2017. godine, a poslednja kašika bagera utovarena je zemljom iz iskopa početkom novembra 2017. Praktično, u periodu od maja do oktobra 2017. godine, završen je iskop oko 37.000 m^3 zemlje, a istovremeno je završeno i razupiranje dijafragme međuspratnim pločama, tako da su uklonjeni svi privremeni čelični razupirači.

5 ZAKLJUČAK

U radu je prikazan način proračuna i izvođenja zaštitne konstrukcije za duboku temeljnu jamu i susedne objekte, u složenim urbanim i geotehničkim uslovima, korišćenjem relativno jednostavne tehnike, koja podrazumeva paralelnu gradnju i razupiranje o izvedene delove objekta. Ovim postupkom, izbegnuta je potreba da se izradi veći broj šipova velikog prečnika, koju zahteva „Top-Down" metoda, odnosno izrada velikog broja prednapregnutih sidara.

Osim dijafragme, sve podzemne radove na iskopu i razupiranju, izvela je građevinska firma – koja je ujedno bila i investitor, i to isključivo korišćenjem standardne građevinske opreme i radne snage. Ovakav postupak potpuno je odgovarao izvođaču radova – i u pogledu vremena i u pogledu dinamike korišćenja raspoloživih sopstvenih materijalnih i ljudskih resursa.

Radovi na izgradnji podzemnog dela objekta protekli su bez problema, a izmerena pomeranja dijafragme u granicama su računskih pomeranja. Na starim susednim objektima, u toku iskopa, pojavila su se manja oštećenja u vidu prslina u zidovima i tavanicama, što je prouzrokovano neizbežnim sleganjem tla u zaleđu dijafragme. Međutim, nivo oštećenja susednih objekata bio je u očekivanom opsegu, pa će nastala oštećenja po završetku radova biti otklonjena o trošku izvođača.

Work on the excavation under the working plateau at 75.0m began at the end of April 2017, and the last bucket of excavator was loaded with excavated soil in early November 2017. Basically, over the period of time from May to October 2017, the excavation of about 37,000m^3 of soil was carried out and simultaneously the struts on the diaphragm wall were replaced with the reinforced slabs of the construction.

5 CONCLUSIONS

This paper presents the method of calculation and execution of a protective structure for a deep foundation pit and adjacent objects, in complex urban and geotechnical conditions, using a relatively simple technique, which implies parallel construction and bracings from the constructed parts of the object. This procedure avoided the need of number of large-diameter piles by the "Top-Down" method, or a large number of prestressed anchors.

Instead of the diaphragm wall, all underground excavation and bracing works were carried out by a construction company, which was also an investor, by using standard construction equipment and labour. This procedure was fully in line with the contractor, both in terms of time and dynamics of using available own material and human resources.

Works on the construction of the underground part of the building have passed without any problems, and the measured diaphragm wall movements are within the limits of calculation shifts. In the old neighbouring buildings, minor damages in the form of cracks in the walls and ceilings occurred during the excavation, which was caused by the inevitable settlement of the soil behind the diaphragm wall. However, the level of damage to adjacent objects was in the expected range, and after the building will be finished, all damage will be removed at the expense of the contractor.

6 LITERATURA
REFERENCES

[1] GeoStudio 2012, GEO-SLOPE International Ltd, Calgary, Alberta, Canada.

[2] Stress-Deformation Modeling with SIGMA/W© 2012, GEO-SLOPE International Ltd, Calgary, Alberta, Canada.

[3] Seepage Modeling with SEEP/W© 2012, GEO-SLOPE International Ltd, Calgary, Alberta, Canada.

[4] Numerička simulacija AB dijafragme i čeličnih razupirača, softverom GeoStudio, za stambeno-poslovni objekat „Pupinova palata" u Novom Sadu, GeoEXPERT DOO, Subotica, 2016.

[5] Proračun AB dijafragme i čeličnih razupirača, softverom TOWER, za stambeno-poslovni objekat „Pupinova palata" u Novom Sadu, FPP STUDIO DOO, Beograd, 2016.

[6] Elaborat o geotehničkim uslovima izgradnje, za stambeno-poslovni objekat „Pupinova palata" u Novom Sadu, DOO Geomehanika, Novi Sad, 2015.

REZIME

PRIMER ZAŠTITE DUBOKE TEMELJNE JAME I SUSEDNIH OBJEKATA U SLOŽENIM URBANIM I GEOTEHNIČKIM USLOVIMA

Željko BAJIĆ
Petar SANTRAČ

Prilikom izgradnje novih objekata s više podzemnih etaža, u urbanim sredinama, poseban problem predstavlja zaštita dubokih temeljnih jama i susednih objekata, posebno kada je reč o starim, plitko fundiranim i zidanim objektima koji nemaju adekvatna ukrućenja vertikalnim i horizontalnim serklažima. Ako se na postojeći problem nadovežu i složeni geotehnički uslovi, u vidu visokog nivoa podzemne vode, velike vodopropusnosti sredine i blizine reke koja je u hidrauličkoj vezi s lokacijom, problem izgradnje podzemnog dela objekta po složenosti može višestruko prevazići složenost izgradnje nadzemnog dela.

U ovom radu prikazan je upravo jedan vrlo složen problem zaštite duboke temeljne jame za trospratnu podzemnu garažu stambeno-poslovnog objekta „Pupinova palata” u Novom Sadu, na Bulevaru Mihajla Pupina, ukupne površine oko 43.000 m^2, i vrlo razuđene spratnosti. Na građevinskoj liniji budućeg objekta bili su višespratni, stari, plitko temeljeni i zidani objekti. Nivo podzemne vode na lokaciji bio je relativno visok, u direktnoj hidrauličkoj vezi s rekom Dunav – udaljenoj oko 800 m - i velikim doticajem podzemne vode kroz vodopropusne i lako pokretljive sedimente dunavskog peska.

Analizirano je više idejnih varijanti zaštitne konstrukcije, razmatrane su prednosti i nedostaci rešenja, a detaljno je opisano usvojeno rešenje zaštite temeljne jame i susednih objekata. U trenutku pisanja ovog članka, podzemni deo objekta uspešno je završen, a rezultati izmerenih pomeranja zaštitne konstrukcije bili su u granicama računskih.

Ključne reči: duboka temeljna jama, armiranobetonska dijafragma, numerička simulacija

SUMMARY

EXAMPLE OF PROTECTION OF DEEP FOUNDATION PIT IN COMPLEX URBAN AND GEOTECHNICAL CONDITIONS

Zeljko BAJIC
Petar SANTRAC

In the construction of new buildings with several underground floors, in urban areas, a special problem is the protection of deep foundation pit and adjacent structures, especially when it comes to old, shallow and masonry structures that lack adequate stiffness. If complex geotechnical conditions are met, in the form of high groundwater level, high water permeability of the soil, and proximity of the river, which is in a hydraulic connection with the site, the complexity of building underground part can be overcome in many ways over the complexity of the above-ground construction. This paper presents a very complex problem of protecting the deep foundation pit for the three-storey underground garage of the residential and business building "Pupin's Palace" in Novi Sad, in Boulevard Mihajla Pupina, with total area of about 43,000m^2, and very diluted floors. On the construction line of the future facility were multi-storey, old, shallow-grounded and masonry buildings. The groundwater level at the site was relatively high, in a direct hydraulic connection with the Danube river, which is about 800m away, and a large subterranean water supply through the permeably and easily mobile sediments of the Danube sand. Several conceptual designs of the protective structure were analyzed, the advantages and disadvantages of the solution were considered, and the adopted solution for protection of the foundation pit and adjacent objects was described in detail. At the moment of writing this article, the underground part of the building was successfully completed, and the results of the measured shifting of the protective structure were within the estimated limits.

