



24. Šukljetrov dan

.....
ZBORNIK VABLJENIH PREDAVANJ

Rogaška Slatina, 2. oktober 2025
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24.

Šukljetov

dan

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ŠUKLJETOV DAN (24; 2025; Rogaška Slatina)

Zbornik vabljenih predavanj / 24. Šukljetov dan, Rogaška Slatina, 2. oktober 2025

Izdaja: Slovensko geotehniško društvo

Zanj: Suzana Svetličič

Uredila: Suzana Svetličič in Sebastjan Kuder – Slovensko Geotehniško društvo SloGeD, 2025

Oblikovanje ovitka in zasnova: Sonja Eržen

Oblikovalska realizacija: Leon Beton

Tisk: Demat d.o.o.

Naklada 200 izvodov

Ljubljana, oktober 2025

Organizacijski odbor 24. Šukljetovega dne:

Predsednica SloGeD: ga. Suzana Svetličič

Podpredsednik: doc. dr. Borut Macuh

Tajnik SloGeD: mag. Sebastjan Kuder

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Član: doc. dr. Stanislav Lenart

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Soorganizator: Inženirska zbornica Slovenije (IZS)



CIP - Kataložni zapis o publikaciji

Narodna in univerzitetna knjižnica, Ljubljana

624.13(082)

ŠUKLJETOV dan (24 ; 2025 ; Rogaška Slatina)

24. Šukljetov dan : zbornik vabljenih predavanj : Rogaška Slatina,

2. oktober 2025 / [uredila Suzana Svetličič]. - Ljubljana : Slovensko

geotehniško društvo, 2025

ISBN 978-961-97134-0-2

COBISS.SI-ID 249855747

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University College London, United Kingdom

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Fakulteta za gradbeništvo, prometno inženirstvo in arhitekturo,

Univerza v Mariboru

Optimalna zasnova geotermalnih pilotov 58

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Uporaba InSAR za spremljanje stabilnosti pobočij in objektov 88

IZ FOTOGRAFSKEGA ARHIVA – 22. ŠUKLJETOV DAN Portorož, 8. december 2023



Otvoritev 22. Šukljeto dne v Portorožu 8. decembra 2022 je zazvenela ob skladbi »In the Mood« v izvedbi skupine Ladybugs.



22. Šukljeto dan je bil posvečen Sekciji za geosintetike. Strokovni del je vodil dr. Stanislav Lenart (predsednik Sekcije za geosintetike).



Poslovili smo se od Mojce. Ostala nam bo v dragocenem spominu.



22. Šukljeto predavatelj je bil svetovno priznani prof. Fumio Tatsuoka z Univerze v Tokiu. Predavanje je potekalo na daljavo.



Med vabljenimi predavatelji je bil prof. Erol Tutumluer z Univerze v Illinoisu (Urbana–Champaign).



Zbralo se je več kot 90 udeležencev: strokovnjaki z univerz in raziskovalnih ustanov, projektanti, predstavniki izvajalcev ter drugih strokovnih institucij s področja geotehnike.



Domači vabljeni predavatelj je bil izr. prof. dr. Primož Jelušič, visokošolski učitelj in raziskovalec na Fakulteti za gradbeništvo, prometno inženirstvo in arhitekturo Univerze v Mariboru.



Prisotnost številnih domačih ter tudi tujih strokovnjakov je še dodatno potrdila pomen srečanja kot osrednjega dogodka geotehniške stroke v Sloveniji.



V odmoru je stekel sproščen pogovor ter izmenjava izkušenj.



V okviru 22. Šukljetovega dne je potekala prva skupščina Sekcije za geosintetike pod vodstvom dr. Stanislava Lenarta.



Glavni sponzor 22. Šukljetovega dne je bilo podjetje Geoinvest, ki je s svojo podporo pomembno prispevalo k izvedbi dogodka. Iskrena zahvala gre tudi vsem ostalim sponzorjem in razstavljavcem, ki so s svojo prisotnostjo in prispevki obogatili program in omogočili uspešno izvedbo srečanja.



S šopkom rož sta predsednica in tajnik društva simbolično zaključila 22. Šukljetov dan.

IZ FOTOGRAFSKEGA ARHIVA – 23. ŠUKLJETOV DAN Postojna, 6. december 2024



Nemoten potek registracije sta zagotovili študentki Julija in Patricija – preddverje plesne dvorane Jamskega dvorca, Postojna, 6. 12. 2024.



Sproščen trenutek tik pred začetkom: Timotej in Janko.



Otvoritev 23. Šukljetovega dne z nastopom violinistke Maše Golob.



Zadnja usklajevanja pred začetkom programa; tajnik in predsednica društva z moderatorko Klaro Evo Kukovičič.



Spomin na pionirja geotehnike prof. Luja Šukljeta je obudil Janko.



Kot 23. Šukljeto predavatelj je zasl. prof. António Gomes Correia (Univerza Minho, Portugalska) predstavil predavanje z naslovom »Uporaba umetne inteligence v geotehniški prometne infrastrukture – Trenutni izzivi in prihodnje priložnosti v raziskavah in inženirski praksi«, pri čemer je poudaril razvojne smernice in vpliv na inženirske odločitve v prihodnosti.



Peter Jemec (ELEA iC) je predstavil prispevek: »Gradnja predorov v močno zakraskeli kamnini z veliko prevotljenostjo« – od načrtov do prakse pri gradnji predorov v zakraskeli kamnini ter o izzivih pri gradnji.



Sledilo je domače predavanje Timoteja Jurčiča (UL FGG – Katedra za geotehniko) z naslovom »Uporaba resonančne kolone pri merjenju dinamičnih lastnosti zemljin Ljubljanskega barja«.



Utrinek iz dvorane: Zbralo se je več kot 100 udeležencev.



Neformalna izmenjava izkušenj v predverju plesne dvorane.



Ob zaključku 23. Šukljatovega dne: prof. Edina Koch, prof. Sonja Zlatović in prof. António Gomes Correia (iz leve proti desni) skupaj s predstavniki Izvršnega odbora SloGeD.



Strokovni klepeti in nova poznanstva.



Strokovna ekskurzija 7. 12. 2024: Ogled gradbišč na trasi drugega tira železniške proge med Divačo in Kopro (Vodenje: Peter Jemec, Elea iC in Katarina Sirk, Yapı Merkezi).



MATTHEW RICHARD COOP

Professor Matthew Coop has over forty years' experience, focusing on research into the behaviour of soils and weak rocks through high-quality laboratory testing. After early industrial practice in off-shore foundations and a doctorate at the University of Oxford under Professor Peter Wroth, he began his academic career at City, University of London in 1986, where he spent the formative part of his career developing new testing techniques to examine the fundamental mechanics of sands. He moved to Imperial College London in 2000, was promoted to Professor in 2007, and worked closely with Professor Richard Jardine on a range of projects, largely on the mechanics of UK clays. He then spent six years at the City University of Hong Kong, returning to University College London (UCL) in 2016.

His recent work has moved away from traditional continuum laboratory techniques, such as triaxial testing, towards micromechanics-based approaches, developing new methods for testing individual soil and rock particles. He has supervised twenty-six completed PhDs and authored around 130 journal papers, cited over 10,000 times. With various co-authors he has received the British Geotechnical Association's annual prize three times, the UK Institution of Civil Engineers' Geotechnical Research Medal three times, two Telford Premiums and the George Stephenson Medal of the ICE, as well as a best paper award from the Japanese Geotechnical Society. He delivered the 2004 Géotechnique Lecture and the 2023 Bishop Lecture, the leading international lecture on laboratory soil testing.

Professor Coop has served on the editorial boards of eleven international journals and on four technical committees of the International Society of Soil Mechanics and Geotechnical Engineering (ISSMGE). He was Editor-in-Chief of *Géotechnique* from 2020–2023 and the Founding Editor of *Géotechnique Letters* from 2011–2014. He is currently Chair of ISSMGE Technical Committee 101, which oversees international activities in the laboratory testing of soils.

Professor Matthew COOP

Professor of Geotechnical Engineering, University College London

FROM CONTINUUM TO PARTICULATE MECHANICS IN COARSE GRAINED SOILS

Abstract

Of all the advances in soil mechanics of the past few decades, few are as dramatic as our understanding of the behaviour of sands and how this affects engineering design in these soils. We have progressed from considering them as the poor relation of clays within the panoply of soil mechanics to having their own critical state based behavioural framework, that is quite distinct from clays and which accounts properly for the particle breakage which is predominant in these soils. The lecture outlines the background mechanics of sands and gives examples of how it may be applied to the design of piles. In recent years, the collapse of numerous tailings disposal structures has finally come to the attention of the geotechnical community, and the insight that our understanding of sand behaviour brings to these fellow non-plastic soils is discussed, highlighting how we can assess correctly their risk of static liquefaction, the major cause of death in previous disasters. Finally, the deficiencies of a traditional continuum-based approach to sand behaviour are discussed, along with our recent forays into the micromechanics of single sand particles, which may well herald a particulate based future for our design in these materials.

Keywords:

Sands, continuum mechanics, piles, tailings, micro-mechanics.

OD MEHANIKE KONTINUUMA DO MEHANIKE DELCEV V GROBOZRNATIH ZEMLJINAH

Povzetek

Med dosežki mehanike tal zadnjih desetletij je le malo tako prelomnih kot novo razumevanje obnašanja peskov ter posledic, ki jih to ima za geotehnično načrtovanje. Od obdobja, ko so bili peski v okviru mehanike tal obravnavani kot »približek« glini, smo napredovali do samostojnega okvira obnašanja, utemeljenega na zasnovi kritičnega stanja, ki je od glin bistveno drugačen in ustrezno upošteva drobljenje zrn – prevladujoč proces v teh zemljinah. Predavanje povzema temeljno mehaniko peskov ter prinaša primere uporabe pri načrtovanju pilotov.

V zadnjih letih so številne porušitve jaloviščnih pregrad dokončno pritegnile pozornost geotehnične skupnosti. V prispevku je predstavljeno, kako naše razumevanje obnašanja peskov prispeva k obravnavi sorodnih neplastičnih (zrnatih) zemljin, zlasti k pravilni oceni tveganja za statično likvefakcijo, ki je bila glavni vzrok smrtnih žrtev pri preteklih nesrečah.

Na koncu so predstavljene pomanjkljivosti tradicionalnega kontinuumskega pristopa pri opisovanju obnašanja peskov ter naši nedavni koraki na področju mikromehanike posameznih zrn peska, kar utegne naznanjati, da bo geotehnično projektiranje v teh materialih v prihodnje temeljilo na delčni – partikularni obravnavi.

Ključne besede:

peski; kontinuumska mehanika; piloti; jaloviščne pregrade; mikromehanika.

1 INTRODUCTION

Over the last few decades, the mechanics of sands has emerged from vague ideas of their following a similar critical state framework to clays to a full appreciation of the real effects of their initial density and the role of particle breakage in their behaviour. Constitutive models of their mechanics are now every bit as sophisticated as those for clays. It would be an exaggeration to say that everything changed because of one episode, but the problems encountered at the North Rankin A platform in 1982 were certainly very influential, first in understanding how carbonate (or calcareous) sands behave, and secondly that these sands were not in fact “special” but just they highlighted how we had not really understood fundamentally the behaviour of any sands, even the more conventionally encountered quartz sands.

The gas platform was founded in 126m of water depth on the Australian North-West shelf. The 32 1.83m diameter open steel pipe piles were designed to be driven to 116m below mudline and to carry their load predominantly in shaft friction (Fig.1). As the piles were stabbed into their sleeves around the platform legs, problems immediately became apparent when they penetrated up to 64m into the carbonate sands under their self-weight. Subsequent driving resistances were then very much lower than predicted and significant resistance was only achieved when the piles reached a cemented stratum, or calcarenite, at around 113m (Senders et al., 2013). The platform could not be certified for the tropical cyclone conditions that can occur at its location until the piles had been drilled out and plugged, half of them being underreamed into the calcarenite. Certification was finally achieved in 1988.

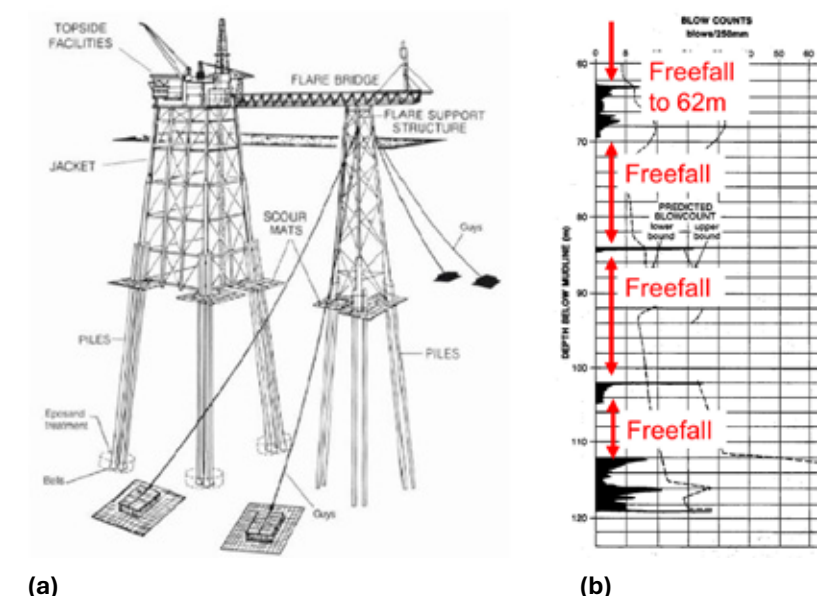


Figure 1: The North Rankin A platform (a) general layout, (b) pile driving record (after Khorshid et al., 1988).

While it had been understood from the start that carbonate sands would not behave like a quartz sand, and alternative so design methods had been adopted, it was not perhaps appreciated exactly how the breakage of the delicate shelly particles of the biogenic sand would lead to a large volumetric compression, reducing dramatically the radial stresses on the piles and hence their driving resistance and capacity. More seriously, the industry had not learnt from previous problems in carbonate sands around the world, particularly in the Persian Gulf and at the Garoupa platform offshore Brazil, which had experienced similar problems just two years previously. The exact costs that occurred through the problems at Rankin are difficult to trace but anecdotally are said to be over \$0.8 billion at present costs.

A major research effort was initiated into the mechanics of carbonate sands, which led to two specialised conferences on these sands in Perth in 1988 and Bahrain in 1999. However these ceased as we began to realise that there was nothing inherently special about carbonate sands and that quartz sands behaved with a basically similar framework, but that the particle breakage which caused problems was generally encountered at stresses that are typically beyond the engineering range. Interestingly, there has been a flurry of new activity on carbonate sands generated by China for their development of the South China sea, although many of the lessons of the past often seem to have been forgotten.

Initially research effort on sands was focussed on understanding the behaviour of sands within a continuum framework, using familiar techniques such as triaxial tests, albeit at higher than traditional stress levels, but as we began to understand the complexities that would arise from the nature of sand particles, notably the breakage they undergo, we started to realise that perhaps the future may lie in a more particulate based approach.

2 BACKGROUND SOIL MECHANICS

The difficulties faced at Rankin prompted a detailed study of the mechanics of carbonate sands. For any new material encountered, the two key types of loading to be investigated are isotropic compression and shearing. Isotropic compression is preferred to one-dimensional, because even if soils are almost never subjected to isotropic stresses in practice, it is a more fundamental than one-dimensional (e.g. the oedometer) because there are no shear stresses within the soil. Figure 2 shows isotropic compression data for Dogs Bay sand, a carbonate sand from Western Ireland. The isotropic compression is simply achieved by increasing the cell pressure in a triaxial apparatus, but it will be noticed that the stresses reached are higher than common in soils testing and specialised high pressure triaxials had to be constructed since suitable commercial apparatus did not exist at that time. The graph is plotted with axes of specific volume, v , against the logarithm of p' where p' is the mean normal effective stress, i.e. $(\sigma'_a + 2 \sigma'_r)/3$ where σ'_a and σ'_r are the axial and radial effective stresses, which are equal during isotropic loading. Within critical state soil mechanics, the specific volume is preferred to the void ratio e , but it is simply $e+1$.

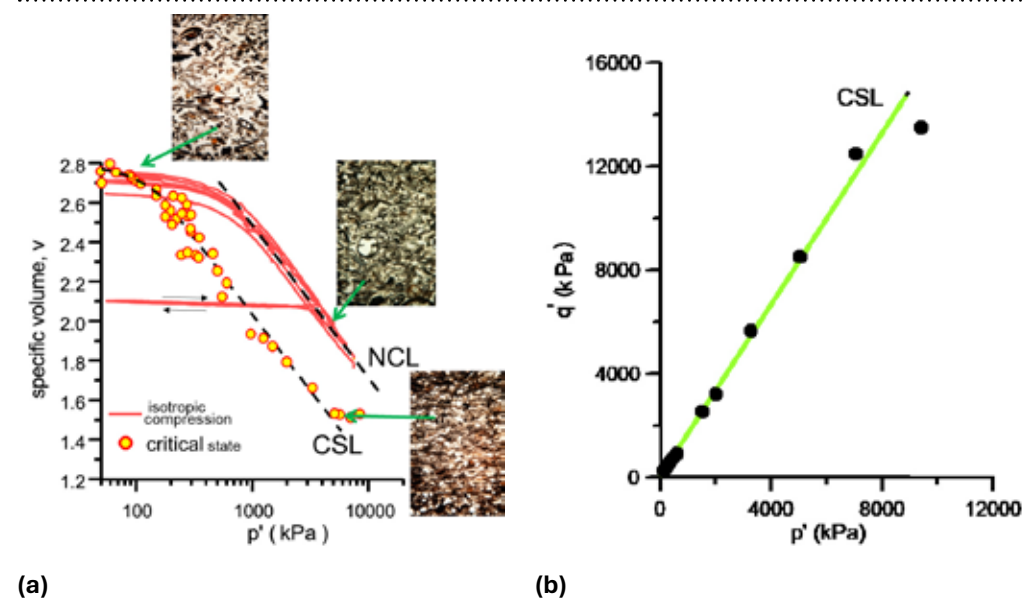


Figure 2: The mechanics of Dogs Bay sand (a) isotropic compression and critical state data, (b) critical state failure envelope (modified from Coop & Lee, 1993 and Coop, 1990).

As for all sands, reconstituted samples may be made at various initial volumes depending on the degree of compaction during sample preparation, but it can be seen that no matter what that initial v is, the compression curves converge towards one unique line at high stresses, and this is referred to as the Normal Compression Line, NCL. It is analogous to that for clays, but for sands it is only reached after an initial stiff compression phase, while in clays the NCL is reached at very low stresses and high volumes that are not normally encountered. For clean or uniformly graded sands like the Dogs Bay sand the yield as the NCL is reached is caused by the onset of particle breakage, that breakage being evident in the thin sections before and after compression in the figure. At that time, it was thought that all non-plastic soils would have particle breakage as they reached their NCL, but we shall see later that this turned out not to be correct. Once sand particles are broken they cannot be “unbroken” so the behaviour during unload-reload (the swelling line) is very stiff indeed and much stiffer than any clay would be.

Following isotropic loading, the samples were sheared using either drained or undrained loading. The key tenet of critical state soil mechanics is that provided a soil is sheared to large enough strains, states of constant stress and constant volume (or pore pressure in an undrained test) can be reached so that the values of p' , v and q' (the deviator stress, $\sigma'_a - \sigma'_r$) are constant (Fig.3). A difficulty in sands is that the strains required to reach those critical states are greater than would typically be applied in a commercial test. Points representing the critical states from many triaxial tests are added to Fig.2, where they identify a locus, the Critical State Line, CSL, that is parallel to the NCL at higher stresses, but which curves to a horizontal asymptote at low stresses. Shearing is accompanied by greatly increased particle breakage, as can be seen in the thin section of the sheared sample, where the original particles are barely distinguishable.