Key words: deep foundation pit, reinforced concrete diaphragm wall, numerical simulation

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Professor Dr. **VLADIMIR SIMONCE**, B.Sc. Eng. civ.
(1934-2016)

Profesor dr Vladimir Simonče zauvek nas je napustio 29.11.2016. godine, u Skoplju. Vladimir Simonče rođen je u Ohridu 18.05.1934. godine. Osnovnu školu i gimnaziju završio je u Ohridu, a na Građevinskom fakultetu u Skoplju diplomirao je 1960. godine. Nakon toga, izabran je za asistenta na Katedri za teoriju konstrukcija, tehničke mehanike i otpornost materijala.

Godine 1965. upisao je poslediplomske studije na Institutu za zemljotresno inženjerstvo i urbanističko planiranje, pri Univerzitetu u Skoplju, u grupi za zemljotresno inženjerstvo. Pomenute studije završio je u junu 1967. godine, odbranom magistarske teze „Dinamika rotaciono simetričnih ljuski".

U martu 1968. godine, izabran je za docenta na Katedri za teoriju konstrukcija. Od oktobra 1969. godine do oktobra 1971. godine bio je prodekan za nastavu na Arhitektonsko-građevinskom fakultetu u Skoplju. Juna 1973. godine izabran je za vanrednog profesora na Katedri za teoriju konstrukcija. Godine 1977. odbranio je doktorsku disertaciju pod naslovom „Statička analiza naklonjenih cilindričnih ljuski kod višelučnih brana".

Za redovnog profesora na grupi predmeta iz teorije konstrukcije izabran je marta 1979. godine. Bio je dekan građevinskog fakulteta u Skoplju u dva mandata – od juna 1987. do juna 1991. godine.

Od oktobra 1968. godine do juna 1969. godine boravi na univerzitetu u Kaliforniji (Los Anđeles, SAD). Na tom univerzitetu, od 1. januara do 31. marta, kao gostujući profesor, drži nastavu iz predmeta

Professor Vladimir Simonče left us forever on November 29, 2016 in Skopje. Vladimir Simonče was born in Ohrid on May 18, 1934. He finished elementary school and grammar school in Ohrid, to graduate from the Faculty of Civil Engineering in Skopje in 1960. After that, he was chosen as assistant at the Chair of Theory of Construction, Structural Mechanics and Material Resistance.

In 1965 he enrolled postgraduate studies at the Institute for Earthquake Engineering and Urban Planning, at the University of Skopje, in the earthquake engineering group. He finished the postgraduate study in June 1967, defending the master thesis "Dynamics of Rotationally Symmetric Shells".

In March 1968, he was elected Assistant Professor at the Department of Theory of Structures. From October 1969 until October 1971, he served as a vice-dean for teaching at the Faculty of Architecture and Engineering in Skopje. In June 1973, he was elected Associate Professor at the Department of Theory of Structures. In 1977, he defended his doctoral thesis titled "Static analysis of inclined cylindrical shells in multi-arch dams".

He was elected a full-time professor in the group of subjects from theory of structures in March 1979. He was dean Faculty of Civil Engineering in Skopje in two terms - from June 1987 to June 1991.

From October 1968 to June 1969 he resides at the University of California (Los Angeles, USA). In this university, from January 1 to March 31, as a visiting

ENGINEERING 165A (Statika linijskih nosača). Od septembra 1974. do avgusta 1975. godine boravi na univerzitetu u Svonziju (Vels, Velika Britanija) i dva meseca u Londonu (Imperial College).

Bio je član u nekoliko republičkih i gradskih komisija iz oblasti nauke i obrazovanja, kao i član više profesionalnih udruženja (Društvo građevinskih konstruktera Makedonije, Društvo za mehaniku Makedonije, GAMM – Društvo za primenjenu matematiku i mehaniku Nemačke). Od 1991. godine, član je Američkog udruženja građevinskih inženjera (*American Society of Civil Engineers*), a od 1971. godine i Svetskog udruženja za mostove i visoke zgrade (*International Association for Bridge and Structural Engineering*), sa sedištem u Cirihu.

Kao nastavnik Građevinskog fakulteta u Skoplju, drži nastavu iz predmeta: teorija konstrukcija I i II, primena računara u građevinarstvu, numeričke metode u građevinarstvu, teorija površinskih nosača i dinamika i stabilnost konstrukcija. Na poslediplomskim studijama drži nastavu iz predmeta: metod konačnih elemenata, nelinearna analiza primenom MKE, betonske ljuske. Za osnovni predmet teorija konstrukcija II napisao je udžbenik „Matrična analiza konstrukcija”.

Osamdesetih godina prošlog veka, prof. Vladimir Simonče držao je predavanja na Tehničkom fakultetu Univerziteta u Prištini iz oblasti teorije konstrukcija i vodio nekoliko diplomskih radova.

Autor je više naučnih radova, uglavnom iz oblasti statičke i dinamičke analize objekata visokogradnje i inženjerskih konstrukcija. Rad „Trodimenzionalne analize višelučne brane Prilep” publikovan je na simpozijumu u Svonziju 1975. godine. Bio je rukovodilac i učesnik brojnih naučnoistraživačkih projekata.

Takođe, bio je mentor i član u komisijama za više doktorskih disertacija i magistarskih radova.

Na profesionalnom planu, radio je na mnogim projektima (studije i projekti betonskih brana, statička i seizmička analiza objekata visokogradnje i inženjerskih objekata, izrada aplikativnog softvera iz oblasti građevinarstva). Njegov poslednji značajni projekat jeste Glavni projekat tanke lučne brane „Sv. Petka” na reci Treski, koja je puštena u rad 2012. godine.

Posebno treba napomenuti doprinos profesora Vladimira Simončeta u reformi obrazovnog procesa Građevinskog fakulteta u Skoplju osamdesetih godina XX veka, uvođenjem računara u nastavi i u građevinskoj operativi.

Aktivno je sarađivao s časopisom „Građevinski materijali i konstrukcije”, a poslednji, veoma zapažen rad – Lučna brana „Sv. Petka” u Republici Makedoniji (*Arch Dam „Sv. Petka” in R. Macedonia*) – objavio je u broju 3 za 2012. godinu (str. 37–54).

Prevremeni odlazak profesora Vladimira Simončeta najviše je pogodio njegovu porodicu – suprugu, kćerku i unuku. Neka počiva u miru. Večna mu slava i hvala mu.

professor, he held classes from the subject of ENGINEERING 165A (Static of Linear Bearings). From September 1974 to August 1975 he resides at the University of Swansea (Wales, UK) and two months in London (Imperial College).

He was member of several republic and city commissions in the field of science and education, as well as member of several professional associations (Association of Macedonian Civil Engineers, Association of Macedonian Mechanics, GAMM – German Association for Applied Mathematics and Mechanics). Since 1991, he has been a member of the American Society of Civil Engineers, and since 1971 also of the International Association for Bridge and Structural Engineering, based in Zurich.

As a teacher at the Faculty of Civil Engineering in Skopje, he held lectures from the following subjects: Theory of structures 1 and 2, Application of computers in civil engineering, Numerical methods in civil engineering, Theory of surface supports and structural dynamics and stability. At the postgraduate studies, he held a course from the following subjects: Finite element method, Nonlinear analysis using FEM, Concrete shells. For the basic subject Theory of structures 2, he published the textbook "Matrix Analysis of Structures".

During the 1980's Professor Vladimir Simonče delivered lectures at the Technical Faculty of the University of Pristina in the field of theory of structures and was a mentor of several graduate theses.

He was the author of several scientific papers, mainly in the field of static and dynamic analysis of building constructions and engineering structures. The paper "Three-dimensional Analysis of the Prilep Multi Arch Dam" was published at the Symposium in Swansea in 1975. He was a leader and participant in a number of scientific research projects.

He was also a mentor and member of committees for several doctoral dissertations and master theses.

On a professional level, he worked on many projects (studies and projects of concrete dams, static and seismic analysis of building constructions and engineering objects, development of applied software in the field of civil engineering). His last significant project was the Main Project of the thin arch dam "Sv. Petka" on the Treska River, which was let in operation in 2012.

The contribution of Professor Vladimir Simonče in the reform of the educational process of the Faculty of Civil Engineering in Skopje was particularly important in the 1980's when computers were introduced in the teaching process and the engineering operations.

He actively cooperated with the journal "Building Materials and Structures". His last, highly acclaimed paper *Arch Dam "Sv. Petka" in the Republic of Macedonia* was published in the issue 3/2012 (pages 37-54) in the same magazine.

The early departure of Professor Vladimir Simonče most affected his family - his wife, daughter and grandchild. May he rest in peace. Eternal glory and thanks to him.

Stanislav Milovanović
Grozde Aleksovski

IN MEMORIAM

Profesor Dr.- Ing. habil. **TOM ŠANC**, dipl.inž.građ.
Professor Dr.-Ing. habil. **TOM SCHANZ**, B.Sc. Eng. civ.
(1962-2017)

U Gelzenkirhenu, 12.10.2017. godine, iznenada, u 55. godini, preminuo je profesor Tom Šanc, šef Katedre za fundiranje, mehaniku tla i mehaniku stena Rur Univerziteta u Bohumu i član uređivačkog odbora časopisa „Građevinski materijali i konstrukcije”. Preranom smrću profesora Šanca, akademska zajednica ostala je bez izuzetne ličnosti, inspiratora i motivatora, posvećenog mentora i iskrenog prijatelja.

Profesor Šanc rođen je 24.05.1962. godine u Darmštatu (SR Nemačka). Studirao je građevinarstvo od 1982. do 1988. i geologiju od 1986. do 1988. godine, na Univerzitetu u Štutgartu. Nakon diplomiranja, radi pod mentorstvom prof. Gusmana, na razvoju metode kinematičkih elemenata na Institutu za mehaniku tla i fundiranje Univerziteta u Štutgartu. Na Institut za geotehniku Saveznog tehničkog instituta (ETH) u Cirihu odlazi 1989. godine i, pod mentorstvom prof. Langa, radi na istraživanju geomehaničkog ponašanja recikliranog betona. Doktorsku disertaciju pod naslovom „Istraživanje mehaničkog ponašanja granularnih mešavina na primeru recikliranog betona” *(Untersuchungen zum mechanischen Verhalten granularer Gemische am Beispiel von Beton-Recycling-Material)* odbranio je 1994. godine, na Saveznom tehničkom institutu u Cirihu. Nakon doktorata, vraća se na Institut za geotehniku Univerziteta u Štutgartu, gde prvo radi kao saradnik prof. Smolčika, a zatim prof. Vermera.

Zajedno s profesorom Vermerom, bavi se istraživa-

In October 12, 2017, Professor Tom Schanz, head of the Chair of Foundation Engineering, Soil and Rock Mechanics of the Ruhr University in Bochum and a member of the editorial board of the journal "Building Materials and Structures" suddenly passed away at the age of 55 in Gelsenkirchen. With the early death of Professor Schanz, the academic community has lost an exceptional personality, inspirer and motivator, dedicated mentor and sincere friend.

Professor Schanz was born on May 24, 1962 in Darmstadt (Germany). He studied civil engineering from 1982 to 1988 and geology from 1986 to 1988 at the University of Stuttgart. After graduation, he worked as a research assistant of Professor Gussmann, developing the method of kinematic elements at the Institute of Soil Mechanics and Foundation Engineering at the University of Stuttgart. In 1989 he came to the Institute of Geotechnics of the Federal Technical Institute (ETH) in Zurich and, as a research assistant of Professor Lang, worked on tne research of the geomechanical behaviour of recycled concrete. The doctoral thesis titled "Investigation of mechanical behaviour of granular mixtures in the example of recycled concrete" (*Untersuchungen zum mechanischen Verhalten granularer Gemische am Beispiel von Beton-Recycling-Material*) he defended in 1994 at the Federal Technical Institute in Zurich. After doctorate, he returned to the Institute of Geotechnics at the University of Stuttgart, where he first njem konstitutivnog modeliranja granularnih materijala, prvenstveno peska i peskovitih materijala. U to vreme, *Cam Clay* model tla bio je široko prihvaćen konstitutivni model za gline, dok je odgovarajući model za peskove nedostajao. U tom periodu, profesor Šanc ostvaruje veoma značajne naučne doprinose. Na osnovu ranijih istraživanja prof. Vermera (*Vermeer PA: A double hardening model for sand, Géotechnique 28/4, 1978*), formuliše konstitutivni model sa ojačanjem i poboljšanom kapom – Hardening Soil Model (HS). Veze između parametara HS modela definisane su na osnovu serija izvedenih edometarskih i triaksijalnih eksperimenata, kao i analizom rezultata iz literature za različite tipove peskova. Rezultati ovog istraživanja objavljeni su u prestižnom časopisu „Géotechnique" (*Schanz T, Vermeer PA: Angles of friction and dilatancy of sand, Géotechnique 46/1, 1996; Schanz T, Vermeer PA: On the stiffness of sands, Géotechnique 48, 1998)*, čime je potvrđen izuzetan kvalitet ostvarenih rezultata. Kasnije su drugi autori implementirali HS model u PLAXIS, softverski paket za za numeričku geotehničku analizu metodom konačnih elemenata. HS Model je danas u upotrebi širom sveta.

Nakon habilitacije na Univerzitetu u Štutgartu 1998. godine, s tezom „Modeliranje mehaničkog ponašanja frikcionih materijala" *(Zur Modellierung des mechanischen Verhaltens von Reibungsmaterialien)*, Tom Šanc postaje redovni profesor na BAUHAUS Univerzitetu u Vajmaru. U to vreme, sa 37 godina, bio je najmlađi redovni profesor geotehnike u Nemačkoj. Potom, započinje rad na istraživanju delimično zasićenog tla i razvoj eksperimentalnih metoda ispitivanja hidrauličkog ponašanja takvog tla. Takođe, započinje istraživačke projekte na temu hidromehaničkog ponašanja veoma zbijenih glina. Svoju istraživačku grupu u Vajmaru povezuje sa istraživačima širom sveta. U više istraživačkih projekata, bio je kodirektor s Radomirom Folićem na NATO projektu – *Science for Peace and Security Programme NATO Advanced Research Workshop 983188 – Coupled site and soil – structure interaction effects*. Ovaj projekat rezultirao je pomenutom radionicom, održanom u planinskom području Borovec (Bugarska), od 30.08. do 3.09. 2008. godine. Pored zbornika radova, u obliku dužeg apstrakta od po dve strane, odabrani radovi objavljeni su u knjizi: *Coupled Site and Soil-Structure Interaction Effects with Application to Seismic Risk Mitigation, Springer Sciences+Business Media* (ISBN 987-90-481-2709-2). Radovi su uvršteni na *Web of Sciences*.