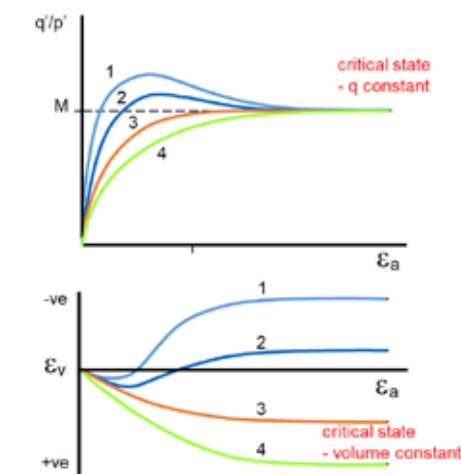


Figure 3: Schematic diagram of shearing behaviour for sands.

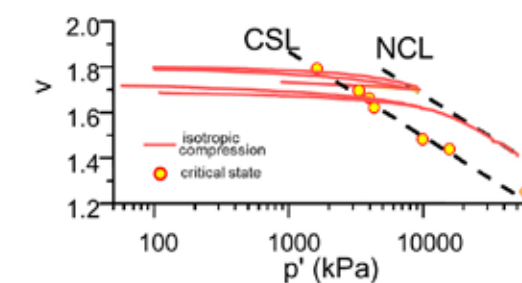


Figure 4: Isotropic compression and critical state data for a quartz sand (modified from Coop & Lee, 1993).

Accurate critical states are actually quite hard to achieve in sands, because of the large strains required, and it is common for papers to claim that the locus of the critical states (the CSL) in the $q':p'$ plane is curved. This is generally the result of incomplete testing, and the data in Fig.2 define a straight line, given some scatter arising probably from the non-uniform deformations that are prevalent within the sample at such large strains. The gradient of the CSL failure envelope, $M (q'/p')$ for this carbonate sand is very high, at 1.65 corresponding to a critical state angle of shearing resistance, ϕ'_{cs} , of 40° , the high value resulting from the angular nature of the largely shelly particles that constitute the sand. Contrary to popular conception, carbonate sands do not have a low strength as a bulk material, but they are very highly compressible as a result of the low strength of individual particles.

By comparison the data for a typical quartz sand (Ham River sand) in Fig.4 show yield in compression at very much higher stresses that would normally be encountered, except around tip of a driven pile, for example. The high yield stress is not only a result of stronger particles but also their solid rounded shape which means that the initial values of v are low. Those shapes also dictate a much lower ϕ'_{cs} , of 32° . The locations of NCLs/CSLs, the values of the yield stress in compression and also ϕ'_{cs} , are therefore functions of the particle shapes and strengths. The general patterns of behaviour in Figs.2 and 4 are the same, so there is nothing inherently unusual about carbonate sands, but one key difficulty lies in the variability of their geological origins and hence particle properties and behaviour, compared to quartz sands that are generally all pretty similar.

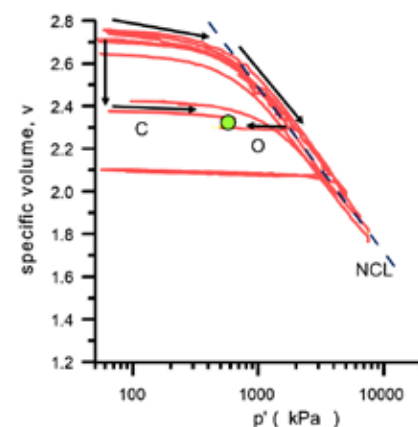


Figure 5: Overconsolidated and compacted states is isotropic compression (modified from Coop & Lee, 1993).

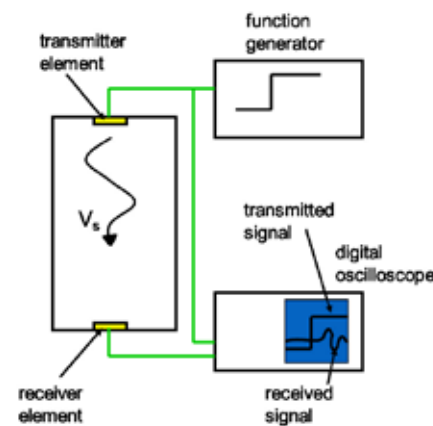


Figure 6: Schematic diagram of bender element tests.

In one aspect the behaviour sands departs significantly from what is seen in clays. This is illustrated in Fig.5 where a given state can be reached either by overconsolidation after loading to a higher stress (Path O) or by compaction and first loading only (Path C). Clearly the subsequent shearing behaviour must be different as sample O will have undergone more particle breakage than C. This was investigated by means of examining the elastic shear modulus, G_0 , of samples of both types using bender element tests. As indicated in the schematic diagram of Fig.6, two piezo-ceramics protrude into the sample from the top platen and base pedestal of the triaxial. The top one is used to transmit a shear wave through the sample that is then received by the base element, using the time delay to calculate the shear wave velocity, v_s and hence G_0 from ρv_s^2 , where ρ is the mass density of the soil. Strictly, this is the G_0 in the vertical plane and for most soils, their anisotropy will mean that G_0 in the horizontal plane will be different. Although the value of G_0 might be thought to be slightly academic, it is the start of the non-linear stiffness decay curve and so will define the stiffnesses needed in engineering design (Fig.7).

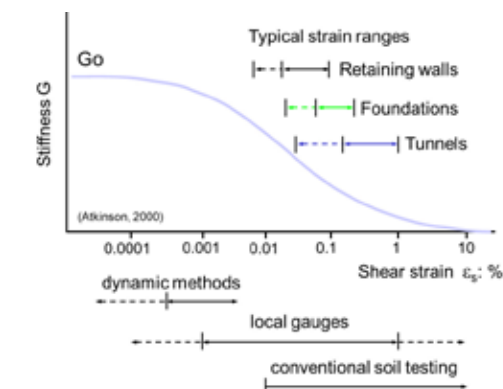


Figure 7: Schematic diagram of shear modulus decay curve (modified from Atkinson, 2000).

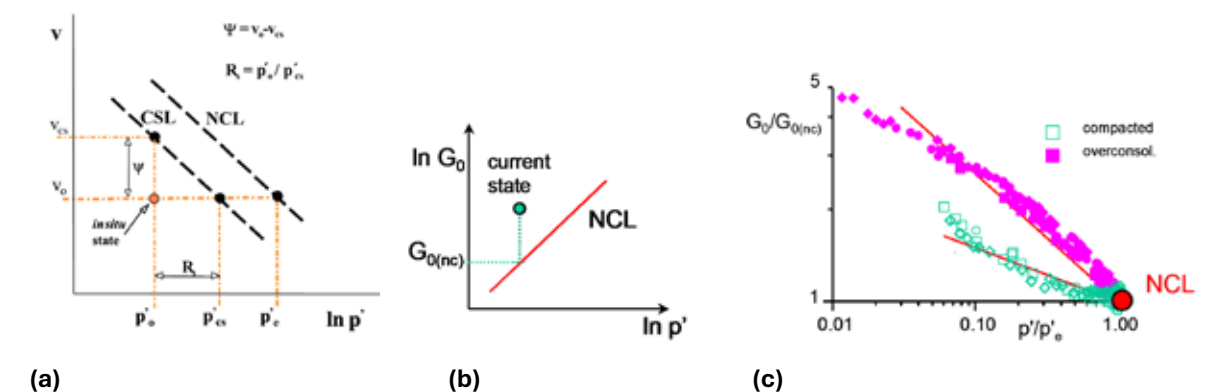


Figure 8: Normalisation of G_0 measurements for Dogs Bay carbonate sand (a) definition of p'_0 , (b) normalisation of G_0 , (c) normalised data (modified from Jovicic & Coop, 1997).

The values of G_0 for samples of varying initial density are given in Fig.8 for both compaction and overconsolidation approach paths. The overconsolidated samples are much stiffer, as might have been expected, and this emphasises the need to understand properly the geological loading history of sands, just as we would routinely do for clays. However, there are clearly effects of the initial sample density and the overconsolidated samples also tend to have lower volumes than the compacted. For a rigorous comparison we need to eliminate the effects of volume and we do that by taking the distance of the current state from either the NCL or CSL and some common normalising parameters for doing this are shown in Fig.8a. Here we have used the equivalent stress on the NCL at the current specific

volume, p'_c , so the distance from the NCL is quantified as p'/p'_c . We similarly need to normalise the G_0 values and this is done in Fig.8b by dividing our measured value by the G_0 we would have for a sample on the NCL at the current stress level p' . If we do that, we find that the data for all densities follow the same trend, but with the expected higher stiffnesses of overconsolidated compared to compacted samples. Although this example is given for a carbonate sand, the same behaviour was seen for simple quartz sands. This dependence of the behaviour of a soil on its location in terms of volume and stress relative to the CSL or NCL is central to critical state soil mechanics and we will see offers us ways to apply these analyses in engineering practice.

In soil mechanics the mechanics of silts have often been confused because we don't know whether to treat them like clays or like sands, and this is often complicated by natural silts often having some clay content and/or a high organic content, especially when found in a natural estuarine environment. This can give them a substantial plasticity, like, for example, the Bothkennar clay that was extensively tested in the UK in the 1980s and 1990s (e.g. Hight et al., 1992). However, pure silts are, like sands, clastic materials, and are composed of broken bits of rock minerals and have none of the plasticity of clays that is created by their complex inter-particle electro-static interactions. It is then perhaps little surprise that in the absence of any substantial clay or organic content, silts behave in a very similar manner to sands. An example is given in Fig.9 where the CSL of the Bostanj silt is shown. One alarm bell started to ring though, as result of these and similar data for silts, which was that there was much less particle breakage than for sands, as can be seen from the gradings before and after testing. The onset of significant breakage therefore did not correspond with the steepening of the CSL as had been found for sands. An interesting aside is that the very smallest particles don't actually break at all. This is because for every mineral a comminution limit exists, typically around a few microns, below which the mineral will only deform plastically but not fracture (Kendall, 1978).

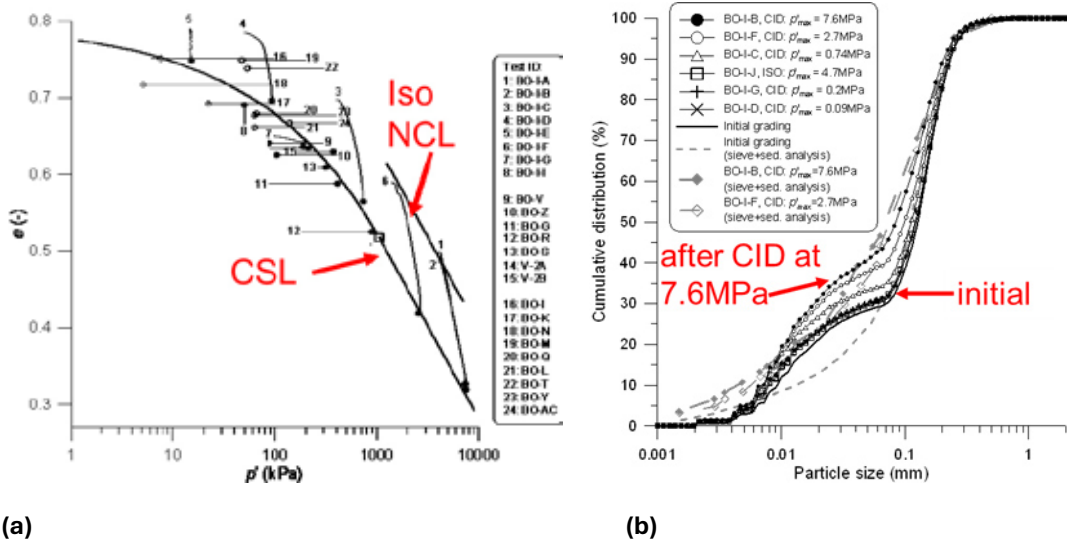


Figure 9: The Bostanj silt (a) CSL, (b) particle breakage (modified from Vilhar et al, 2013).

3 STATIC LIQUEFACTION IN MINE TAILINGS

The simple critical state type framework used above to describe the behaviour of sands leads to some immediate design applications, without any further complexity. One of key importance is the stability of disposal facilities for mine waste. Typically these are placed behind tailings dams, for example at the Stava facility shown in Fig.10. The dam is generally constructed from the more sandy tailings with the siltier materials deposited behind in a lagoon. This particular facility was built with the upstream method in which each heightening of the dam was constructed on top of finer tailings that had already been placed, a method that has now been largely discredited because it has been associated with a high proportion of problems. It was also unusual in that it was constructed on a hillside with one dam placed above another, so the consequences of the failure in 1985 were especially catastrophic with about 240,000 m³ of liquefied tailings moving at speeds of up to 90 km/h 4 km down the valley, destroying one village and killing 268 people.

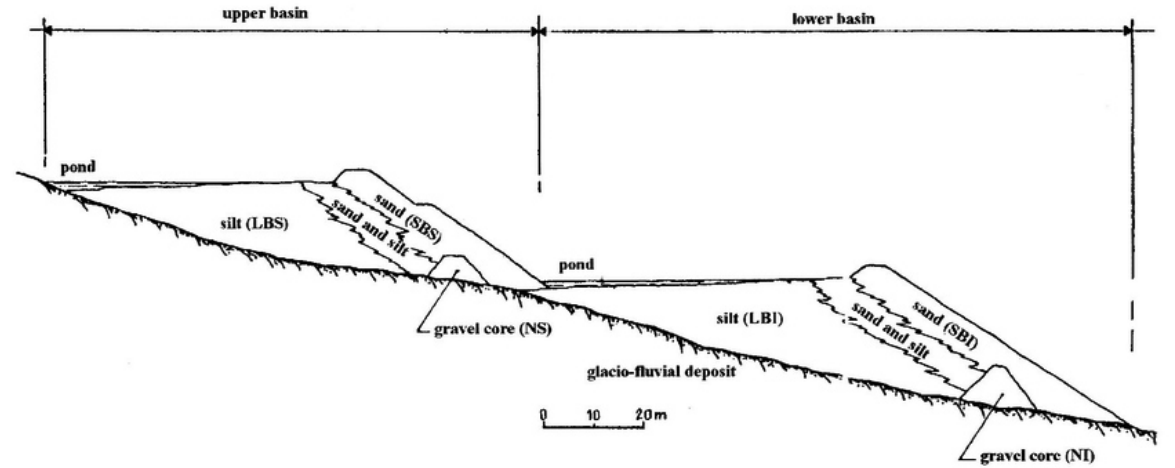
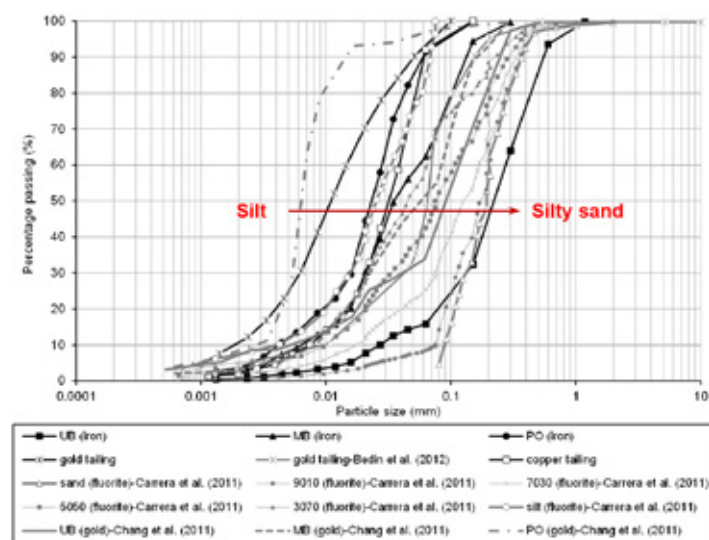
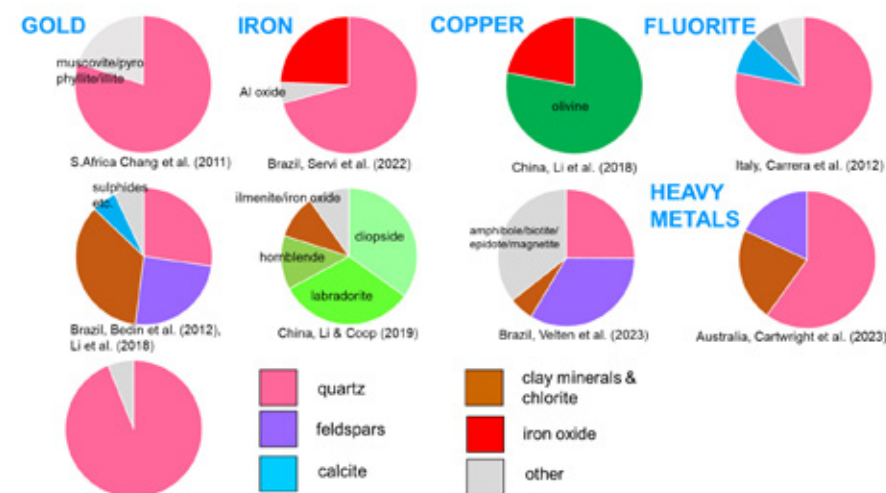


Figure 10: The Stava tailings dams (after Colombo & Colleselli, 2003).



(a)



(b)

Figure 11: The variability of tailings (a) gradings (after Li et al., 2018), (b) mineralogy.

Annual tailings production worldwide is currently at about 5–7 billion tonnes, and there are now about 8000 tailings disposal facilities containing 217 km³ (Qi & Fourie, 2019). Franks et al. (2021) have estimated that around 10% of existing tailings dam facilities have stability issues. In the decade to 2021 there were 39 major failures, 11 causing death with at least 620 killed (WISE, 2021). Most notable had been the Bento-Rodrigues 2015 and the Brumadinho 2019 failures in Brazil. The liquefied tailings from both flowed down their respective valleys, Bento-Rodrigues killing 19 and releasing 62 million m³ of tailings that travelled 650 km and caused an ecological disaster with the almost complete extermination of the Doce river's fauna. Brumadinho was smaller and released 11 million m³ but killed 270 people. The problems of tailings dams might be thought to have been exacerbated by the lack of high-level geotechnical design input until after, perhaps, the Stava disaster.

Since they are the waste materials of crushed rock from the mineral extraction process, tailings are necessarily generally clastic, non-plastic and quite fine grained, rather similar to many natural silts or fine sands, although their particles tend to be more angular because of their origin as crushed

materials. Typical gradings curves are shown in Fig.11a. A key difference is that they often have unusual mineralogies inherited from the parent rocks (Fig.11b), and these are not even consistent for one type of extracted mineral. But perhaps the focus on the unusual mineralogy of tailings has been a distraction from what is essentially some quite simple soil mechanics that can explain their instability.

Tailings gradings vary with location within the facility. When the outfall into the lagoon is placed on the dam, the gradings become finer with distance from the dam, from sandier materials on the beach adjacent to the dam, to finer further down into the “pond”. The Stava tailings were tested at different gradings by combining a silty grading and a sandy grading in various proportions (Fig.12). As the grading became finer the CSL moved first down in the $e: \ln p'$ plane and then back up, as is typical for sands mixed with fines. We use natural logarithms $\ln$ for this graph only because the equations are defined in those terms not base 10 logs. In every case, the CSL is flat at lower stresses and curves at higher stresses, although the stresses applied here have not been sufficient to reach the straight CSL seen in Fig.2. However, in no case was there any significant particle breakage, and we can see that our previous assumption that the steepening of the CSL resulted necessarily from breakage was incorrect.

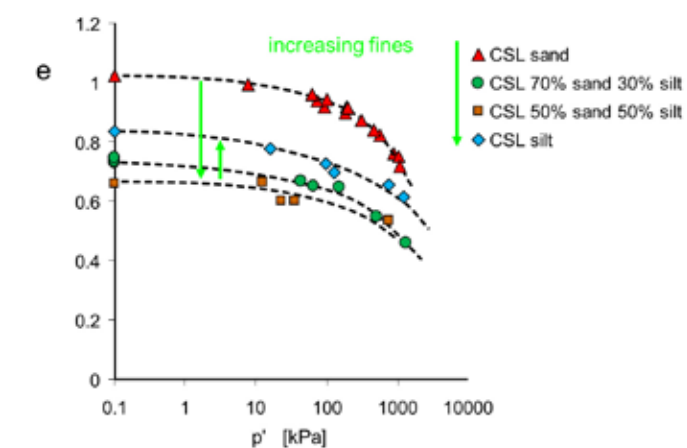


Figure 12: CSLs of Stava tailings of various gradings (modified from Carrera et al., 2010).