U 2009. godini, Tom Šanc prelazi na Rur Univerzitet u Bohumu i postaje šef Katedre za fundiranje, mehaniku tla i mehaniku stena. Profesor Šanc uspešno povezuje svoje istraživačke grupe iz Vajmara i Bohuma, kao i brojne nove studente i stipendiste iz inostranstva (npr. Iran, Irak, Kina, Vijetnam, Sirija) i tako formira međunarodno prepoznatljivu grupu istraživača u oblasti geotehnike, pokrivajući širok spektar oblasti istraživanja. Na Rur Univerzitetu u Bohumu, profesor Šanc imao je ključnu ulogu u naučnoistraživačkom projektu SFB 837 – *Interactions Modelling in Mechanized Tunneling*, gde je rukovodio potprojektima na teme adaptivnog konstitutivnog modeliranja i numeričkih metoda za analize parametara geotehničkih modela.

Profesor Šanc bio je istinski naučnik, sa obaveznim pitanjem: *Zašto su rezultati takvi kakvi jesu?* Njegov

worked as associate to Professor Smolčik, and then Professor Vermeer.

Together with Professor Vermeer, he was researching the constitutive modelling of granular materials, primarily sand and sandy materials. At the time, the *Cam Clay* soil model was a widely accepted constitutive model for clays, while an adequate model for sands was missing. In that period, Professor Schanz made significant scientific contributions. Based on the previous research of Professor Vermeer (*Vermeer PA: A double hardening model for sand, Géotechnique 28/4, 1978*) he formulated a hardening soil (HS) model with improved cap. The relations between parameters of the HS model were defined on the basis of series of edometric and triaxial experiments, as well as based on analysis of results from the literature for different types of sand. The results of this research were published in the prestigious journal Géotechnique (Schanz T, Vermeer PA: Angles of friction and dilatancy of sand, Géotechnique 46/1, 1996; Schanz T, Vermeer PA: On the stiffness of sands, Géotechnique 48, 1998), which confirmed the exceptional quality of the results achieved. HS model was later implemented by other authors in the PLAXIS software for numerical geotechnical analysis using FEM and today it is a worldwide used constitutive model.

After habilitation at the University of Stuttgart in 1998 with the thesis "Modelling the Mechanical Behaviour of Friction Materials" (*Zur Modellierung des mechanischen Verhaltens von Reibungsmaterialien*), Tom Schanz became a professor at BAUHAUS University in Weimar in 1999. At that time, at age 37, he was the youngest professor of geotechnics in Germany. After that, he started to research the unsaturated soil mechanics and develop experimental methods for testing the hydraulic behaviour of such soil. He also set up research projects on the subject of hydromechanical behaviour of highly compact clays. He linked his Weimar research group with researchers around the world. He co-ordinated several research projects with Radomir Folic on the NATO project: *Science for Peace and Security Programme NATO Advanced Research Workshop 983188 – Coupled site and soil – structure interaction effects*. This project resulted in the above mentioned workshop, held in the mountainous region of Borovets (Bulgaria), from September 30 to October 3, 2008. In addition to proceedings in the form of a longer abstract on two pages each, the selected works were published in the book: *Coupled Site and Soil-Structure Interaction Effects with Application to Seismic Risk Mitigation, Springer Sciences + Business Media* (ISBN 987-90-481-2709- 2). Papers were listed in the *Web of Sciences*.

In 2009, Tom Schanz came to the Ruhr University in Bochum and became a head of the Chair of Foundation Engineering, Soil and Rock Mechanics. Professor Schanz successfully connected his research groups from Weimar and Bochum, as well as numerous new students and scholars from abroad (Iran, Iraq, China, Vietnam, Syria), thus forming an internationally recognizable group of researchers in the field of geotechnics, covering a wide range of research areas. At Ruhr University in Bochum, Professor Schanz played a key role in the SFB 837 scientific-research project *Interaction Modeling in Mechanized Tunneling*, where he managed subprojects on the topics of adaptive opus čini preko 200 naučnih publikacija u najistaknutijim naučnim časopisima i s međunarodnih konferencija. Prof. Šanc održao je mnogobrojna predavanja iz oblasti kojima se bavio, a jedno od poslednjih bilo je predavanje članovima Srpskog društva za mehaniku tla i geotehničko inženjerstvo, na Građevinskom fakultetu Univerziteta u Beogradu, septembra 2017. godine. Bio je recenzent i član uređivačkih odbora značajnih naučnih časopisa. Pored izuzetnog naučnog doprinosa na polju geotehnike, profesor Šanc ostaće upamćen i kao posvećen mentor, koji je nesebično i snažno podržavao naučnoistraživački rad na svojoj katedri. Motivisanjem, rukovođenjem i inspirisanjem, posvećenošću i entuzijazmom, stvaranjem otvorene, dobre atmosfere, pozivanjem i okupljanjem uspešnih naučnika iz drugih istraživačkih grupa, kao i stvaranjem mogućnosti svojim studentima i saradnicima da posete druge iskusne naučnike i institute, na svojoj katedri na Rur Univerzitetu u Bohumu stvorio je izuzetan ambijent za razvoj mladih naučnika. Uvek otvoren za multidisciplinarnu saradnju i nove ideje, kombinovanjem različitih istraživačkih metoda, tragao je za naučnom istinom. Istovremeno, vodio je računa o stalnom napredovanju svojih studenata i saradnika, kojima je bio podrška u teškim situacijama, često i izvan poslovnih okvira. Ovakav pristup rezultovao je stvaranjem široke međunarodne mreže istraživača i velikim brojem naučnih publikacija visokog kvaliteta. Njegova kreativnost, akademska misao i nesebična podrška nedostajaće svima koji su ga poznavali i imali tu privilegiju da s njim sarađuju.

Prevremeni odlazak profesora Šanca najviše je pogodio njegovu porodicu – suprugu i troje dece. Neka počiva u miru. Večna mu slava i hvala mu!

constitutive modeling and numerical methods for model parameter identification.

Professor Schanz was a true scientist, with the compulsory question: *Why are the results the way they are?* His work comprised over 200 scientific publications in the most prominent scientific journals and international conferences. Professor Schanz delivered many lectures in his scientific fields, and one of the last was a lecture delivered to members of Society for Soil Mechanics and Geotechnical Engineering of Serbia at the Faculty of Civil Engineering, University of Belgrade, in September 2017. He was a reviewer and member of editorial boards of prominent scientific journals. In addition to the exceptional scientific contribution in the field of geotechnics, Professor Schanz will be remembered as a dedicated mentor, who unselfishly and strongly supported the research work on his Chair. By motivating, managing and inspiring, dedication and enthusiasm, creating an open, good atmosphere, inviting and gathering successful scientists from other research groups, as well as creating opportunities for students and associates to visit other experienced scientists and institutes, at his Chair at the Ruhr University in Bochum, Professor Schanz created a remarkable environment for the development of young scientists. Always open to multidisciplinary collaboration and new ideas, combining different research methods, he was searching for the scientific truth. At the same time, he cared for the constant progress of his students and associates and was supportive in difficult situations, often beyond the professional framework. This approach resulted in the creation of a wide international network of researchers and a large number of high-quality scientific publications. His creativity, academic mind and unselfish support are missed by everyone who knew him and had the privilege of cooperating with him.