We can understand whether a tailings will liquefy simply with reference to this $e: \ln p'$ graph (Fig.13). Any sample in the red zone, e.g. Test 1, must liquefy when sheared undrained because the constant volume path that it must follow can never reach the CSL. It will therefore necessarily reach $p'=0$ and if so the strength q' must also be zero. Many published data are not clear about this because the tests are poorly done and the data not corrected properly for the restraint caused by the membrane stiffness. At higher stresses (the blue zone) Test 2 starts with a lower value of e as it has been compressed to higher stresses. At this value of v it is able to reach the CSL by following its constant e undrained path, but with a considerable reduction of p' and hence strain softening. It will however have a stable final critical state strength and will not liquefy. At higher stress levels (purple zone, Test 3) the samples will have even lower initial e values and the strain softening is less and the soil may even strain harden. The vocabulary used in the industry and even in academia is unhelpful to say the least. A test like Test 2 would be said to undergo “flow liquefaction” because of its strong strain softening. Such material behaviour would of course give a severe failure of the dam, but it could not account for the deadly rapid flow velocities and very long runouts that result from a true liquefaction like Test 1, which can only occur from volumes above the horizontal asymptote of the CSL. So to know which are the most dangerous tailings we need only to know their CSL location and their in-situ voids ratio, although that is of course not a trivial task. There is a small caveat on this framework; all of this is for triaxial compression tests, while the loading in a dam will generally be closer to plane strain and this may

well move the CSL. We should therefore also do some simple shear tests in the laboratory. The better understanding of liquefaction that the industry now has means that tailings are now often placed dry stacked at higher densities.

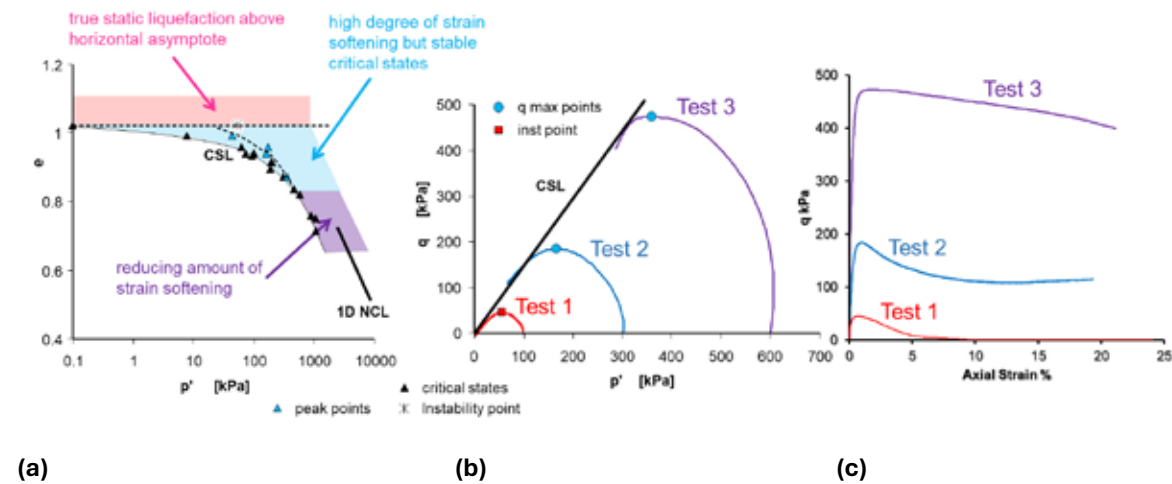


Figure 13: Liquefaction of Stava tailings of 100% sand grading (modified from Carrera et al., 2010).

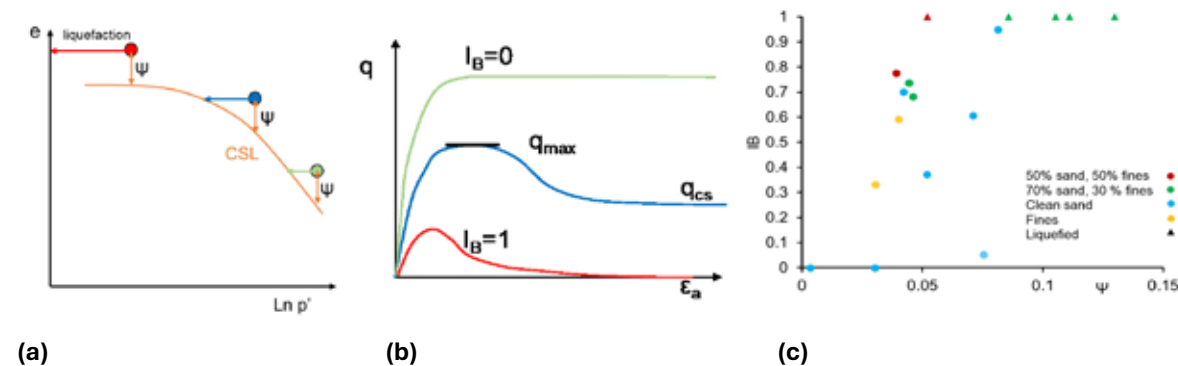


Figure 14: Using state parameter to identify liquefaction, (a) definition of state parameter, (b) schematic stress-strain behaviour, (c) relationship between state parameter and brittleness index for Stava triaxial data.

To understand whether a material will liquefy or not we therefore need to quantify where the state of the tailings is relative to the CSL in the $e: \ln p'$ plane. This could be with an equivalent pressure taken at the current value of e (or v) on the CSL, p'_{cs} in Fig.8, but much more commonly the state parameter, ψ , is used (Been & Jefferies, 1985), which is the vertical not horizontal distance from the CSL, i.e. the difference of e between the current state and that on the CSL at the same p' . This is illustrated schematically in Fig.14a. There would be no difficulty with this approach if the CSL were straight, but for a curved CSL the problems are manifest. The three states illustrated all have the same ψ but it is evident from Fig.13 that the stress-strain behaviours must be completely different (Fig.14b). Here the brittleness index, I_B , is introduced, which simply indicates the degree of strain soften and is defined as $(q_{max} - q_{cs})/q_{max}$ where q_{max} is the peak q of the stress-strain curve and q_{cs} is that at the end of the test (critical state). When there is no peak strength $I_B = 0$ and when there is complete liquefaction with zero critical state strength $I_B = 1$. When a correlation is tried between ψ and I_B for the Stava triaxial data, unsurprisingly it is unsuccessful (Fig.14c) with no clear correlation for any of the mixes tested. Because of these problems with state parameter, Bobei et al. (2009) introduced a modified state parameter, ψ_m , defined

in Fig.15a. The resulting correlation with I_B now works well in Fig.15b for the clean sand. However, even if there is insufficient data for the other mixes to establish similar relationships, it is clear that every soil mix gives a different relationship. This is not terribly helpful as it means that every new material encountered would need the CSL to be established in the $e: \ln p'$ plane in order to define ψ_m , but if you knew the CSL location you would already know which states would be those that liquefy, so nothing is gained.

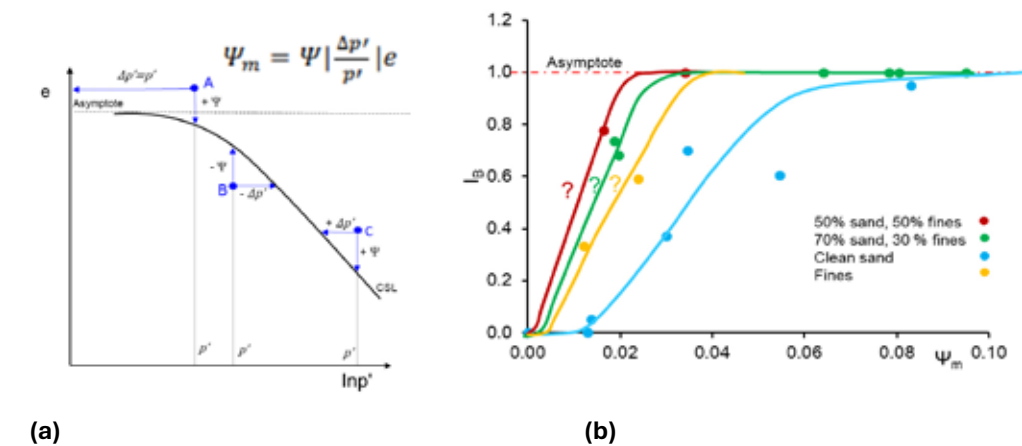


Figure 15: Using modified state parameter to identify liquefaction, (a) definition of modified state parameter of Bobei et al. (2009), (b) relationship between ψ_m and I_B for Stava triaxial data.

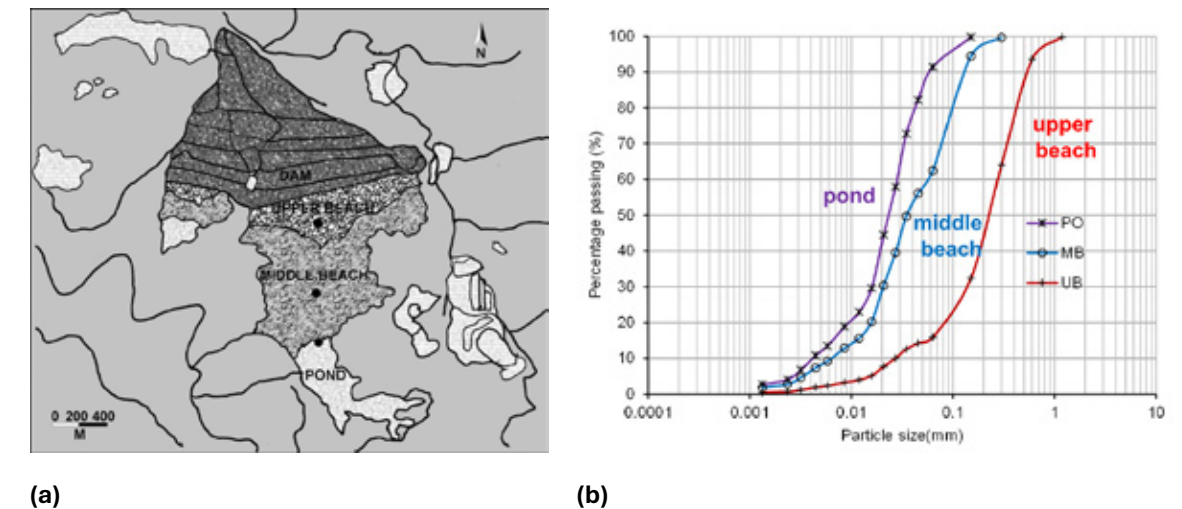


Figure 16: The Panzhuhua iron tailings, (a) locations and (b) gradings (after Li & Coop, 2019).

The Panzhuhua iron tailings studied by Li & Coop (2019) were sampled from three locations at the Wanniangou dam, the Upper Beach (UB), Lower Beach (LB) and Pond (PO). The gradings become finer with distance from the dam (Fig.16). In Fig.17 the CSL for the two coarser gradings is “sand” like and curved, while that for the Pond is straight within the range of stresses that could be reached in the triaxial, with no horizontal asymptote. This may be thought to be more “clay” like, but these are essentially non-plastic materials. Comparing with the CSLs for other tailings we can see that while the CSLs at higher stresses are all fairly similar, it is the horizontal asymptotes that show most variability and it is this that controls liquefaction. Meanwhile, the angles of shearing resistance are quite similar for all gradings and type of tailings.

The potential of a tailings to liquefy can be defined by considering where the horizontal asymptote of the CSL, v_{liq} , lies relative to the maximum and minimum possible specific volumes, v_{max} and v_{min} (or void ratios) that can be made during sample preparation; the closer v_{liq} is to v_{max} the wider the range of possible states that could potentially liquefy. Data from various tailings are gathered in Fig.18, including the iron tailings from Panzihua. With some scatter, especially an outlier from Chang et al. (2012), it can be seen that the v_{liq} typically lies mid-way between v_{max} and v_{min} and that location does not change much with the grading from Beach to Pond, so all gradings will be approximately equally susceptible.

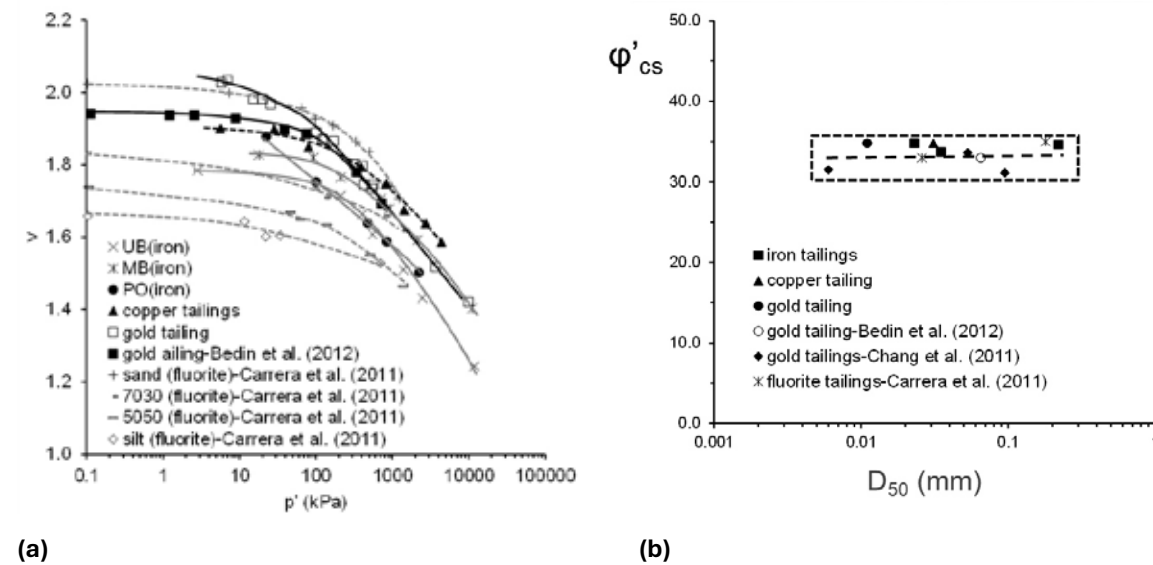


Figure 17: The behaviour of the Panzihua iron tailings compared with other tailings, (a) CSL location, (b) ϕ'_{cs} (after Li et al., 2018).

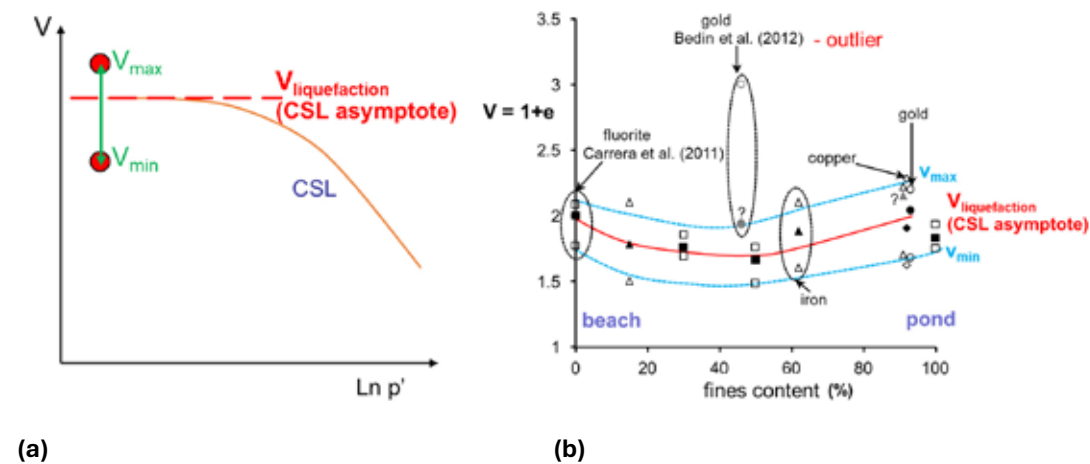


Figure 18: The liquefaction potential of tailings for different gradings, (a) definition of terms, (b) liquefaction potential (v_{max} and v_{min} open symbols, v_{liq} closed symbols) (modified from Li et al., 2018).

With the example of tailings liquefaction we can see how a simple consideration of the basic mechanics of the “soil” is all that is needed to understand deadly failures. We will now apply similar techniques to pile capacity in sands, although this will require us to quantify the location of the current soil state in the $v: \ln p'$ plane (or e) with some sort of state parameter, which was not strictly helpful for tailings.

4 THE CAPACITY OF PILES

Based on our proper understanding of the mechanics of sands, and of carbonate sands in particular, we can now return to the question of why pile capacity is so much lower in carbonate sands than in the more commonly encountered quartzitic sands. A series of geotechnical centrifuge tests was undertaken to investigate this, and a sketch of the experiment is given in Fig.19. The 16 mm model pile was very heavily instrumented and, in addition to measuring the end bearing, four measurements of the radial stress and five of the shaft resistance were made along the shaft. It penetrated by jacking up to around 350 mm into a strongbox containing the sand, which, using gravitational accelerations of 50 g, 100 g and 200 g, gave the equivalent prototype penetrations of 17.5, 35 and 70 m. Naturally such a model has many simplifications; it is jacked, not driven, it measures the installation resistance not the final capacity, the soils were dry not saturated, and the pile is solid and rigid compared to the prototype piles which are rather flexible open-ended steel tubes. Nevertheless, such a technique is the only way the mechanisms behind pile behaviour in sands can be investigated rigorously, and this type of research is just a first stage before one returns to examine prototype scale data.

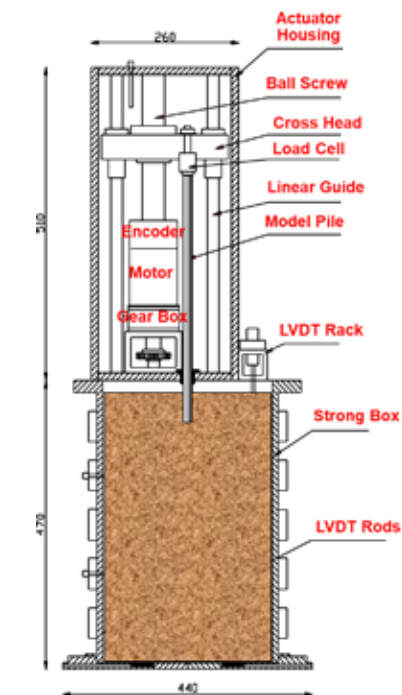


Figure 19: Geotechnical centrifuge model for a jacked pile in sand (after Klotz & Coop, 2001).

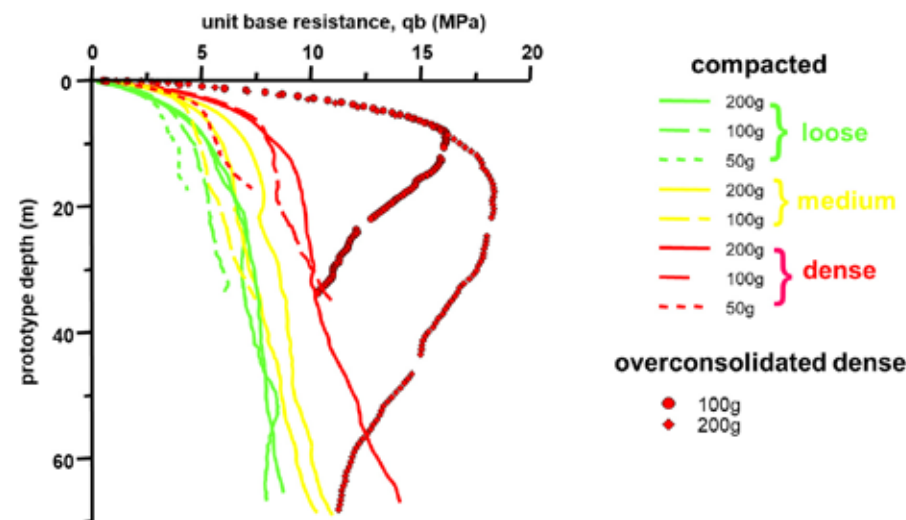


Figure 20: Geotechnical centrifuge test data for end bearing on a jacked pile in Dogs Bay sand (after Klotz & Coop, 2001).