The early departure of Professor Schanz most affected his family - his wife and three children. May he rest in peace. Eternal glory and thanks to him!

Miloš Marjanović
Radomir Folić

ТЕХНИЧАРСКА ДРУЖИНА

1868 2018

САВЕЗ ИНЖЕЊЕРА И ТЕХНИЧАРА СРБИЈЕ

POVODOM 150 GODINA SAVEZA INŽENJERA I TEHNIČARA SRBIJE

Koreni srpske tehničke civilizacije počinju još u srednjem veku u doba Nemanjića. Začeci inženjerstva su u rudarsko-metalurškim poduhvatima kao što je značajni rudnik Novo Brdo i građenju veličanstvenih sakralnih i drugih objekata.

Obnavljanjem srpske države posle viševekovne Otomanske vlasti i stvaranjem moderne države u 19. veku oživelo je i inženjerstvo u Srbiji. Inženjeri se tada pretežno školuju u Austrougarskom carstvu i u Francuskoj. Već 1868. godine 3. februara bila je osnovana „Tehničarska družina“ koja je preteča današnjeg Saveza inženjera i tehničara Srbije.

Inženjerski Savez je za svojih 150 godina prolazio kroz razne mene, ali je stalno bio aktivan i društveno prepoznatljiv. Mnogi značajni inženjeri i naučnici svih struka su bili i sada su aktivni članovi. Prvi predsednik je bio arhitekta i urbanista Emilijan Josimović, a istaknuti počasni član Nikola Tesla.

Vrlo značajan momenat u radu i afirmaciji Saveza je bila izgradnja zgrade Doma inženjera Srbije 1936. godine i novog Doma inženjera „Nikola Tesla“ 1967. godine. Sredstva za izgradnju domova su obezbeđivali inženjeri, privrednici i dobrotvori čime je inženjerska inteligencija iskazala značaj i volju za okupljanjem i delovanjem kroz formu udruženja i saveza kao izraz stručnog, naučnog i intelektualnog, te kritičkog angažovanja.

Savez danas ima preko četrdeset, što strukovnih, multidisciplinarnih, tematskih, gradskih i regionalnih članica. U njegovom sastavu je Razvojni centar, kao i Inženjerska akademija Srbije. Aktivnosti su raznorazne: okupljanje, debate, konferencije, izdavaštvo, saradnja sa drugim strukama i udruženjima, održavanje stručnih ispita, izložbe, rad sa studentima, srednjoškolcima, mladim istraživačima.

Članstvo Saveza broji više hiljada inženjera iz svih gradova i opština Srbije. Savez i njegove članice su nevladine organizacije, koje se samofinansiraju iz svojih aktivnosti i članarine.

Značaj i uloga Saveza u društvu su veliki i u Srbiji i u široj evropskoj i svetskoj inženjerskoj zajednici, što se očituje kroz vidove članstva u međunarodnim, srodnim, organizacijama, te u domaćem ambijentu kroz afirmaciju znanja i saradnju sa drugim udruženjima, državnim organima, privredom, školstvom i naročito po brojnosti i kvalitetu svojih članova.

UPUTSTVO AUTORIMA*

Prihvatanje radova i vrste priloga

U časopisu Materijli i konstrukcije štampaće se neobjavljeni radovi ili članci i konferencijska saopštenja sa određenim dopunama, iz oblasti građevinarstva I srodnih disciolina (geodezija I arhitektura). Vrste priloga autora i saradnika koji će se štampati su: originalni naučni radovi, prethodna saopštenja, pregledni radovi, stručni radovi, prikazi objekata i iskustava (studija slučaja), kao i diskusije povodom objavljenih radova.

Originalni naučni rad je primarni izvor naučnih informacija i novih ideja i saznanja kao rezultat izvornih istraživanja uz primenu adekvatnih naučnih metoda. Dobijeni rezultati se izlažu sažeto, ali tako da poznavalac problema može proceniti rezultate eksperimentalnih ili teorijsko numeričkih analiza, tako da se istraživanje može ponoviti i pri tome dobiti iste ili rezultate u okvirima dopuštenih odstupanja, kako se to u radu navodi.

Prethodno saopštenje sadrži prva kratka obaveštenja o rezultatima istraživanja ali bez podrobnih objašnjenja, tj. kraće je od originalnog naučnog rada.

Pregledni rad je naučni rad koji prikazuje stanje nauke u određenoj oblasti kao plod analize, kritike i komentara i zaključaka publikovanih radova o kojima se daju svi neophodni podaci pregledno i kritički uključujući i sopstvene radove. Navode se sve bibliografske jedinice korišćene u obradi tematike, kao i radovi koji mogu doprineti rezultatima daljih istraživanja. Ukoliko su bibliografski podaci metodski sistematizovani, ali ne i analizirani i raspravljeni, takvi pregledni radovi se klasifikuju kao stručni radovi.

Stručni rad predstavlja koristan prilog u kome se iznose poznate spoznaje koje doprinose širenju znanja i prilagođavanja rezultata izvornih istraživanja potrebama teorije i prakse.

Ostali prilozi su prikazi objekata, tj. njihove konstrukcije i iskustava-primeri u građenju i primeni različitih materijala (studije slučaja).

Da bi se ubrzao postupak prihvatanja radova za publikovanje, potrebno je da autori uvažavaju Uputstva za pripremu radova koja su navedena u daljem tekstu.

Uputstva za pripremu rukopisa

Rukopis otkucati jednostrano na listovima A-4 sa marginama od 31 mm (gore i dole) a 20 mm (levo i desno), u Wordu fontom Arial sa 12 pt. Potrebno je uz jednu kopiju svih delova rada i priloga, dostaviti i elektronsku verziju na navedene E-mail adrese, ili na CD-u. Autor je obavezan da čuva jednu kopiju rukopisa kod sebe.

Od broja 1/2010, prema odluci Upravnog odbora Društva i Redakcionog odbora, radovi sa pozitivnim recenzijama i prihvaćeni za štampu, publikovaće se na srpskom i engleskom jeziku, a za inostrane autore na engleskom (izuzev autora sa govornog područja srpskog I hrvatskog jezika).

Svaka stranica treba da bude numerisana, a optimalni obim članka na jednom jeziku, je oko 16 stranica (30000 slovnih mesta) uključujući slike, fotografije, tabele i popis literature. Za radove većeg obima potrebna je saglasnost Redakcionog odbora.

* U**putstvo autorima je modifikovano i treba ga, u pripremi radova, slediti.**

GUIDELINES TO AUTHORS

Acceptance and types of contributions

The Building Materials and Structures journal will publish unpublished papers, articles and conference reports with modifications in the field of Civil Engineering and similar areas (Geodesy and Architecture).The following types of contributions will be published: original scientific papers, preliminary reports, review papers, professional papers, objects describe / presentations and experiences (case studies), as well as discussions on published papers.

Original scientific paper is the primary source of scientific information and new ideas and insights as a result of original research using appropriate scientific methods. The achieved results are presented briefly, but in a way to enable proficient readers to assess the results of experimental or theoretical numerical analyses, so that the research can be repeated and yield with the same or results within the limits of tolerable deviations, as stated in the paper.

Preliminary report contains the first short notifications on the results of research but without detailed explanation, i.e. it is shorter than the original scientific paper.