Typical data are shown in Fig.20. These are for end bearing (unit base resistance, q_b) during penetration in Dog's Bay carbonate sand but tests were also carried out in Leighton Buzzard quartz sand. The end resistance clearly depends on the density the sample was prepared with, but it is also clear that the overconsolidated samples that have been loaded to a higher vertical stress and unloaded, behave differently, just as they did when we examined sand stiffness. Conventionally, we think of sand behaviour in terms of relative density, $D_r (=e_{\max} - e)/e_{\max} - e_{\min})$, even if we know that the values of the maximum and minimum void ratios $e_{\max}$ and $e_{\min}$ are somewhat arbitrary. In Fig.21 the unit base resistances have been normalised with respect to the in-situ vertical effective stress (σ'_v) to give end bearing capacity factor N_q . There is no reason to think that any relative density based method would work, and it does not and there is no useful trend to be seen. Each test has a high N_q at small penetrations that reduces with depth, but there is no relationship to D_r .

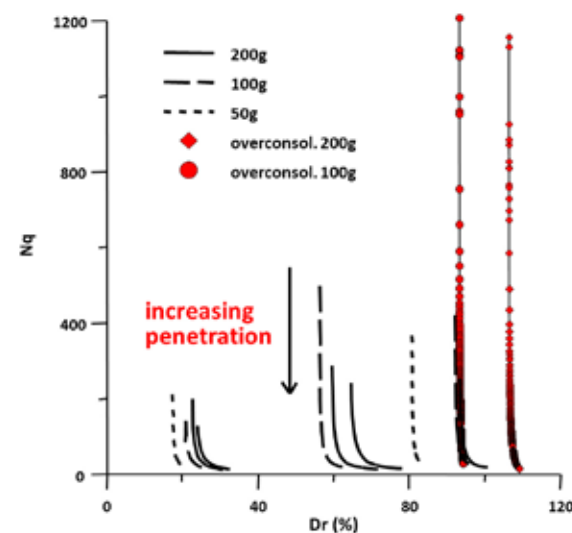


Figure 21: Pile end bearing in carbonate sand normalised by vertical effective stress and analysed in terms of relative density (after Klotz & Coop, 2001).

The correct approach, as discussed above, is to quantify the distance of the current state from either the NCL or CSL by means of some form of state parameter, either in terms of volumes, like ψ , or in terms of stresses. Here a stress state parameter is used, R_s which is equal to the current p' divided by that at the same v (or e) on the CSL (p'_{cs} , see Fig.8). To achieve this the in-situ v was needed and this was obtained from the initial v when the sample was made along with measurements of the settlement of the soil at various depths that were made during the centrifuge spin-up. The resulting plot is much more satisfactory (Fig.22). While the linear trend sketched is perhaps optimistic, it is clear that there are no longer any consistent differences between samples of different initial densities, and that the only thing that does have an effect is overconsolidation, exactly as we saw on Fig.8 for sand stiffness. This example is for end bearing in the carbonate Dogs Bay sand, but similar trends (but of course with different locations and gradients) were seen for the quartz sand and also for shaft friction in both soils. There was therefore nothing inherently different in the behaviour of piles in a carbonate sand compared to a quartz sand and their lower capacity arose simply from 1) their initially much higher in-situ values of v , and 2) different locations of the trends relating pile capacity to R_s .

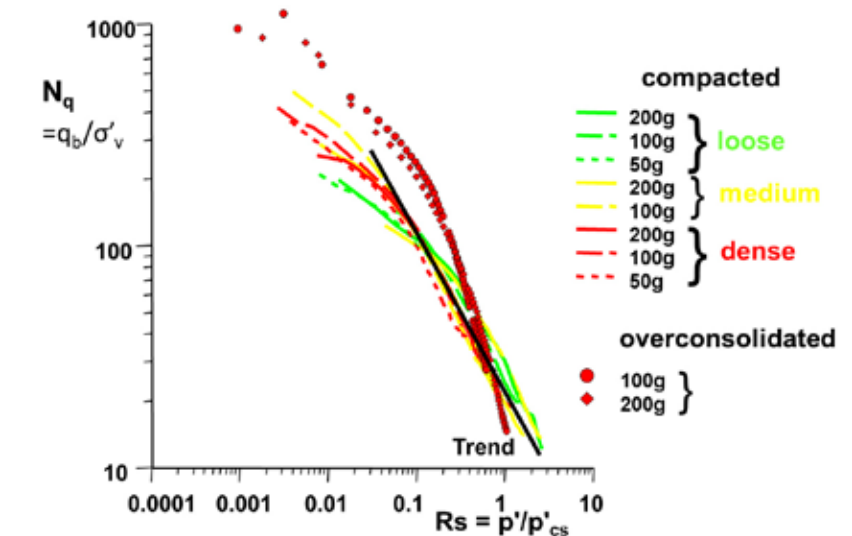


Figure 22: Pile end bearing in carbonate sand normalised by vertical effective stress and analysed in terms of stress state parameter, R_s (after Klotz & Coop, 2001).

In their load tests on bored piles in London's Docklands, Nicholson et al. (2002) found the unusual feature that the end bearing capacity reduced with increasing penetration into the Thanet sand (Fig.23). This they realised was associated with a change of stratum from a cleaner grading to a siltier one, but it was not understood why this should be the cause. A series of triaxial tests, however, revealed that the CSL of the low fines grading lay higher in the $e:\ln p'$ plane than the high fines (Fig.24). Together with the lower stress levels at smaller depths and the slightly denser in-situ states of the low fines sand, this meant that the distance of the in-situ state from the CSL was much greater, so that $R_s (p'/p'_{cs})$ was much lower. Analysed in these terms it makes perfect sense that there should be a lower capacity in the deeper, high fines, stratum, with a similar trend in Figs.22 and 24b, although the values are of course different as the sands and pile types are both different.

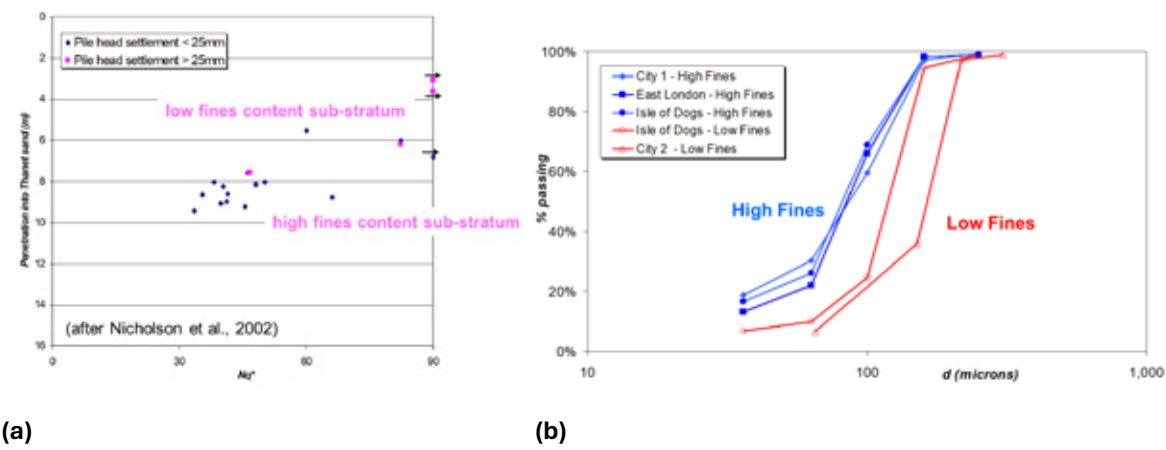


Figure 23: Pile end bearing for bored piles in Thanet sand at the Isle of Dogs, London, (a) pile test data (Nicholson et al., 2002), (b) gradings of the sub-strata (Ventouras & Coop, 2009).

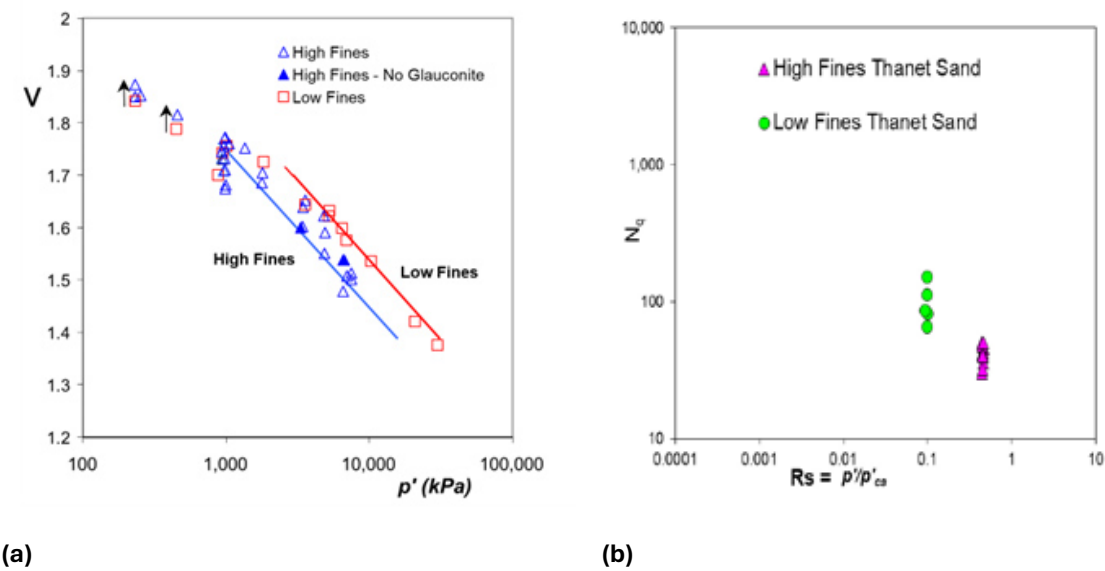


Figure 24: Pile end bearing for bored piles in Thanet sand at the Isle of Dogs, London, (a) critical state lines for Thanet sand, (b) normalised pile end bearing test data (Ventouras & Coop, 2009).

Figure 25 shows an attempt to apply the same methodology to the unit shaft friction, q_s , of piles measured in field tests. Values of the shaft friction coefficient β ($=q_s/\sigma'_v$) are plotted, taking a mean value for the whole pile. The trends for the shaft friction measured in the centrifuge tests are shown for reference, but they are low, probably as a result of the very smooth model pile. Given the very wide range of soil and pile types included, the scatter is remarkably small and there is a fairly good trend showing the influence of the in-situ state, apart from a few very low values on very flexible conductors in carbonate sands. This methodology can be applied equally well to pile load-deflection behaviour. This is illustrated in Fig.26 where the stiffness of the pile response in the centrifuge tests in Leighton Buzzard sand is analysed, showing a clear trend with R_s .

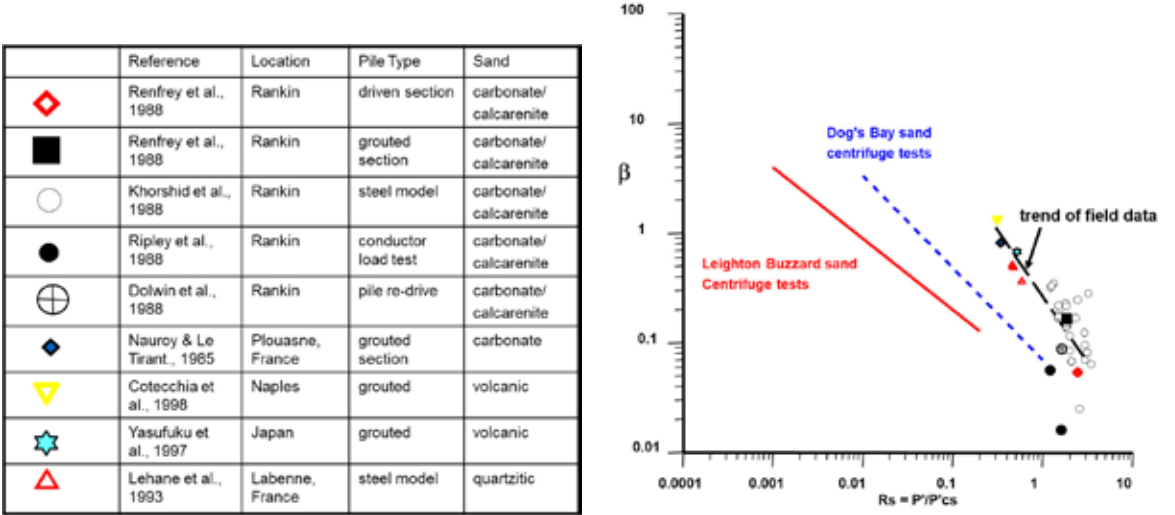


Figure 25: Pile shaft friction for various piles (modified from Klotz & Coop, 2001).

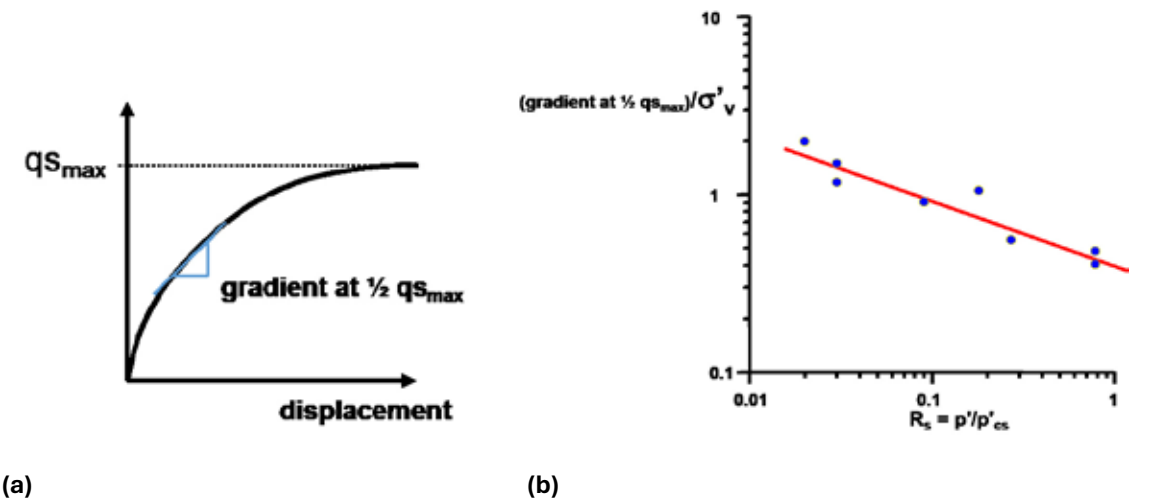


Figure 26: Pile stiffness from centrifuge tests (a) schematic diagram showing definition of stiffness, (b) pile stiffness analysed in terms of state from centrifuge tests in Leighton Buzzard quartz sand (Coop et al., 2005).

5 PROBLEMS WITH THE CONTINUUM MECHANICS OF SANDS

This lecture has only focused on some simple mechanics for sands based on the interaction of their volumes, their strength and stiffnesses, and how this is influenced and, in some senses complicated, by particle breakage. But for many engineering applications we might have to account for further complexities, notably 1) the mode of loading, i.e. triaxial compression, extension, plane strain etc., 2) anisotropy of properties and the effects of the rotation of stresses on those properties, 3) the influence of the recent loading history on the stiffnesses, 4) the state boundary surface being highly asymmetrical in shape so that the flow rule relating strain to stress increments is non-associated, or 5) the effects of the natural structure created by the geological history (fabric and bonding). Unfortunately these are not the limit of our woes with a continuum approach. A key assumption behind the critical state framework as originally proposed for sands was that compressive volumetric strains, either during compression or shear, were related to particle breakage. We have already seen that this breaks down for siltier materials, but in fact it is also incorrect even for clean sands.

When Chandler (1985) proposed his model for materials with deformable grains, he had in mind agricultural grains rather than sands, and he realised that an apparent critical state, where the volumetric strain ceased during shearing, could be brought about by the volumetric compression from the particle deformation being balanced by a dilation of the matrix of particles. This hypothesis was quickly extrapolated to breakable particles leading to two possible scenarios, 1) the breakage ceases at the end of a triaxial test when a critical state is reached at a few tens of percent strain, or 2) the critical state is only apparent and results from a balance of compression arising from breakage and dilation in the matrix. This was investigated by Coop et al. (2004) who carried out ring shear tests on sands, using the Dogs Bay carbonate sand to amplify the amount of breakage. The Bishop ring shear apparatus was used (Fig.27), making the vertical stress more difficult to control and the data are therefore grouped in three ranges of stress levels. The tests were stopped at chosen strains and the grading in the middle section of the sample, where the strains were more uniform, was examined.

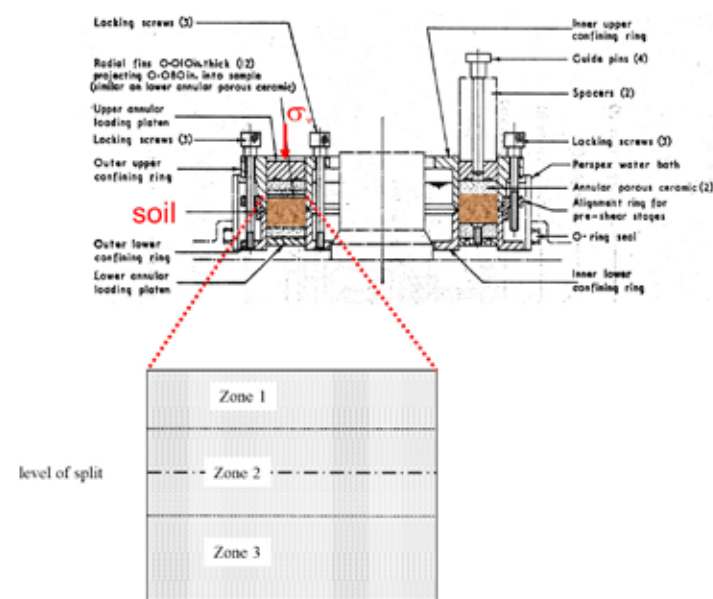


Figure 27: Configuration of the ring shear tests (modified from Bishop et al., 1971).

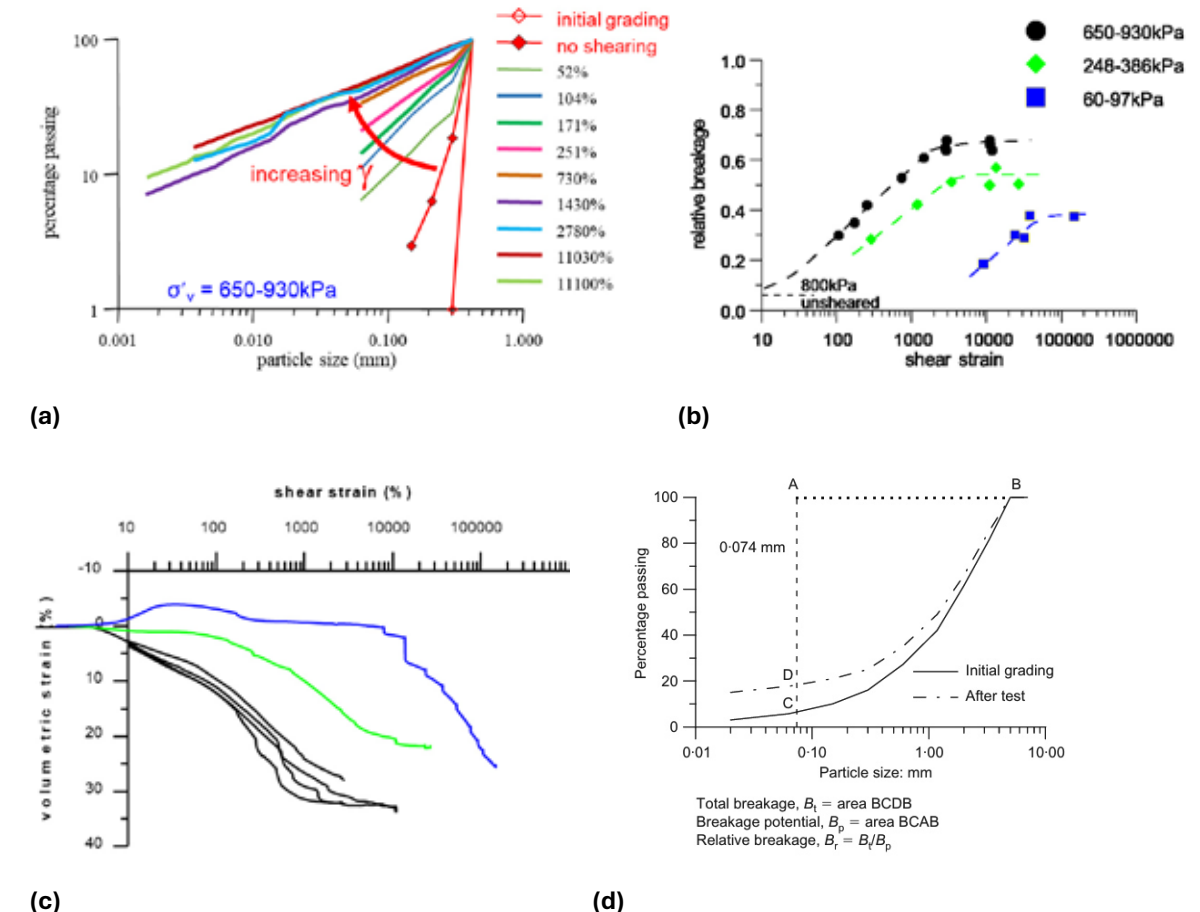


Figure 28: The evolution of particle size distribution in ring shear tests on carbonate sand (Coop et al., 2004) (a) gradings curves, (b) relative breakage, (c) volumetric strains, (d) definition of relative breakage, B_r (Hardin, 1985).