Review paper is a scientific work that presents the state of science in a particular area as a result of analysis, review and comments, and conclusions of published papers, on which the necessary data are presented clearly and critically, including the own papers. Any reference units used in the analysis of the topic are indicated, as well as papers that may contribute to the results of further research. If the reference data are methodically systematized, but not analyzed and discussed, such review papers are classified as technical papers.

Technical paper is a useful contribution which outlines the known insights that contribute to the dissemination of knowledge and adaptation of the results of original research to the needs of theory and practice.

Other contributions are presentations of objects, i.e. their structures and experiences (examples) in the construction and application of various materials (case studies).

In order to speed up the acceptance of papers for publication, authors need to take into account the Instructions for the preparation of papers which can be found in the text below.

Instructions for writing manuscripts

The manuscript should be typed one-sided on A-4 sheets with margins of 31 mm (top and bottom) and 20 mm (left and right) in Word, font Arial 12 pt. The entire paper should be submitted also in electronic format to e-mail address provided here, or on CD. The author is obliged to keep one copy of the manuscript.

As of issue 1/2010, in line with the decision of the Management Board of the Society and the Board of Editors, papers with positive reviews, accepted for publication, will be published in Serbian and English, and in English for foreign authors (except for authors coming from the Serbian and Croatian speaking area).

Each page should be numbered, and the optimal length of the paper in one language is about 16 pages (30.000 characters) including pictures, images, tables and references. Larger scale works require the approval of the Board of Editors.

Naslov rada treba sa što manje reči (poželjno osam, a najviše do jedanaeset) da opiše sadržaj članka. U naslovu ne koristiti skraćenice ni formule. U radu se iza naslova daju ime i prezime autora, a titule i zvanja, kao i ime institucije u podnožnoj napomeni. Autor za kontakt daje telephon, adresu elektronske pošte I poštansku adresu.

Uz sažetak (rezime) od oko 150-250 na srpskom i engleskom jeziku daju se ključne reči (do sedam). To je jezgrovit prikaz celog članka i čitaocima omogućuje uvid u njegove bitne elemente.

Rukopis se deli na poglavlja i potpoglovlja uz numeraciju, po hijerarhiji, arapskim brojevima. Svaki rad ima uvod, sadržinu rada sa rezultatima, analizom i zaključcima. Na kraju rada se daje popis literature.

Kod svih dimenzionalnih veličina obavezna je primena međunarodnih SI mernih jedinica.

Formule i jednačine treba pisati pažljivo vodeći računa o indeksima i eksponentima. Autori uz izraze u tekstu definšu simbole redom kako se pojavljuju, ali se može dati i posebna lista simbola u prilogu.

Prilozi (tabele, grafikoni, sheme i fotografije) rade se u crno-beloj tehnici, u formatu koji obezbeđuje da pri smanjenju na razmere za štampu, po širini jedan do dva stupca (8 cm ili 16,5 cm), a po visini najviše 24,5 cm, ostanu jasni i čitljivi, tj. da veličine slova i brojeva budu najmanje 1,5 mm. Originalni crteži treba da budu kvalitetni i u potpunosti pripremljeni za presnimavanje. Mogu biti i dobre, oštre i kontrastne fotokopije. Koristiti fotogrfije, u crno-beloj tehnici, na kvalitetnoj hartiji sa oštrim konturama, koje omogućuju jasnu reprodukciju.

U popisu literature na kraju rada daju se samo oni radovi koji se pominju u tekstu. Citirane radove treba prikazati po abecednom redu prezimena prvog autora. Literaturu u tekstu označiti arapskim brojevima u uglastim zagradama, kako se navodi i u Popisu citirane literature, napr [1]. Svaki citat u tekstu mora se naći u Popisu citirane literature i obrnuto svaki podatak iz Popisa se mora citirati u tekstu.

U Popisu literature se navode prezime i inicijali imena autora, zatim potpuni naslov citiranog članka, iza toga sledi ime časopisa, godina izdavanja i početna i završna stranica (od - do). Za knjige iza naslova upisuje se ime urednika (ako ih ima), broj izdanja, prva i poslednja stranica poglavlja ili dela knjige, ime izdavača i mesto objavljivanja, ako je navedeno više gradova navodi se samo prvi po redu. Kada autor citirane podatke ne uzima iz izvornog rada, već ih je pronašao u drugom delu, uz citat se dodaje «citirano prema...».

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The title should describe the content of the paper using a few words (preferably eight, and up to eleven). Abbreviations and formulas should be omitted in the title. The name and surname of the author should be provided after the title of the paper, while authors' title and position, as well as affiliation in the footnote. The author should provide his/her phone number, e-mail address and mailing address.

The abstract (summary) of about 150-250 words in Serbian and English should be followed by key words (up to seven). This is a concise presentation of the entire article and provides the readers with insight into the essential elements of the paper.

The manuscript is divided into chapters and subchapters, which are hierarchically numbered with Arabic numerals. The paper consists of introduction and content with results, analysis and conclusions. The paper ends with the list of references. All dimensional units must be presented in international SI measurement units. The formulas and equations should be written carefully taking into account the indexes and exponents. Symbols in formulas should be defined in the order they appear, or alternatively, symbols may be explained in a specific list in the appendix. Illustrations (tables, charts, diagrams and photos) should be in black and white, in a format that enables them to remain clear and legible when downscaled for printing: one to two columns (8 cm or 16.5 cm) in height, and maximum of 24.5 cm high, i.e. the size of the letters and numbers should be at least 1.5 mm. Original drawings should be of high quality and fully prepared for copying. They also can be high-quality, sharp and contrasting photocopies. Photos should be in black and white, on quality paper with sharp contours, which enable clear reproduction.

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Authors are responsible for the content presented and must themselves provide any necessary consent for specific information and illustrations used in the work to be published.