The data for the highest stresses show an evolution during shearing from the initial uniform grading towards a fractal grading, plotting as a straight line on a log: log graph. This has a fractal dimension of 2.57, which was slightly higher than the value of 2.5 that McDowell & Bolton (1998) had shown to be the ultimate grading that a sand could reach by breakage in compression. The degree of breakage was quantified with the relative breakage of Hardin (1985). The data in Fig.28 highlighted that at each stress level a final degree of breakage was indeed reached, and that reaching a state of no further breakage did correspond to the cessation of the volumetric strain. However, this occurred at strain levels far beyond anything that could be reached either in a triaxial test or in typical engineering applications. The critical state we use is indeed therefore only apparent. This might be thought, however to be a slightly academic point, unless one has applications where there are very large strains, such as a debris flow or along the shaft of a driven pile.

It had often been assumed that the angle of shearing resistance would reduce as breakage made the grading finer, but with high pressure triaxial that was shown in Fig.2 to be incorrect, provided the tests were taken to adequate strains. Here, in Fig.29 we have a further confirmation that even under extreme degrees of breakage there is no degradation of ϕ' . Initially dense samples have peaks and loose no peak strengths, but after they settle down at the critical state ϕ'_{cs} there is no further change of strength.

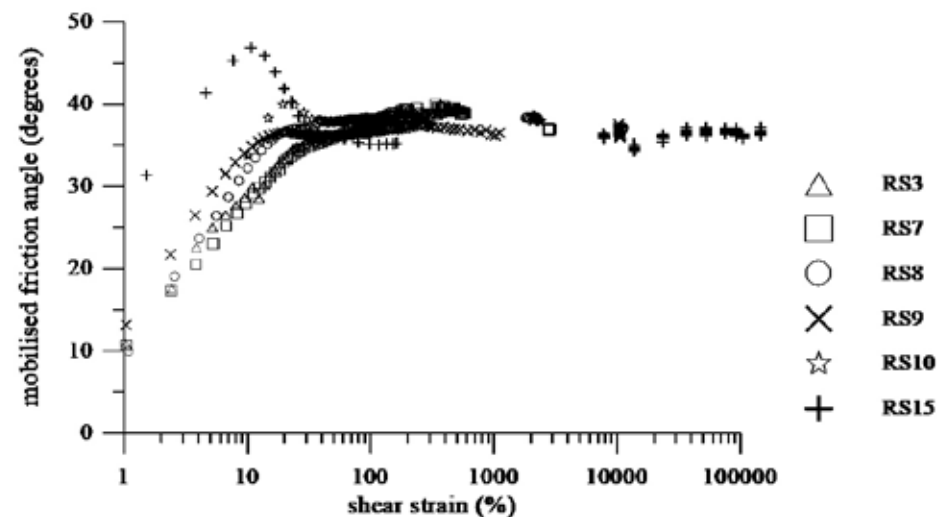


Figure 29: The evolution of mobilised angle of shearing resistance in ring shear tests on carbonate sand (Coop et al., 2004)

If sands can exist with any grading from uniform to fractal, that begs the question of how the behaviour then evolves with a change of initial grading. Following pioneering work by Altuhafi et al. (2006, 2010) on glacial sediments which become fractally graded through intense shearing beneath the glacier, Altuhafi & Coop (2011) made a systematic investigation of how the compression of sands would change with the initial grading. They did this for three sands and data for the two most extreme gradings, uniform and fractal, on carbonate sands are shown in Fig.30, but a quartz sand and a glacially deposited sand behaved in a similar fashion. The uniform sand gives a clear yield in compression and plenty of particle breakage when loaded to high stresses. The degree of breakage did, however, vary with the initial sample density, which was perhaps unexpected since the samples, after compression have the same voids ratio and are at the same stress. However, the fractally graded sand (labelled CD) shows no breakage and the compression curves are flatter, with no distinct yield. More importantly, loose and dense samples do not converge to a unique normal compression line, NCL, and exhibit a “transitional” mode of behaviour. Transitional behaviour is when a soil’s compression and shearing behaviour are dominated by the initial density the soil is formed at, and no amount of compression or shearing (within the limited strains imposed by our simple lab tests) can make their compression or shearing paths converge. While transitional soils do have unique NCLs and CSLs, those lines are unique for each initial voids ratio (see e.g. Shipton & Coop, 2015), so in fact each soil has an infinite number of NCLs and CSLs. This mode of behaviour can occur in a wide range of soils with mixed gradings and/or mixed mineralogies.

The revelation of the true extent of particle breakage in sands led to researchers questioning whether it was appropriate that they should simply have a single CSL as in Fig.2 or whether in fact they would have a new and evolving CSL as the breakage they suffer increases and the grading changes. Muir Wood (2008) proposed a model based on this assumption. When we see a unique CSL for sands at higher pressures, this model assumes that that is only because we are only testing our samples under ever increasing stress levels and we have not reduced the stresses after breakage to see if the CSL is still the same as it was for a monotonically increasing stress. The work of Altuhafi & Coop (2011) in Fig.30 might be thought to support that hypothesis, but of course these were tests done on samples with a changing initial grading, which does not address the question of whether a change to the grading during the loading history of a soil can affect the behaviour in the absence of reconstituting or thoroughly mixing the soil. Put more simply, is it the initial or the current grading of a soil that governs its behaviour, or one could ask, does the soil know that some of its particles have been broken?

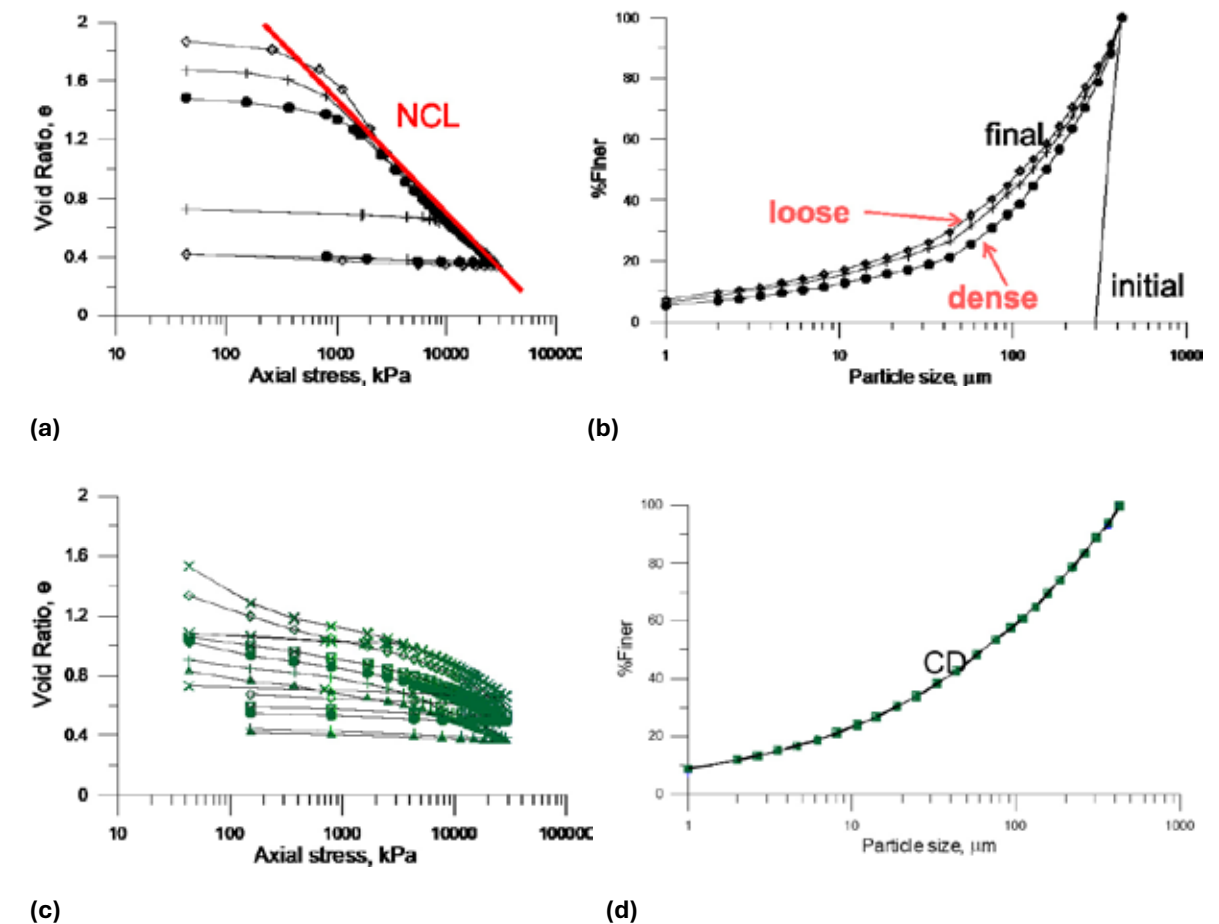


Figure 30: Changes to the oedometric compression behaviour of Dogs Bay carbonate sand with initial grading, (a) compression for uniform initial grading, (b) particle breakage for uniform initial grading, (c) compression for fractal initial grading (CD), (d) particle breakage for fractal initial grading (Altuhafi & Coop, 2011).

The test procedure to investigate this is far from easy and is illustrated by the example test in Fig.31a. As always, the Dogs Bay carbonate sand was used as an extreme case that gives plenty of particle breakage, which we would need much higher pressures to see in a quartz sand. The sample was first isotropically compressed to 1 MPa and then sheared drained at that stress level. The shearing was via a path of constant p' achieved by reducing the radial effective stress at half the rate of the increase of deviatoric stress. This path was followed simply to create some breakage in the soil, hence the elevated pressure. The constant p' path was used as it allows the critical state to be reached at lower strains compared to a standard drained shearing at constant effective cell pressure, or radial stress. Having created the breakage, the sample was unloaded under a constant effective radial stress and then extended slightly to recover the sample shape so that it did not become too barrelled. The final shearing, again constant p' , was then at 500 kPa, after which the sample was recovered and its final grading checked. Three first loading stress levels (0.5, 1 and 4 MPa) were used to create three different amounts of breakage, quantified with B_p , and it was checked that this breakage was mainly caused by the first loading and that the second loading was not contributing significantly. Each degree of breakage did indeed create a new CSL (Fig.31b) but the differences between these and the CSL of the original grading were small and an enormous amount of breakage was needed to move it significantly. So for most practical purposes we can probably ignore this problem and simply use our unique CSL

as in Fig.2, because we are unlikely to have engineering structures where we create a lot of breakage at high stresses and then need the soil properties after it has unloaded to much lower stresses.

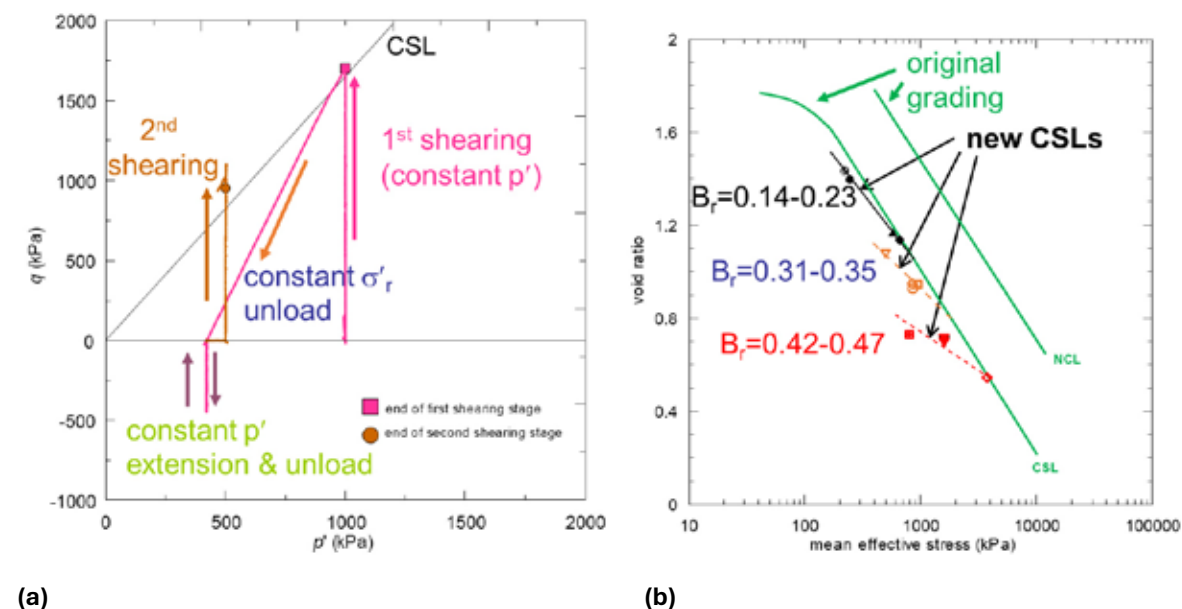


Figure 31: Changes to the CSL of Dogs Bay carbonate sand with particle breakage (a) example of test procedure, (b) new CSLs (Bandini & Coop, 2011).

6 PARTICULATE APPROACHES TO SAND BEHAVIOUR

The complexity, difficulty and confusion of the mechanics of sands arises almost solely because we insist on applying continuum mechanics to a material that is patently not continuous. We should then ask ourselves why on Earth are we torturing ourselves with this hideous mechanics and we don't adopt a discrete or particulate approach. Sands, of all soils, naturally lend themselves to discrete analyses such as DEM (discrete element modelling). The behaviour is dominated by the contact forces and there are no significant particle surface forces to worry about. DEM analyses of sands have been around for 45 years or so, but many have been simplistic as the DEM particles have been spherical, which no sand particles are. This then means that the contact parameters assumed in the analyses are artificial and they are typically found by curve fitting a DEM analysis of an element test like a triaxial with a real triaxial test. The contact parameters required in the analyses are indicated in Fig.32. The normal loading is often assumed to be linear as is the tangential loading, with stiffnesses K_N and K_T , which are then often assumed to be the same. In tangential loading, "failure" occurs when inter-particle sliding occurs at the friction coefficient μ . In normal loading, failure occurs when the particles break, but this is seldom modelled in DEM, despite the importance that we have understood it has for many sands at the continuum or macro-scale. A rolling resistance is then generally employed in analyses with spherical particles to prevent them spinning. This is mostly artificial as the true rolling resistance is likely to be very small. The twisting resistance is real but generally ignored because it is assumed to be small. In some more advanced analyses, a non-linear Hertz (1882) contact model is used, which is based on the contact between two smooth elastic spheres. This was extended to give a non-linear tangential behaviour by Mindlin & Deresiewicz (1953).

To create a perfect Avatar or digital twin of a sand, three key aspects need to be addressed: 1) modelling the correct particle shapes, 2) modelling the fabric (particle arrangements) of the prototype sand, and 3) using the correct contact mechanics. There are a plethora of methods for capturing the shape, for example overlapping spheres (Fellerec & McDowell, 2010), non-overlapping clumps of spheres (e.g. Tapias et al., 2015), polyhedrons (e.g. Tutumluer et al., 2013), potential particles (e.g. Ahmed et al., 2016), spherical harmonics (e.g. Zhao et al., 2017) or level-set DEM (Karapiperis & Andrade, (2021)). Then the fabric is fairly routinely captured using X-Ray CT (e.g. Fonseca et al., 2013). What is seldom investigated is the contact mechanics and even using realistic particle shapes the tendency is to obtain that by curve fitting triaxial test simulations. But of course curve-fitting a model with at least four parameters is fraught with problems and the values obtained have little to do with reality.

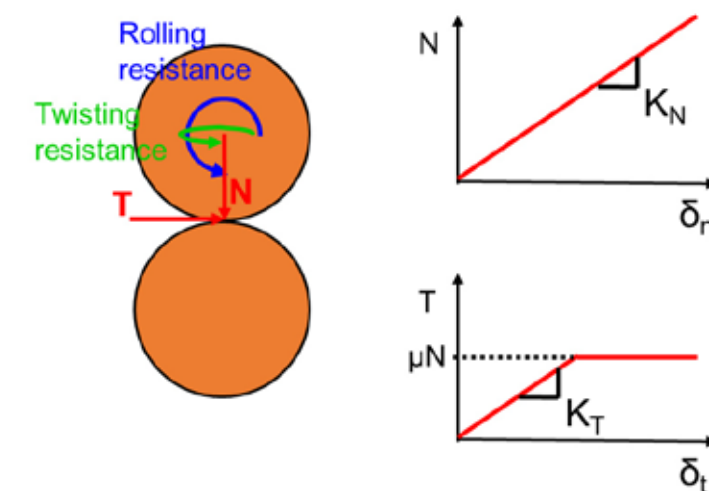


Figure 32: Simple contact models used in DEM.

Instead of being half-hearted about discrete analyses, we need to have the courage of our convictions and throw away our triaxial apparatus, but then we will need an apparatus to investigate the contact mechanics. Such an apparatus is shown in Fig.33. Going back to the 1960s researchers have investigated individual aspects of sand contact behaviour, for example Skinner (1969) who measured contact friction coefficients. This is however, the first apparatus to allow a complete investigation of the contact mechanics in normal and tangential loading, from the small displacement stiffness to failure (sliding or particle breakage). Three loading arms apply computer controlled forces and/or displacements at the contact of two particles, the lower one being mounted on a sliding sled. Each arm has a linear actuator, load cell and non-contact displacement transducer. This version of the apparatus is designed for coarse sands has a load capacity of 100 N, but can resolve forces and displacements down to 0.01 N and 0.01 μm , while a larger version for ballast and rockfill reaches 1 kN. The extreme accuracy of the displacements is necessary because soil contact behaviour is very stiff, and this accuracy can only be achieved by using a very stiff apparatus.

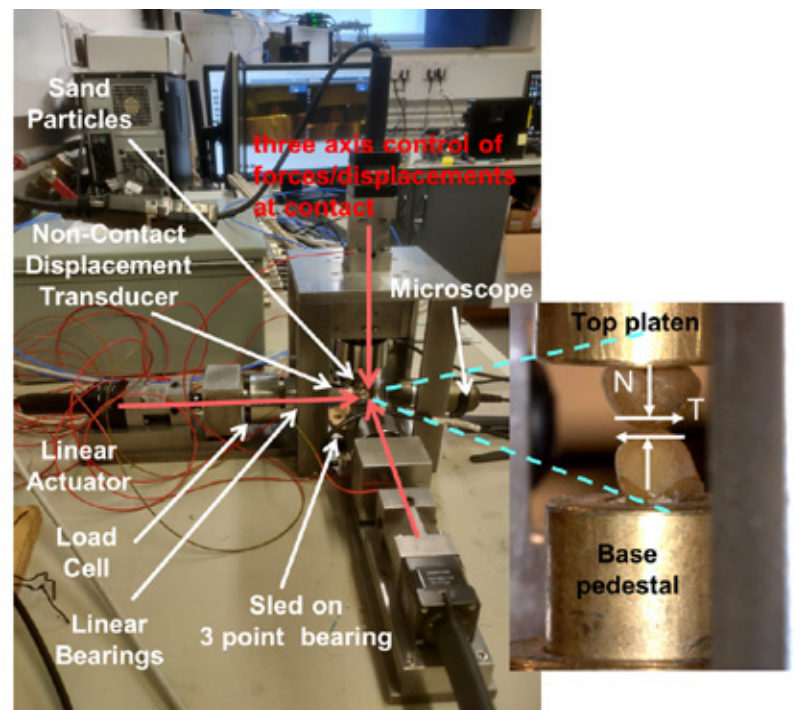


Figure 33: The inter-particle contact apparatus.

Typical normal loading responses for four sands are shown in Fig.34a. Each of the particles was in the size range 1.18–2.36mm. The differences between the responses arise partly from differences of size and the shape at the contact, but also from their different mineralogies and hence different Young's moduli of the minerals forming them. The decomposed granite sand is particularly soft because the weathering that created it left a clay coating on the particles. We can compare the normal loading behaviour with that predicted by Herz (1882):

$$\delta_N = \frac{a_H^2}{R} = \left(\frac{9N^2}{16RE^*} \right)^{1/3} \quad (1)$$

where δ_N is the normal displacement, a_H is the radius of the particle contact area, R the equivalent radius ($1/R = 1/R_1 + 1/R_2$), N the normal force and E^* the equivalent Young's modulus, taken as $1/E^* = (1-\nu_1^2)/E_1 + (1-\nu_2^2)/E_2$ in which ν_1 , E_1 , ν_2 and E_2 are the Poisson's ratios and Young's moduli of the

two contacting materials. The Poisson's ratio of quartz is 0.065–0.068 and its Young's modulus 93GPa (Mavko et al., 1998; Jaeger et al., 2007). R_1 and R_2 are the radii of the two particles in the vicinity of the contact. For highly spherical particles these may be assumed to be the particle size, but for more irregular particles they have to be measured from the microscope images of the contact (Fig.33).

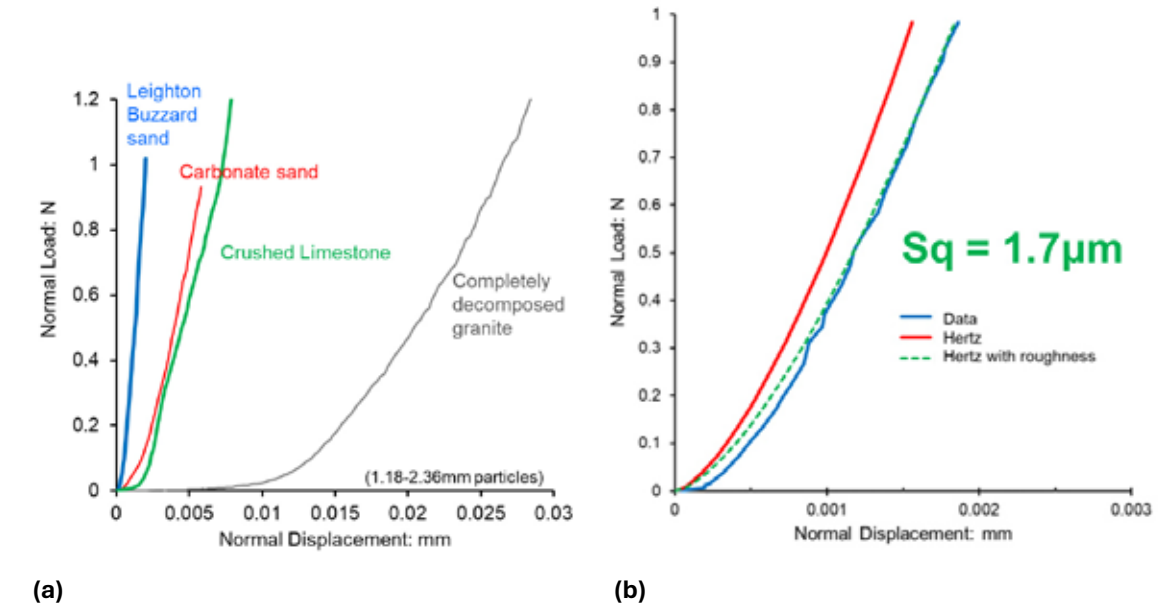


Figure 34: Normal loading behaviour of sands (a) comparison of sands of different origins, (b) comparison of a test on a quartz sand with contact models (modified from Nardelli & Coop, 2019).

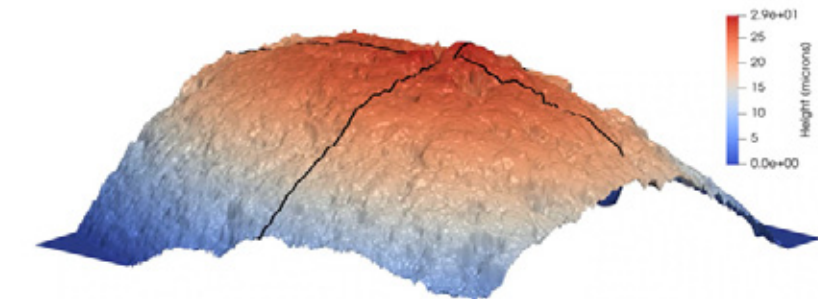


Figure 35: Laser confocal microscopy image of a sand particle (Singh, 2025)

The comparison between the data for a quartz Leighton Buzzard sand particle and Hertz in Fig.34b reveals that Hertz is a little too stiff. This is because the real particles are not smooth and have a finite roughness. This can be measured using white light interferometry or laser confocal microscopy. Figure 35 shows a typical image of a sand particle. The overall particle shape has to be removed from the image before the roughness can be calculated, which is generally quantified with a root mean square value of the deviation from a reference plane, S_q , which for the example in Fig.34b was 1.7 μm . Hertz was modified by Greenwood et al. (1984) and Yimsiri & Soga (2000) for the particle roughness. The equations are too involved to repeat in this paper and details can be found in e.g. Nardelli et al. (2019), but the agreement with the data is much better. Here, for a relatively low roughness sand, the

influence of roughness is modest, but it becomes crucial for rougher materials, for example a railway ballast (Altuhafi et al., 2023).

At larger loads there is a surprising amount of plastic deformation at particle contacts before the particles fracture. Figure 36 shows surface profiles of a Leighton Buzzard quartz sand particle measured after the application of various loads against a steel platen. These were taken with white light interferometry by Yao et al. (2022). This is not simply plastic deformation of the roughness asperities but a widespread plasticity in the region of the contact at which point all of our contact models fail to fit the data.

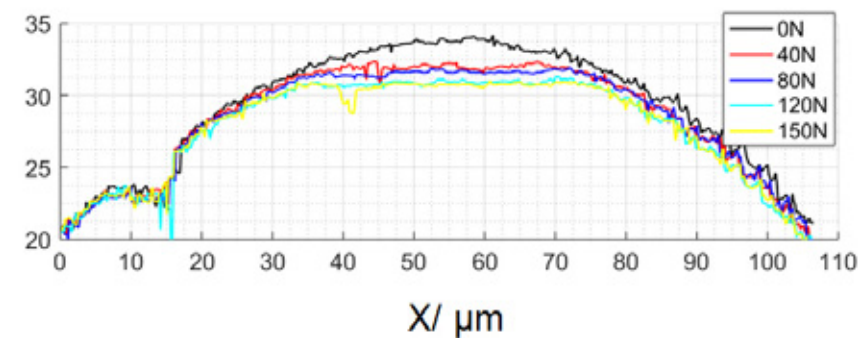


Figure 36: Surface profiles of a quartz sand particle contact deformed at larger loads (Yao et al., 2022).

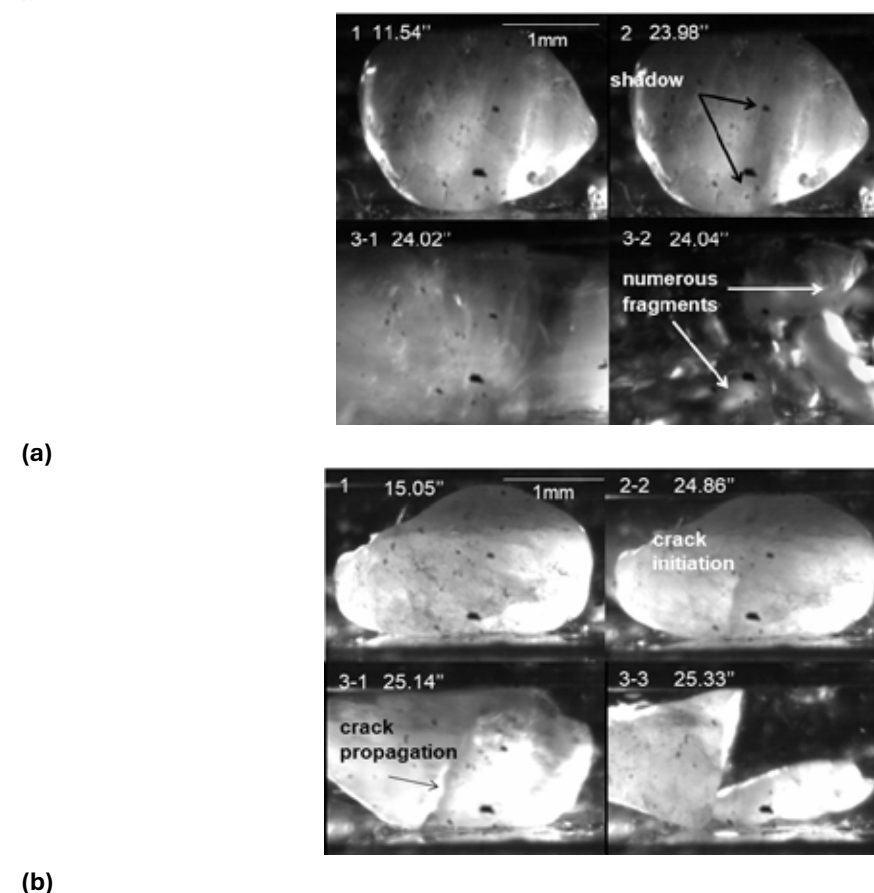


Figure 37: Breakage modes of particles of Leighton Buzzard quartz sand (a) explosive, (b) splitting (Wang & Coop, 2016).

When the particles reach failure, the two principal modes of rupture are splitting and explosive. Figure 37 shows images from a high speed camera during particle failure when loaded between two platens. The strengths of a number of quartz sand particles were measured and then, separating by mode, they were plotted against the probability of survival, i.e. the probability of a particle having a strength greater than a particular value (Fig.38). The splitting particles are generally weaker and more numerous than the explosive ones, and this corresponds to the splitting ones, on average, having greater angularity and so more stress concentration at the loading contact. In contrast the explosive particles are more rounded and store more energy leading to a more violent failure.

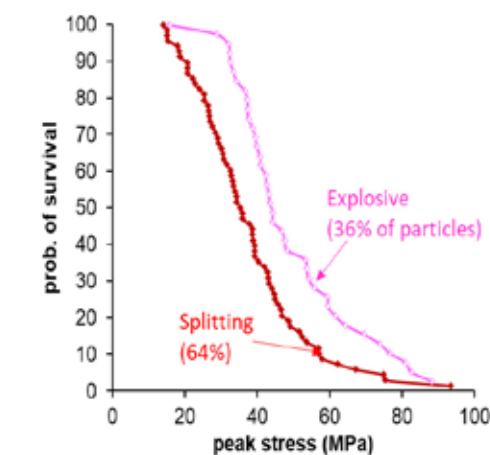


Figure 38: Strengths of Leighton Buzzard quartz sand particles (Wang & Coop, 2016).

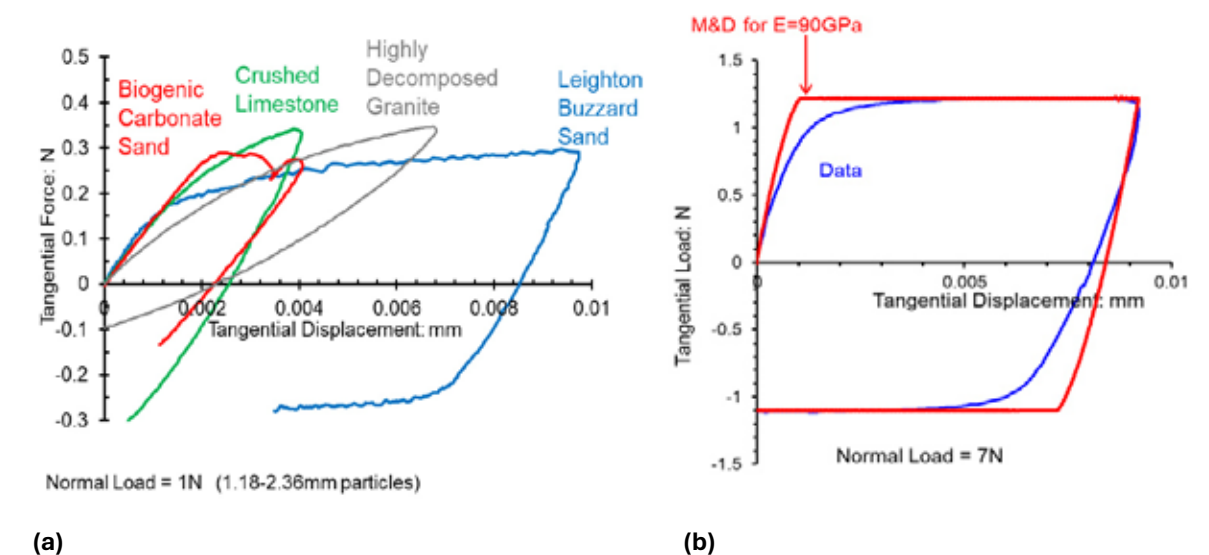


Figure 39: Tangential loading sands, (a) comparison of various sands, (b) comparison with Mindlin-Deresiewicz (modified from Nardelli & Coop, 2019).

In tangential loading of sands (Fig.39a) there is again some variability of response because of differences in contact geometry, mineralogy and surface roughness and the weathered surface of the decomposed granite again makes it considerably softer. Comparing the Mindlin & Deresiewicz (1953) predictions with the real particle contact behaviour in Fig.39b, the predictions are rather worse than that in normal loading compared to Hertz, and are consistently around a factor of 2–3 too stiff. There is no adequate model for tangential loading that accounts for roughness, but as we move to very rough

particles like railway ballasts the M&D predictions get even worse (Wong & Coop, 2023), so undoubtedly the poor predictions result from the effects of particle roughness. In Fig.40 the sliding failure envelopes are shown for various materials. While there is considerable scatter for each material there are consistent differences in the overall values of μ . These do not correlate well with the shape of the particles, since the biogenic carbonate sand, which has the highest μ , is also one of the most spherical and roundest. Instead, Fig.41 reveals that it is the particle surface roughness that controls μ .

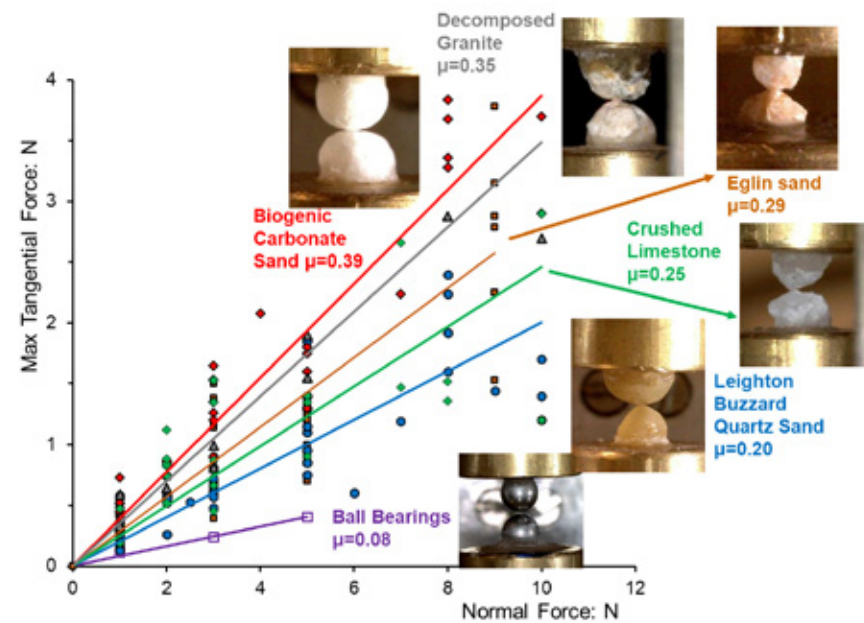


Figure 40: Tangential loading failure envelopes for various sands (modified from Nardelli & Coop, 2019).

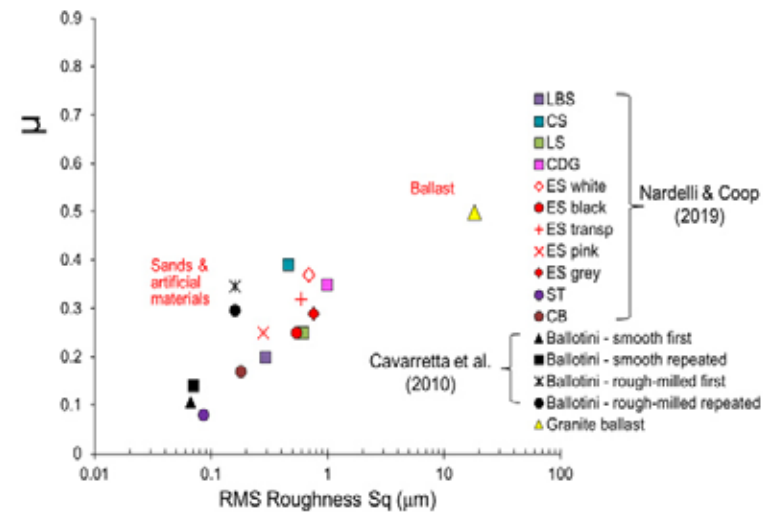


Figure 41: Coefficients of sliding friction for a range of materials (modified from Wong & Coop, 2023 and Nardelli & Coop, 2019).

The contact mechanics of sands in normal and tangential loading is therefore fairly simple to understand, even if current models are inadequate for tangential loading. However, just as in the continuum mechanics from a triaxial test, there are a range of other issues to consider beyond simple monotonic normal and tangential loading. Some of these have already been addressed, for example contact creep or maturing (Michalowski et al., 2018), rate effects (Wong & Coop, 2023), cyclic loading (Wong & Coop, 2023), the influence of the plasticity at the contacts on their behaviour (Altuhafi et al., 2024, Nardelli & Coop, 2019). Some of the aspects of behaviour we observe in laboratory tests may also prove very challenging to implement in numerical modelling, especially those relating to particle damage and breakage.

7 CONCLUDING REMARKS

Over the last few decades our understanding of the mechanics of sands has progressed immensely. In the first instance this was through the development of a complete framework within critical state soil mechanics that accounted properly for the particle damage which sands undergo at higher stresses and which related the behaviour to the in-situ state relative to reference conditions such as normal compression or critical state lines. Examples of the application of this framework to pile behaviour demonstrate its value by comparison with our crude empirical methods like relative density. Extending the framework to finer materials, even when there is no breakage, allows us to understand phenomena like static liquefaction in tailings, with all its deadly consequences. However, as the decades have progressed, the aspects of behaviour that we might account for in our modelling and design have continued to multiply, so that we have generally now given up any hope of ever having a all-encompassing constitutive model for sands. Instead, engineers pick their model off the shelf according to which aspect of behaviour they believe might be most important for their application, volume change, small strain stiffness, anisotropy etc. etc. Many of these are so complex that they are in any case ignored by industry. More recent work has also highlighted that much of our cherished critical state framework was in any case a chimera that turned out to be convenient for most applications, but not strictly accurate, with the discovery of features such as continued particle damage well beyond the engineering strain level and transitional behaviour for more complex gradings and mineralogies.