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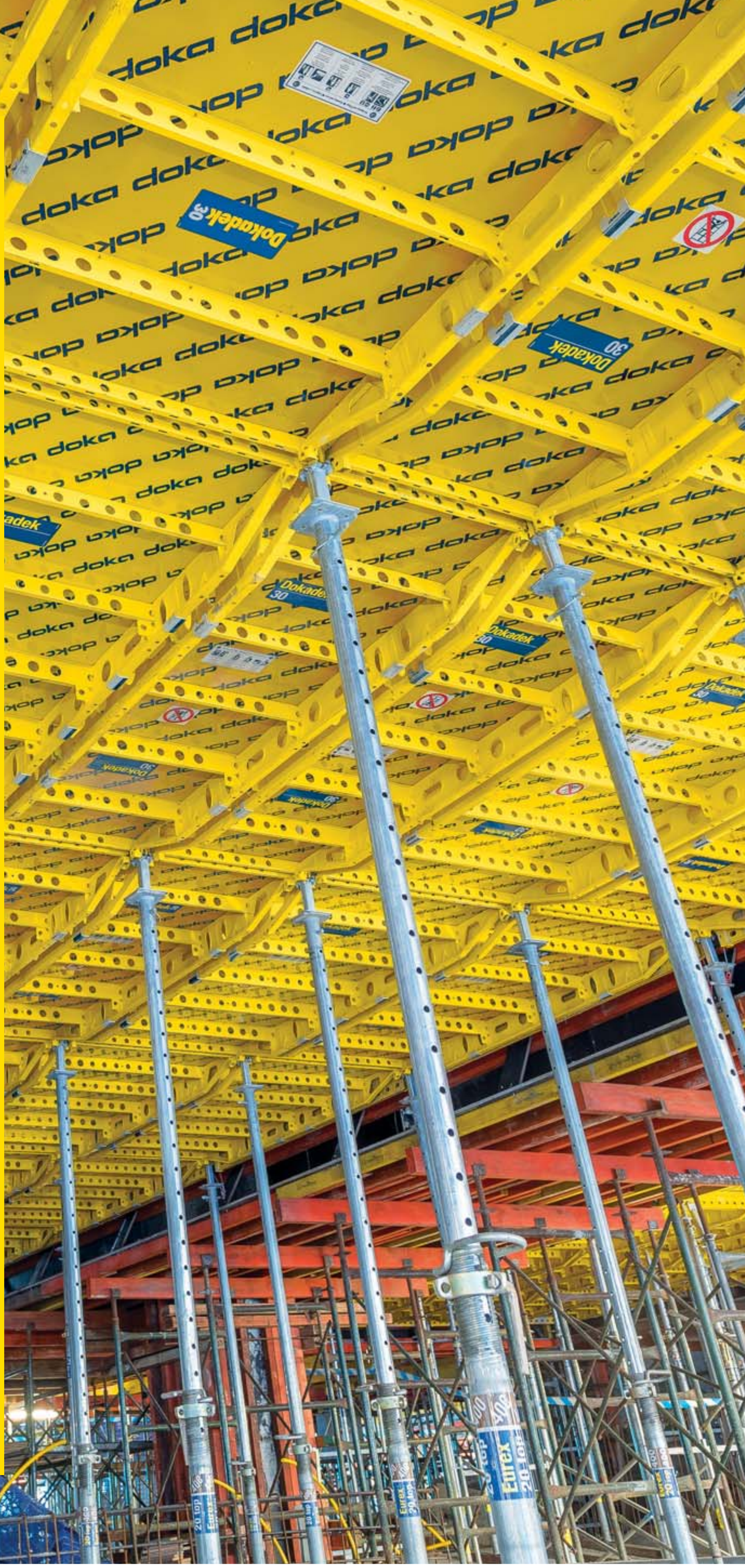

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Doka Serb je srpski ogranak austrijske kompanije **Doka GmbH**, jednog od svetskih lidera na polju inovacija, proizvodnje i distribucije oplatnih sistema za sve oblasti građevinarstva. Delatnost kompanije Doka Serb jeste isporuka oplatnih sistema i komponetni za primenu u visokogradnji i niskogradnji, pružanje usluga konsaltinga, izrade tehničkih planova i asistenicije na gradilištu.

Panelna oplata za ploče Dokadek 30 – Evolucija u sistemima oplate za ploče
Dokadek 30 je ručna oplata lake čelične konstrukcije, bez nosača sa plastificiranim ramovima, koji su prekriveni kompozitnim drveno-plastičnim panelom površine do 3 m^2.

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- Mali broj delova sistema i pregledna logistika uz samo dve veličine panela (2,44 x 1,22 m i 2,44 x 0,81 m)
- Dovoljan 2-člani tim za jednostavnu i brzu montažu elemenata sa tla bez merdevina i bez krana
- Sistemski određen položaj i broj podupirača i panela, unapred definisan redosled postupaka
- Prilagođavanje svim osnovama zahvaljujući optimalnom uklapanju sa Dokaflex-om
- Specijalni dizajn sprečava odizanje panela pod uticajem vetra
- Horizontalno premeštanje do 12 m^2 Dokadek 30 pomoću DekDrive

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INSTITUT IMS a.d.

Institut za ispitivanje materijala IMS, sa tradicijom od 1929. godine, predstavlja najstariju naučno-istraživačku instituciju u Srbiji. Osnovna ideja prilikom osnivanja bila je potreba za jedinstvenom institucijom koja bi se osim istraživanja bavila i kontrolom građevinske industrije.

Delatnost Instituta IMS obuhvata laboratorijska ispitivanja građevinskih materijala, sertifikaciju proizvoda, nadzor nad izvođenjem radova i ispitivanje različitih tipova konstrukcija, izradu projektne dokumentacije, kao i naučno - istraživački rad u svim oblastima građevinarstva.

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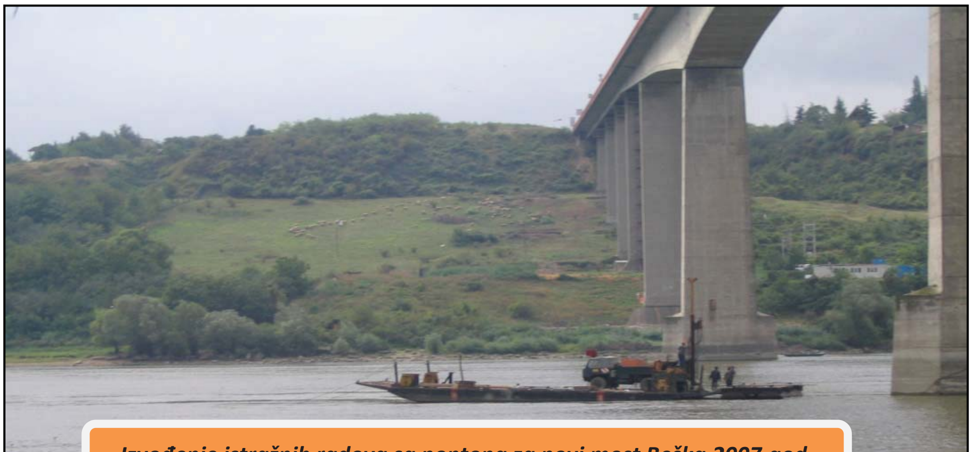


Izvođenje istražnih radova sa pontona za novi most Beška,2007.god.

Geotehnička istraživanja i ispitivanja – in situ

Od terenskih istražnih radova izdvajamo izvođenje istražnih bušotina (IB), standardnih penetracionih opita (SPT), statičkih penetracionih opita (CPT i CPTU), opita dilatometarskom sondom (DMT i SDMT), ispitivanja vodopropustljivosti tla različitim terenskim metodama (VDP), ugradnja pijezometara i dr.

Terenske metode ispitivanja šipova zauzimaju značajno mesto u našoj delatnosti, a na tržištu se izdvajamo kao lideri u toj oblasti u protekloj deceniji.

DLT-dinamičko ispitivanje šipova

Ispitivanje šipova

SLT metoda (Static load test) ispitivanje nosivosti šipova statičkim opterećenjem;

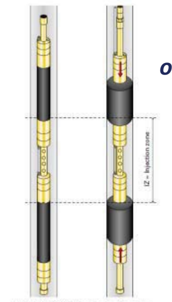
DLT metoda (Dynamic load test) ispitivanje nosivosti šipova dinamičkim opterećenjem;

PDA metoda (Pile driving analysis) omogućava praćenje i optimizaciju procesa pobijanja prefabrikovanih betonskih i čeličnih šipova u tlo;

PIT (SIT) metoda (Pile(Sonic) integrity testing) koristi se za ispitivanje integriteta izvedenih šipova (dužine, prekida, suženja ili proširenja).

CPT/CPTU opiti

Aktivno klizište



oprema za ispitivanje vodopropusnosti stena pod pritiskom do 10 bar-a metodom LIŽONA

Laboratorija za puteve i geotehniku

Laboratorija za puteve i geotehniku akreditovana je kod Akreditacionog tela Srbije – ATS prema SRPS ISO/IEC 17025:2006. U njoj se vrše ispitivanja tla (identifikaciono–klasifikaciona ispitivanja, fizičko–mehanička modelska ispitivanja), kamenog agregata i brašna, bitumena i bitumenskih emulzija, asfaltnih mešavina. U okviru laboratorijskih ispitivanja na terenu vrši se kontrola kvaliteta ugrađenog materijala i izvedenih radova (prethodna, tekuća, kontrolna ispitivanja i izvođenja opita in situ).