Our better understanding of the continuum mechanics of sands, from element tests such as triaxials, comes largely from a better appreciation of what is happening at the particulate scale, such as particle breakage or fabric, so one might rightly ask why we bother chasing our tails with ever more complex models at the macro-scale and why we don't just embrace the particulate nature of soils and adopt a fully micromechanical methodology. This could be the point of paradigm change, since the existing paradigm seems to have run its course, and it is important that we do not miss that opportunity.

In June 1950 there was the defining London conference on “The Measurement of the Shear Strength of Soils” at which Bishop, amongst others, laid out the basics of modern triaxial testing apparatus and methodology. Even then there were bitter feuds with advocates of alternative, now long forgotten apparatus and methods. We are now at a similar position with micromechanics. While we are easily able now to quantify and model particle shapes and soil fabrics, we know much less about the key input parameters for discrete analyses, the contact mechanics. The apparatus to develop them are in a relatively crude and early stage of development and doubtless the coming decades will see wider interest and participation in this research, with improved apparatus. There will, no doubt, be much complication still to come in particulate mechanics, but it is very unlikely that it ever reaches the byzantine complexity of continuum mechanics. Indeed how could it? Any continuum model must have all the particle scale complexity embedded within it. The key question facing geotechnical engineering is whether the future is discrete or continuous. While for clays here must be considerable doubt that it could ever be fully discrete, and even for sands we might worry about whether computing power will ever be large enough for practical design, we will not find out if we don't try.

ACKNOWLEDGEMENTS

Some of this work was funded by UK EPSRC grants Nos. EP/S026460/1 and EP/W000563/1. Amongst the many co-workers who have contributed to this work over the years, the author especially indebted to Professor Béatrice Baudet.

NOMENCLATURE

a_H radius of particle contact area
 B_r relative breakage
 D_r relative desnity

e	void ratio
$e_{\max}$	maximum e
$e_{\min}$	minimum e
E^*	equivalent Young's modulus
E_1, E_2	Young's moduli for particles 1 and 2 of contact
G	shear modulus
G_0	elastic shear modulus
$G_{0(nc)}$	elastic shear modulus for a sample on the NCL
I_B	brittleness index = $(q_{\max} - q_{cs})/q_{\max}$
K_N	normal contact stiffness
K_T	tangential contact stiffness
M	critical state line gradient, q'/p'
N	normal contact force
N_q	pile bearing capacity factor for end bearing ($=q_b/\sigma'_v$)
p'	mean normal effective stress $(s'_a + 2s'_r)/3$ or $(s'_v + 2s'_h)/3$
p'_{cs}	equivalent pressure take on CSL in e : $\ln p'$ plane at current e (or v)
p'_e	equivalent pressure take on NCL in e : $\ln p'$ plane at current e (or v)
q'	deviator stress $\sigma'_a - \sigma'_r$
q_b	pile unit base resistance
q_{cs}	q at critical state
$q_{\max}$	peak q in stress-strain curve
q_s	pile unit shaft resistance
R	equivalent radius
R_1, R_2	radii of particles 1 and 2 at contact
R_s	stress state parameter ($=p'/p'_{cs}$)
S_q	RMS roughness
T	tangential contact force
v	specific volume $=1+e$
v_{cs}	value of v on the CSL at current p'
v_{liq}	v above which a soil liquefies on shearing
$v_{\max}$	maximum v during sample preparation
$v_{\min}$	minimum v during sample preparation
v_s	shear wave velocity
β	pile shaft friction coefficient ($=q_s/\sigma'_v$)
δ_N	normal contact displacement
δ_T	tangential contact displacement
ϕ'_{cs}	critical state angle of shearing resistance
μ	coefficient of contact friction
ν_1, ν_2	Poisson's ratios for particles 1 and 2 of contact
ρ	mass density
σ'_a	axial effective stress
σ'_h	horizontal effective stress
σ'_r	radial effective stress
σ'_v	vertical effective stress
ψ	state parameter (difference between current e and that on CSL at current p')
ψ_m	modified state parameter

CSL Critical State Line
DEM discrete element analysis
NCL Normal compression line

REFERENCES

- [1] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, “State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering,” *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Ahmed, S., Harkness, J., Le Pen, L., Powrie, W. & Zervos, A. (2016). Numerical modelling of railway ballast at the particle scale. *Int. J. Numer. Anal. Meth. Geomech.*, 40, 713–73.
- [2] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, “State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering,” *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Altuhafi, F. N., Baudet, B. A. & Coop, M. R. (2023) An investigation of the applicability of contact models to the normal load-deformation behaviour of artificially shaped granite. *Acta Geotechnica*. 16 pp.
- [3] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, “State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering,” *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Altuhafi, F.N., Baudet, B.A. & Coop, M.R. (2024) Plastic deformation effect on contact behaviour in granular materials. *Granular Matter*. 26(2), Article No., 34.
- [4] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, “State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering,” *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Altuhafi, F., Baudet, B. A. & Sammonds, P. (2006). Particle breakage in glacial sediments. In *Geomechanics and geotechnics of particulate media* (eds M. Hyodo, H. Murata and Y. Nakata), London: Taylor & Francis. 21–24.
- [5] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, “State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering,” *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Altuhafi, F. N., Baudet, B. A. & Sammonds, P. (2010). The mechanics of subglacial sediment: an example of new ‘transitional’ behaviour. *Can. Geo. J.* 47(7), 775–790.
- [6] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, “State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering,” *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Altuhafi, F. & Coop, M.R. (2011) Changes to particle characteristics associated with the compression of sands. *Géotechnique* 61(6), 459-471.
- [7] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, “State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering,” *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Atkinson, J.H. (2000) Non-linear soil stiffness in routine design. *Géotechnique*, 50(5), 487-507
- [8] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, “State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering,” *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Bandini, V. & Coop, M.R. (2011) The influence of particle breakage on the location of the critical state line of sands. *Soils & Foundations* 51(4), 591-600.
- [9] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, “State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering,” *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Bedin, J., Schnaid, F., da Fonseca, A. V., & de Costa Filho, L. M. (2012). Gold tailings liquefaction under critical state soil mechanics. *Géotechnique*, 62(3), 263-267.
- [10] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, “State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering,” *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Been, K. and Jefferies, M.G., (1985) A state parameter for sands. *Géotechnique*, 35(2), 99-112.
- [11] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, “State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering,” *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Bishop, A. W., Green, G. E., Garga, V. K., Andresen, A. & Brown, J. D. (1971). A new ring shear apparatus and its application to the measurement of residual strength. *Géotechnique* 21(4), 273–328.
- [12] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, “State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering,” *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Bobei, D. C., Lo, S. R., Wanatowski, D., Gnanendran, C. T., & Rahman, M. M. (2009). Modified state parameter for characterizing static liquefaction of sand with fines. *Canadian Geotec. J.*, 46(3).
- [13] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, “State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering,” *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Carrera, A., Coop, M. R., & Lancellotta, R. (2011). Influence of grading on the mechanical behaviour of Stava tailings. *Géotechnique*, 61(11), 935-946.
- [14] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, “State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering,” *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Cartwright, A., Coop, M.R. & Fourie, A. (2025) Cyclic Liquefaction in Transitional and Non-Transitional Tailings, *Géotechnique*, 75 (4), 446–458.
- [15] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, “State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering,” *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Chandler, H. W. (1985). A plasticity theory without Drucker’s postulate, suitable for granular materials. *J. Mech. Phys. Solids*, 33, 215–226.
- [16] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, “State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering,” *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Chang, N., Heymann, G., & Clayton, C. (2011). The Effect of Fabric on the Behaviour of Gold Tailings. *Géotechnique*, 61(3), 187-197.
- [17] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, “State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering,” *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Colombo, P. & Colleselli, F. (2003). The collapse of the Stava tailings basins. Analysis of the event and observations. In: G. Tosatti (ed.) “A review of scientific contributions on the Stava Valley disaster (Eastern Italian Alps), 19th July 1985”, Pitagora, Bologna: 335-358.
- [18] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, “State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering,” *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Coop, M.R. (1990) The mechanics of uncemented carbonate sands. *Géotechnique*, 40(4), 607-626.
- [19] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, “State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering,” *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Coop, M.R. & Lee, I.K. (1993) The behaviour of granular soils at elevated stresses. *Predictive Soil Mechanics*, Proc. C.P.Wroth Mem Symp., Thomas Telford, London, 186-198.
- [20] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, “State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering,” *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Coop, M.R., Sorensen, K.K., Bodas Freitas, T. & Georgoutsos, G. (2004) Particle breakage during shearing of a carbonate sand. *Géotechnique* 54(3), 157-163.
- [21] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, “State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering,” *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Coop, M.R., Klotz, E.U. & Clinton, L. (2005) The influence of the in situ state of sands on the load-deflection behaviour of driven piles. *Géotechnique* 55(10), 721-730.
- [22] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, “State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering,” *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Cotecchia, F., Santaloia, F., Lagioia, R. & Coop,

- M.R. (1998) An investigation of the behaviour of bored piles in pyroclastic sands. Int. Symp. on Problematic Soils, Yanagisawa, Japan, Moroto & Mitachi (eds), Balkema, Rotterdam, 715-719.
- [23] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, “State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering,” *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Dolwin, J., Khorshid, M.S. & van Goudoever, P. (1988) Evaluation of driven pile capacity – Methods and results. Proc. Int. Conf. Calcareous Sediments, Perth, (1), 429-438.
- [24] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, “State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering,” *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Dos Santos, C.P., Bruschi, G.J., Mattos, J.R.G. & Consoli, N.C. (2022) Stabilization of gold mining tailings with alkali-activated carbide lime and sugarcane bagasse ash. Transportation Geotechnics, 32, 100704.
- [25] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, “State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering,” *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Fellerec, J. & McDowell, G. (2010) Modelling realistic shape and particle inertia in DEM. Géotechnique, 60(3), 227-232.
- [26] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, “State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering,” *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Fonseca, J., O’Sullivan, C., Coop, M.R. & Lee, P.D. (2013) Quantifying the Evolution of Soil Fabric during Shearing using Directional Parameters. Géotechnique 63(6), 487-499.
- [27] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, “State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering,” *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Franks, M., Stringer, M., Torres-Cruz, L.A., Baker, E. Valenta, R., Thygesen, K. Matthews, A., Howchin, J. & Barrie, S. (2021) Tailings facility disclosures reveal stability risks. Scientific Reports vol. 11, Article number: 5353.
- [28] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, “State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering,” *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Greenwood, J. A., Johnson, K. L. & Matsubara, E. (1984). A surface roughness parameter in Hertz contact. Wear 100, No. 1–3, 47–57.
- [29] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, “State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering,” *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Hardin, B. O. (1985). Crushing of soil particles. J. Geotech. Engng, Proc. ASCE 111(10), 1177–1192.
- [30] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, “State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering,” *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Hertz, H. (1882). Ueber die Berührung fester elastischer Körper. Journal für die reine und angewandte Mathematik, 92, 156-171.
- [31] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, “State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering,” *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Hight, D.W., Bond, A.J. & Legge, I.D. Characterization of the Bothkennar clay – an overview. Géotechnique 42(2), 303-347.
- [32] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, “State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering,” *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Jaeger, J. C., Cook, N. G. W. & Zimmerman, R. (2007). Fundamentals of rock mechanics, 4th edn. Oxford, UK: Wiley-Blackwell.
- [33] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, “State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering,” *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Jovicic, V. & Coop, M.R. (1997). Stiffness of coarse grained soils at small strains. Géotechnique, 47(3), 545-561.
- [34] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, “State-of-the-art review on the use of AI-enhanced

- computational mechanics in geotechnical engineering,” *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Karapiperis K. & Andrade, J.E. (2021) Nonlocality in granular complex networks: Linking topology, kinematics and forces. Extreme Mechanics Letters, 42 (2021) 101041.
- [35] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, “State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering,” *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Kendall, K. (1978). The impossibility of comminuting small particles by compression. Nature, 272, 710–711.
- [36] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, “State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering,” *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Khorshid, M.S., Haggerty, B.C. & Male, R. (1988) Development of geotechnical aspects of the investigation programme. Proc. Int. Conf. Calcareous Sediments, Perth, (2), 377-386.
- [37] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, “State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering,” *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Klotz, E.U. & Coop, M.R. (2001) An investigation of the effect of soil state on the capacity of driven piles in sands. Géotechnique, 51(9), 733-751.
- [38] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, “State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering,” *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Lehane, B.M., Jardine, R.J., Bond, A.J. & Frank, R. (1993) Mechanisms of shaft friction in sand from instrumented pile tests. Proc. ASCE, Journal Geotech. Engng, 119(GT1), 19-35.
- [39] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, “State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering,” *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Li, W., & Coop M. R. (2019) The mechanical behaviour of Panzhihua iron tailings. Canadian Geotech. J., 56(3), 420-435.
- [40] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, “State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering,” *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Li, W., Coop M. R., Senetakis K., & Schnaid F. (2018) The mechanics of a silt-sized gold tailing. Engineering Geology. 241, 97-108.
- [41] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, “State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering,” *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. McDowell, G. R. & Bolton, M. D. (1998). On the micro mechanics of crushable aggregates. Géotechnique 48(5), 667–679.
- [42] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, “State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering,” *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Mavko, G., Mukerji, T. & Dvorkin, J. (1998). The rock physics handbook: tools for seismic analysis in porous media. Cambridge, UK: Cambridge University Press.
- [43] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, “State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering,” *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Michalowski, R.L., Wang, Z. & Nadukuru, S.S. (2018). Maturing of contacts and ageing of silica sand. Géotechnique, 68(2), 133–145.
- [44] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, “State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering,” *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Mindlin, R. D. & Deresiewicz, H. (1953). Elastic spheres in contact under varying oblique forces. Trans. ASME, J. Appl. Mech., 20, 327–343.
- [45] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, “State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering,” *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Muir Wood, D. (2008) Critical states and soil modelling, Deformational Characteristics of Geomaterials, 1, IOS Amsterdam, 51–72.

- [46] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, "State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering," *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Nardelli, V. & Coop, M.R. (2019). The experimental contact behaviour of natural sands: normal and tangential loading. *Géotechnique*, 69(8), 672-686.
- [47] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, "State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering," *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Nauroy, J.F. & Le Tirant, P. (1985) Driven and drilled and grouted piles in calcareous sands. Offshore Tech. Conf., Paper No.OTC 4850, 83-91.
- [48] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, "State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering," *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Nicholson, D., Chapman, T. & Morrison, P. (2002). Pressuremeter proves its worth in London's Docklands. *Ground Engng*, 35(3), 32-34.
- [49] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, "State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering," *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Qi, C. & Fourie, A. (2019) Cemented paste backfill for mineral tailings management: Review and future perspectives. *Minerals Engineering*, vol. 144, Dec. 2019, 106025.
- [50] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, "State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering," *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Renfrey, G.E., Waterton, C.A. & van Goudoever, P. (1988). Geotechnical data used for the design of the North Rankin 'A' platform. *Engineering for calcareous sediments*, R.J.Jewell & M.S.Khorshid eds., Balkema, Rotterdam, Vol.2, 343-355.
- [51] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, "State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering," *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Ripley, I., Keulers, A.J.C. & Creed, S.G. (1988) Conductor load tests. *Proc. Int. Conf. Calcareous Sediments*, Perth, (2), 429-438.
- [52] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, "State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering," *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Senders, M., Banimahd, M., Zhang, T. & Lane, A. (2013) Piled foundations on the north-west shelf. *Australian Geomechanics*, 48(4), 149-160.
- [53] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, "State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering," *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Servi, S, Lotero, A., Silva, J.P.S., Bastos, C. & Consoli, N.C. (2022). Mechanical response of filtered and compacted iron ore tailings with different cementing agents: Focus on tailings-binder mixtures disposal by stacking. *Construction and Building Materials*, 349, 128770.
- [54] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, "State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering," *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Singh, S. (2025) Personal communication.
- [55] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, "State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering," *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Shipton, B. & Coop, M.R. (2015) Transitional Behaviour in Sands with Plastic and Non-plastic Fines. *Soils & Foundations* 55(1), 1-16.
- [56] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, "State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering," *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Skinner, A. E. (1969). A note on the influence of interparticle friction on shearing strength of a random assembly of spherical particles. *Géotechnique*, 19(1), 150-157.
- [57] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, "State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering," *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul.

- 2024, doi: 10.1007/s10462-024-10836-w. Tapias, M., Alonso, E. E. & Gili, J. (2015) A particle model for rockfill behaviour. *Géotechnique*, 65(12), 975-994.
- [58] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, "State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering," *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Tutumluer, E., Qian, Y., Hashash, Y. M. A., Ghaboussi, J. & Davis, D.D. (2013). Discrete element modelling of ballasted track deformation behaviour. *Int. J. Rail Transportation*. 1(1-2): 57-73.
- [59] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, "State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering," *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Velten, R.Z., Ulsen, C., Rodrigues da Costa, J.P., Lima, C.A.A., Mafessoli, M., Carvalho, J.V.A. & Consoli, N.C. (2023). Role of mineralogy on the undrained monotonic simple shear response of compacted filtered copper tailings. *Soils and Foundations* 65, 101636.
- [60] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, "State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering," *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Ventouras, K. & Coop, M. R. (2009). On the behaviour of Thanet Sand: an example of an uncemented natural sand. *Géotechnique*, 59(9), 727-738.
- [61] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, "State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering," *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Vilhar, G., Jovicic, V & Coop, M.R. (2013) The role of particle breakage in the mechanics of a non-plastic silty sand. *Soils & Foundations* 53(1), 91-104.
- [62] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, "State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering," *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Wang, W. & Coop, M.R. (2016) An investigation of breakage behaviour of single sand particles using a high-speed microscope camera. *Géotechnique*, 66(12), 984-998.
- [63] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, "State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering," *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. WISE (2021) World information service on energy.
- [64] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, "State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering," *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Wong, C.P.Y. & Coop, M.R. (2025) The Contact Mechanics of a UK Railway Ballast. *Géotechnique*, 74 (13), 700-1712.
- [65] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, "State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering," *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Yao, T., Baudet, B. A. & Lourenço, S. D. N. (2022). Evolution of surface roughness of single sand grains with normal loading. *Géotechnique*, 72(6), 543-555.
- [66] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, "State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering," *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Yasufuku, N., Ochiai, H. & Maeda, Y. (1997) Geotechnical analysis of skin friction of cast-in-place piles. *Proc. 14th ICSMFE*, (2), 921-924.
- [67] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, "State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering," *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Yimsiri, S., & Soga, K. (2000). Micromechanics-based stress-strain behaviour of soils at small strains. *Géotechnique*, 50(5), 559-571.
- [68] H. Liu, H. Su, L. Sun, and D. Dias-da-Costa, "State-of-the-art review on the use of AI-enhanced computational mechanics in geotechnical engineering," *Artif Intell Rev*, vol. 57, no. 8, p. 196, Jul. 2024, doi: 10.1007/s10462-024-10836-w. Zhao, B. D., Wei, D. H. & Wang, J. F. (2017) Particle shape quantification using rotation-invariant spherical harmonic analysis. *Géotechnique Letters*, 7, 190-196.



ROK VARGA

Rok Varga, rojen 17. maja 1993 v Murski Soboti, je po zaključku gimnazije v domačem kraju leta 2012 začel študij na Fakulteti za gradbeništvo, prometno inženirstvo in arhitekturo Univerze v Mariboru (UM FGPA). Kot prejemnik zlate štipendije Mestne občine Murska Sobota je že med študijem pridobil praktične izkušnje v gradbenih podjetjih, med drugim v Pomgradu d. d., Imo Real in Arcontu d. d.

Diplomsko delo s področja gradbene fizike je zagovarjal leta 2017, nato pa študij nadaljeval na magistrskem programu gradbeništva. Leta 2020 je pod mentorstvom red. prof. dr. Bojana Žlendra magistriral iz geotehnike.