Projektovanje puteva i sanacija klizišta

U okviru projektovanja značajno mesto u radu zauzimaju geotehnička istraživanja terena i projekti sanacije klizišta - nestabilnih kosina useka i nasipa puteva i prirodno nestabilnih padina . Značajna su i projekovanja svih vrsta fundiranja specijalnih geotehničkih konstrukcija. Ističe se i iskustvo u oblasti putarstva, na projektovanju novih, rehabilitacija i rekonstrukcija postojećih puteva svih rangova sa pratećim objektima i dimenzionisanjem kolovoznih konstrukcija.

Nadzor

Naši inženjeri imaju veliko iskustvo u kontroli i proveri kvaliteta izvođenja svih vrsta radova, kontroli građevinske dokumentacije i praćenju radova u skladu sa njom, kao i rešavanju novonastalih situacija tokom izvođenja radova.

BUILDING TRUST

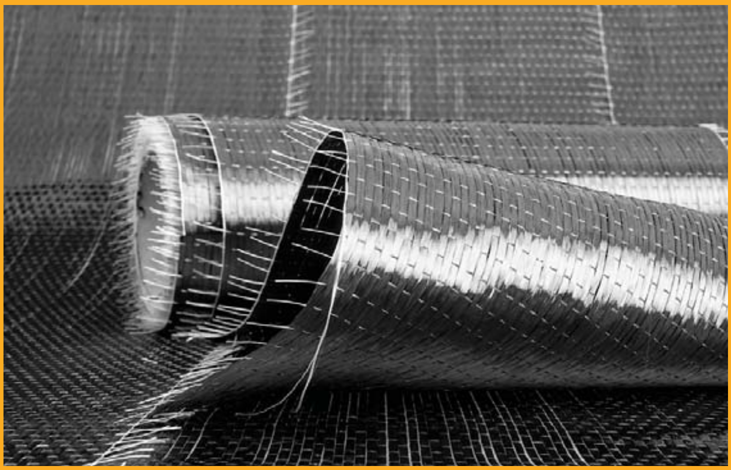
NAPREDNA SIKA REŠENJA U OBLASTI STRUKTURALNIH OJAČANJA

Kompanija Sika pruža trajnu dodatnu vrednost vlasnicima građevinskih objekata, njihovim konsultantima i izvođačima, kao i tehničku podršku tokom svih faza projekta, od ispitivanja uslova i razvoja inicijalnog koncepta ojačanja pa sve do uspešnog završetka i primopredaje projekta

SIKA – VAŠ PARTNER NA GRADILIŠTU

- Globalni lider na tržištu građevine i građevinske hemije
- Najbolja tehnička ekspertiza i praksa za sanaciju betona i strukturalna ojačanja
- Odlična reputacija kod vodećih izvođača i ugovarača posla

SIKA VREDNOSTI I INOVACIJE U GRAĐEVINI



- Integrisani proizvodi i sistemi visokih performansi koji mogu da povećaju i poboljšaju kapacitet, efikasnost, trajnost i estetiku zgrada i drugih objekata – u korist naših klijenata i boljeg održivog razvoja
- Sika mreža obučenih i iskusnih građevinskih stručnjaka

JEDINSTVENA SIKA REŠENJA U ZAHTEVNIM USLOVIMA

- Rešenja za gotovo sve uslove apliciranja
- Kontrolisano vreme rada, vreme sazrevanja i očvršćavanja za različite vremenske uslove
- Posebna rešenja završnih ojačanja za korišćenje kod betona slabije jačine i drugih podloga

POTVRĐENI SIKA SISTEMI I TEHNIKE APLICIRANJA

- Preko 40 godina iskustva u strukturalnim ojačanjima, sistemima i tehnikama
- Proizvodi i sistemi sa brojnim testovima i procenama kako internim tako i eksternim
- Najviši međunarodni standardi proizvodnje i kontrole kvaliteta

PUT INŽENJERING

Put inženjering d.o.o punih 25 godina radi kao specijalizovano preduzeće za izgradnju infrastrukture u niskogradnji i visokogradnji, kao i proizvodnjom kamenog agregata i betona. Preduzeće se bavi i transportom, uslugama građevinske mehanizacije i specijalne opreme.

Za spravljanje betona koristimo drobljeni krečnjački agregat sa našeg kamenoloma, deklarisanih frakcija, kontrolisane vlažnosti. Kompletan proces proizvodnje i kontrole kvaliteta vršimo prema važećim standardima.

Kao generalni izvođač radova, vršimo koordinaciju svih učesnika na projektu, planiranje, praćenje i nabavku materijala, kontrolu kvaliteta izvedenih radova, poštujući zadate vremenske rokove i finansijski okvir investitora.

Obradu armature vršimo brzo, stručno i kvalitetno, sa kompjuterskom preciznošću i dimenzijama po projektu.

Osnovi princip našeg poslovanja zasniva se na individualnom pristupu svakom klijentu i pronalaženje najoptimalnijeg rešenja za njegove transportne i logističke potrebe.

Koristeći inovativne tehnike i kvalitetan građevinski materijal iz sopstvenih resursa, spremni smo da odgovorimo na mnoge zahteve naših klijenata iz oblasti niskogradnje.

Naša kompanija u oblasti visokogradnje primenjuje sistem prefabrikovnih betonskih elemenata koji u odnosu na klasičnu gradnju ima brojne prednosti.

Usluge građevinskom mehanizacijom vršimo tehnički ispravnim mašinama, sa potrebnim sertifikatima kako za rukovaoce građevinskim mašinama tako i za same mašine.

Osnovna prednost prefabrikovane konstrukcije jeste brzina kojom konstrukcija može biti projektovana, proizvedena, transportovana i namontirana.

Prednapregnute šuplje ploče su konstruktivni elementi visokog kvaliteta, proizvedeni u fabrički kontrolisanim uslovima.

Raspolažemo opremom i mašinama za sve zemljane radove, kipere i dampere za rad u teškim terenskim uslovima, automiksere i pumpe za beton, autodizalice, podizne platforme.

Izvodimo hidrograđevinske radove u izgradnji kanalizacionih mreža za odvođenje atmosferskih, otpadnih i upotrebljenih voda, izvođenjem hidrograđevinskih radova u okviru regulacije rečnih tokova, kao i izvođenjem hidrotehničkih objekata.

Izrađujemo betonske "New Jersey profile" koji se u svetu koriste za preusmeravanje saobraćaja i zaštitu pešaka u toku izgradnje puta, kao i Betonblock sistem betonskih blokova.

Sakupljanje i privremeno skladištenje otpada vršimo našim specijalizovanim vozilima i deponujemo na našu lokaciju sa odgovarajućom dozvolom. Kapacitet mašine je 250 t/h građevinskog neopasnog otpada.

Površinski kop udaljen je 35 km od Niša. Savremene drobilice, postrojenje za separaciju i sejalica efikasno usitnjavaju i razdvajaju kamene agregate po veličinama. Tehnički kapacitet trenutne primarne drobilice je 300 t/h.

Uslugu transporta vršimo automikserima, kapaciteta bubnja od 7 m³ do 10 m³ betonske mase. Za ugradnju betona posedujemo auto-pumpu za beton, radnog učinka 150 m³/h, sa dužinom strele od 36 m.

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