Po magistrskem študiju je kot mladi raziskovalec nadaljeval akademsko pot na Katedri za geotehniko UM FGPA. Pod mentorstvom izr. prof. dr. Primoža Jelušiča se je posvetil raziskavam na področju optimizacije geotehničnih konstrukcij in verjetnostnih analiz, rezultate pa je objavljajl v znanstvenih revijah s faktorjem vpliva (JCR) ter predstavljal na mednarodnih konferencah.

Leta 2024 je uspešno zaključil doktorski študij in se zaposlil kot asistent z doktoratom na Katedri za geotehniko UM FGPA. Pedagoško sodeluje pri vajah iz mehanike zemljin in temeljenja, hkrati pa nadaljuje raziskovalno delo na področju optimizacije geotehničnih konstrukcij.

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OPTIMALNA ZASNOVA GEOTERMALNIH PILOTOV

Povzetek

V članku je predstavljena študija optimizacije trenjskih in stoječih geotermalnih pilotov pri stalnih vertikalnih obremenitvah 1 MN in 4 MN ter v razponu togosti zemljin z modulom elastičnosti med 4 MPa in 400 MPa. Z analizo različnih mejnih vrednosti posedka ($0,1D$; $0,05D$; $0,01D$) je bilo ugotovljeno, v katerih pogojih ima vpliv temperature ključno vlogo pri doseganju mejnega stanja uporabnosti. Rezultati kažejo, da postane vpliv termičnih obremenitev kritičen predvsem pri večjih obremenitvah (4 MN) v kombinaciji z zemljinami z nizko togostjo ($E_{soil} < 15$ MPa). Pri še večjih obremenitvah (≥ 8 MN) pa so lahko kritične tudi zemljine z modulom elastičnosti do 40 MPa. V vseh analiziranih primerih je bil termični posedek pomemben, pri strožjih omejitvah posedka pa je predstavljal najmanj 40 % celotnega posedka. Pri določenih pogojih je vpliv spremembe temperature prispevalo tudi k povečanju napetosti v armiranobetonskem prerezu. Analiza stroškov je pokazala, da so stoječi piloti stroškovno učinkoviti za integracijo geotermalnega sistema v vseh vrstah zemljin. Pri trenjskih pilotih pa se zaradi večjih dimenzij in posledično višjih stroškov pojavijo večje razlike med konvencionalnimi in geotermalnimi piloti, za kar je potrebna dodatna obravnava učinkovitosti geotermalnega pilota in energetskih potreb konstrukcije.

Ključne besede:

stroškovna optimizacija, geotermalne geokonstrukcije, geotermalni piloti, optimalne zasnove.

OPTIMAL DESIGN OF GEOTHERMAL PILES

Abstract

The article presents an optimization study of floating and end-bearing geothermal piles subjected to constant vertical loads of 1 MN and 4 MN, across a range of soil stiffness with elastic modulus values between 4 MPa and 400 MPa. By analysing various settlement limits ($0.1D$, $0.05D$, $0.01D$), the study identifies the conditions under which temperature effects play a critical role in satisfying serviceability limit states. The results show that thermal loading becomes particularly significant under higher loads (4 MN) and in soils with low stiffness ($E_{soil} < 15$ MPa), and in some cases up to 40 MPa. In all analysed cases, thermal settlement was found to be substantial, and with stricter settlement limitations the contribution was at least 40% of the total settlement. In some instances, we also observed increasing in the stress within the reinforced concrete cross-section. A cost analysis revealed that end-bearing piles are economically viable for geothermal system integration across all soil types. In contrast, floating piles exhibited greater cost variability due to their larger dimensions, indicating a need for further calculations on the efficiency and energy needs of the structure.

Key words:

cost optimisation, geothermal geostructures, geothermal piles, optimal designs.

1 UVOD

Skupni cilj Evropske unije (EU) je povečati delež obnovljivih virov energije do leta 2030 na 32 % (Direktiva EU 2018/2001), pri čemer pa novejša Direktiva EU 2023/2413 to ambicijo še nadgrajuje na 45 % [1]. Med pomembne in dostopne obnovljive vire v Sloveniji spada poleg sončne, vetrne in biomase tudi geotermalna energija, ki je zaradi svojih naravnih značilnosti izjemno primerna za dolgoročno in stabilno oskrbo z energijo. Geotermalna energija je neizčrpen, stalen in lokalno dostopen vir, ki ni odvisen od zunanjih dejavnikov, kot so sončno obsevanje, hitrost vetra ali količina padavin. Ker je vir energije pod površjem Zemlje, nanj ne vplivajo sezonska nihanja ali vremenske spremembe. To pomeni, da lahko naprave za izkoriščanje geotermalne energije obratujejo z visoko učinkovitostjo skozi vse leto.

Zaradi teh lastnosti je pridobivanje toplotne ali električne energije iz plitve geotermalne energije izredno stabilno. To potrjuje tudi obsežna študija, izvedena v Washingtonu, kjer so raziskovalci ugotovili, da je faktor obratovanja geotermalne energije kar 95 %, kar pomeni, da sistem deluje ves čas znotraj svojega projektiranega potenciala. Za primerjavo – sončna energija dosega v povprečju faktor obratovanja le 10–25 %, vetrna pa okoli 25–45 %, saj je proizvodnja energije v teh primerih odvisna od naravnih, nepredvidljivih pojavov [2]. Takšne razlike pomembno vplivajo na dolgoročno zanesljivost oskrbe, predvsem v regijah, kjer stabilnost geotermalnih virov pomeni večjo varnost in manjšo potrebo po shranjevalnih sistemih ali rezervnih virih energije.

Poleg stabilnosti pa ima geotermalna energija še druge prednosti, ki jo postavljajo v ospredje pri trajnostnem načrtovanju mest in infrastrukture. Mednje spadajo nizek ogljični odtis, možnost vključevanja v obstoječo infrastrukturo ter dolgoročno nižji stroški obratovanja v primerjavi s fosilnimi gorivi. Zaradi teh koristi je plitva geotermalna energija zelo primerna tudi za uporabo v okviru tako imenovanih geotermalnih geokonstrukcij – konstrukcijskih elementov, kot so temelji, piloti in stene, ki poleg svoje nosilne funkcije vključujejo sistem cevi za izmenjavo toplote z zemljino.

Z namenom celostne ocene potenciala tovrstnih rešitev je bila izvedena SOAR analiza (Slika 1), ki z uporabo elementov strateškega načrtovanja ocenjuje projekt glede na njegove prednosti (Strengths), priložnosti (Opportunities), prizadevanja (Aspirations) in pričakovane rezultate (Results) uporabe geotermalnih geokonstrukcij. Takšna analiza omogoča vpogled v dolgoročni razvojni potencial in prispeva k oblikovanju konkretnih strateških ciljev v okviru trajnostne gradnje in energetike. Poleg tega pa nudi tudi podlago za vključevanje geotermalnih tehnologij v lokalne in nacionalne geotermalne strategije, skladno s prej omenjenimi evropskimi cilji.

S	Prednosti	O	Priložnosti	A	Prizadevanja	R	Rezultati
1	Združitev nosilne konstrukcije z obnovljivo energijo	1	Subvencije za obnovljive vire energije	1	Spodbujanje širše uporabe energetskih geokonstrukcij	1	Znižani stroški energije
2	Zmanjšanje ogljičnega odtisa	2	Tehnološki napredek za učinkovitejše toplotne izmenjave	2	Energetska neodvisnost	2	Zmanjšanje emisij ogljika
3	Nižji stroški življenjskega cikla	3	Interdisciplinarno sodelovanje	3	Zmanjšanje negotovosti pri zasnovi tovrstnih konstrukcij	3	Izboljšati javno percepcijo o kombiniranih geokonstrukcijah
4	Uporabno za več geokonstrukcij	4	Trajnostno bivanje	4	Doseči neto ničelne emisije ogljika	4	Odpornost na energetski trg

Slika 1: SOAR analiza, kot podlaga za celovito vključevanje geotermalnih geokonstrukcij v sodobno gradbeno prakso.

Zaradi geološke in hidrogeološke raznolikosti ima slovensko ozemlje velik potencial za izrabo plitve in globoke geotermalne energije. Slovenija leži na stičišču različnih geoloških enot, kar omogoča široko razpoložljivost vodonosnikov, primernih za geotermalni izkoristek, še posebej v vzhodni in severovzhodni Sloveniji. V ta namen so bile že izdelane in posodobljene smernice za vrtanje v plitvi geotermalni energiji, ki urejajo zajetja plitve geotermalne energije do globine 300 m in se nanašajo na odprte in zaprte sisteme izrabe [3].

V odprtem sistemu geotermalne izmenjave se toplota prenaša neposredno preko črpanja podzemne vode iz vodonosnika. Ta voda se nato vodi skozi toplotni izmenjevalnik in se – po odvzemu toplote – vrača v isti vodonosnik. Prednost takega sistema je visoka učinkovitost izmenjave toplote zaradi neposrednega stika s toplim virom (voda), slabost pa kompleksnejša izvedba in potreba po dovoljenjih zaradi rabe vodnega vira. Odprti sistemi so posebej primerni tam, kjer je vodonosnik bogat in stalno obnovljiv ter ima ugodno temperaturo in kemično sestavo. Zaradi svoje narave ti sistemi zahtevajo natančne hidrogeološke raziskave in pogosto vključujejo dve vrtini – črpalno in reinjekcijsko [4]

Nasprotno pa zaprti sistemi ne uporabljajo podzemne vode neposredno, temveč se toplota odvzema ali oddaja preko kroženja tekočine v zaprti cevni zanki, ki je vgrajena v zemljo – to so t. i. geosonde. Lahko pa je cevna zanka vgrajena tudi v geotehnično konstrukcijo (pilot, plošča, diafragma, predor). Toplotni prenosnik (npr. mešanica vode in glikola) kroži v cevnem sistemu in prenaša toploto med zemljo in toplotno črpalko. Ta sistem nima neposrednega stika z naravnim okoljem in zato ne vpliva neposredno na vodonosnike, kar poenostavlja izvedbo in dovoljenja, še posebej v urbanih okoljih ali na območjih z omejenimi vodnimi viri [3].

V Sloveniji se investitorji kljub visokemu potencialu še vedno pogosto odločajo za ločene izvedbe sistemov za izrabo geotermalne energije – to pomeni ločeno izvedbo temeljev (npr. piloti, plošča) in vrtin za toplotne črpalke. Torej, kljub številnim tehničnim in ekonomskim prednostim širša uporaba tovrstnih rešitev še vedno naleti na ovire, kot so nepoznavanje tehnologije, nejasna zakonodaja, višji začetni stroški ter odsotnost spodbud za integracijo takšnih sistemov v zgodnje faze projektiranja.

Za dolgoročni trajnostni prehod bi bilo torej smiselno spodbujati integracijo geotermalne energije neposredno v konstrukcijo objektov, s poudarkom na plitvi geotermalni energiji, ki je tehnično dostopna večini slovenskega ozemlja in v večini primerov ne zahteva večjih dovoljenj. Uporaba obstoječih gradbenih elementov kot del geotermalnega sistema pomeni pomemben korak k zmanjšanju naložbenih stroškov in povečanju geotermalne učinkovitosti novih stavb – kar je tudi v skladu z zakonodajnimi smernicami evropske in slovenske geotermalne politike.

1.1 Primeri dobre prakse

V Sloveniji je uporaba geotermalnih virov v gradbeništvu še vedno razmeroma omejena, zlasti kadar gre za integracijo geotermalnih sistemov neposredno v konstrukcijske elemente, kot so temelji, piloti ali stene. Večina obstoječih rešitev temelji na ločenem pristopu – posebej izvedeni temelji in posebej izvedeni geotermalni sistemi, kar pomeni zamujeno priložnost za učinkovitejšo in stroškovno ugodnejšo zasnovo objekta. V nasprotju s tem v tujini že obstajajo številni inovativni projekti, kjer se geotermalne geokonstrukcije testirajo, razvijajo in uporabljajo v realnem okolju kot del celostnega trajnostnega načrtovanja. V nadaljevanju bomo predstavili nekaj primerov dobre prakse iz tujine, med katerimi izstopa tudi projekt Green Village na Tehniški univerzi Delft, ki služi kot živi laboratorij za preizkušanje tovrstnih naprednih rešitev. Projekt »zelena vas« oziroma Green Village je napredno raziskovalno okolje, namenjeno preizkušanju trajnostnih gradbenih tehnologij v realnih pogojih.

Ključno vlogo pri tem ima uporaba geotermalnih pilotov. V okviru naselja je bilo nameščenih 51 geotermalnih pilotov, dolgih približno 20 metrov, po katerih kroži prenosna tekočina v zaprtem sistemu. Ta sistem omogoča stabilno izmenjavo toplote med zemljo in stavbami preko toplotne črpalke.

Sistem je zasnovan za ogrevanje in hlajenje tipične nizkoenergijske stavbe z ogrevalno močjo približno 8–10 kW in hladilno močjo približno 6–8 kW. Pri tem dosega t. i. grelni števil (angl. COP – Coefficient of Performance) približno 4, kar pomeni, da toplotna črpalka iz 1 enote električne energije pridobi kar 4 enote toplotne energije. Testi so pokazali, da vsak meter geotermalnega pilota prispeva približno 90 W toplotne energije za namene ogrevanja ali hlajenja. Letno to pomeni približno 20–25 MWh koristne toplotne energije za ogrevanje ali hlajenje. Sistem vključuje tudi meritve temperature, pretoka in zmogljivosti, kar omogoča raziskave učinkovitega delovanja ter medsebojnih toplotnih vplivov med piloti.

Projekt služi kot realna raziskovalna platforma za testiranje geotermalnih rešitev v gradbeništvu. Poudarja, kako je mogoče z vključevanjem geotermalnih sistemov neposredno v konstrukcijske elemente zmanjšati potrebo po dodatnih virih ogrevanja, zmanjšati emisije in povečati geotermalno učinkovitost stavb. Green Village tako predstavlja konkreten primer prihodnje trajnostne, geotermalno samoskrbne gradnje [5]. Projekt Green village ni le pomemben za gospodarstvo, ampak tudi za akademsko sfero, saj kot že omenjeno predstavlja živi laboratorij, kjer je vsak dogodek znotraj in v okolišu geotermalnih pilotov izmerjen in zabeležen.

Poznamo pa še dva večja projekta, ki sta mejnika v gradbenem smislu, in sicer prvi primer je Švicarsko letališče Zürich, ki je pionir na področju uporabe geotermalnih pilotov za ogrevanje in hlajenje svojih objektov [6]. Prvi večji projekt so izvedli pri gradnji Terminala E (Dock Midfield) leta 2003, kjer so vgradili 441 pilotov, od tega 306 geotermalno aktiviranih, globokih do 30 m. Sistem s toplotno močjo 0,63 MW je pokrival večino geotermalnih potreb terminala in dokazal visoko učinkovitost (COP > 5).

Uspeh tega sistema je vodil k ambicioznejšemu projektu The Circle, odprtem leta 2020. Gre za večnamenski kompleks ob letališču, pod katerim so vgradili približno 1.150 pilotov, od katerih jih je več kot 1.000 geotermalnih, globokih do 55 m. Sistem dosega skupno 1,8 MW toplotne moči in pokriva večino potreb za ogrevanje in hlajenje brez fosilnih goriv.

Zürich Airport s tem postaja primer dobre prakse trajnostnega razvoja, z dolgoročno strategijo razogljichenja. Dodatno razvijajo tudi pilotni podzemni kanal za sezonsko shranjevanje toplote (globina ~ 300 m), kar kaže na njihovo zavezanost k doseganju geotermalne nevtralnosti do leta 2040 [6].

Drugi veliki inženirski podvig z vidika geotermalne konstrukcije je projekt v avstrijski Tirolski, pod krajem Jenbach, kjer se skriva poseben primer trajnostne rabe infrastrukture – geotermalno ogrevanje iz železniškega predora. V sklopu gradnje nove železniške povezave proti Brennerskemu baznemu predoru je bil v predoru Jenbach izveden pilotni projekt, kjer so 54 metrov predora opremili z inovativnimi betonskimi oblogami – t. i. energijskimi segmenti. Ti segmenti vsebujejo tako imenovan HDPE (High-density polyethylene) cevni sistem, ki prenaša toploto iz okoliške zemljine z najvišjo učinkovitostjo, kar privede do grelnega števila in sezonskega grelnega števila COP/SPF 3.0. Sistem je neposredno povezan z bližnjo upravno stavbo občine Jenbach, kjer toplotno energijo iz predora izkorišča plinska toplotna črpalka. Zasnovan je tako, da pokrije osnovne ogrevalne potrebe objekta in prispeva 40 % vseh potreb po ogrevanju in hlajenju v širšem vidiku kampusa stavb [7].

2 GEOTEHNIČNO DIMENZIONIRANJE GEOTERMALNIH GEOKONSTRUKCIJ

V tem poglavju obravnavamo geotehnično dimenzioniranje geotermalnih geokonstrukcij, s posebnim poudarkom na pilotih kot trenutno najbolj izvedljivih in geotermalno učinkovitih rešitvah za slovenske razmere. Geotermalni piloti omogočajo hkratno nosilno funkcijo temeljenja objekta in rabo geotermalne energije za ogrevanje ali hlajenje, kar pomeni, da se njihov toplotni del vključuje neposredno v obstoječo konstrukcijo brez dodatnih gradbenih posegov.

Dimenzioniranje takih konstrukcij zahteva integracijo klasičnih geotehničnih metod z dodatnimi vidiki toplotno-mehanske interakcije med zemljino in pilotom. Poleg nosilnosti in posedkov moramo upoštevati tudi vplive cikličnega segrevanja in hlajenja in morebitno spremembo lastnosti okoliške zemljine.

V večini evropskih držav še ne obstajajo popolnoma poenoteni tehnični standardi za projektiranje geotermalnih pilotov, vendar so na voljo pomembna nacionalna priporočila v Švici, Franciji, Združenem kraljestvu in Nemčiji.

Ker so piloti najpogosteje uporabljena geotehnična rešitev pri večjih objektih tudi v Sloveniji – zlasti tam, kjer se že sicer zahteva globinsko temeljenje – se poglavje omejuje na podrobnejšo obravnavo dimenzioniranja ravno teh konstrukcij, saj zaradi že vseh prej naštetih lastnosti predstavljajo najprimernejši tip geotermalnih geokonstrukcij za nadaljnji razvoj v slovenskem prostoru.

2.1 Geotermalne geokonstrukcije – pregled literature

Projekti, ki smo jih predstavili v podpoglavju »Primeri dobre prakse«, skupaj z ostalimi projekti, ki se dnevno izvajajo širom sveta, pomembno prispevajo k zmanjšanju emisij ogljikovega dioksida in utrujejo zlasti geotermalne pilote kot najpogostejšo uporabo med geotermalnimi geokonstrukcijami. Osnovno načelo delovanja teh pilotov je izkoriščanje stalne podzemne temperature na globinah med 15 in 50 metri, pri čemer toplota prehaja skozi cevi toplotnega izmenjevalnika, vgrajene v armaturni koš pilota.

Ker služijo hkrati kot nosilni temelji in kot toplotni izmenjevalniki, je treba pri načrtovanju skrbno upoštevati njihovo termomehansko obnašanje. Zato se je razvilo veliko raziskav, ki vključujejo terenske preizkuse, laboratorijske teste in numerične analize. Pretekle pregledne študije so se osredotočale na toplotno učinkovitost in termomehansko odzivanje, a celostni optimizacijski pristopi, ki bi hkrati obravnavali toplotne in nosilne vidike, so še vedno redki [8], [9], [10], [11], [12].

Termomehanska analiza geotermalnih pilotov predstavlja enega najzahtevnejših vidikov njihovega načrtovanja, saj se učinki temperaturno povzročene razteznosti in krčenja prepletajo s klasičnimi geotehničnimi obtežbami. Eksperimentalne raziskave so pokazale, da se v geotermalnih pilotih ob toplotni obremenitvi pojavijo pomembne spremembe napetosti: pri segrevanju se tlačne napetosti praviloma povečajo za približno 40–360 kPa/°C, medtem ko se pri ohlajanju zmanjšajo za približno 15–180 kPa/°C [13]. Velikost teh osnih napetosti je močno odvisna od vrste pilota (trenjski, stoječi) ter velikosti temperaturnih sprememb. Hkrati pa toplotna obremenitev mobilizira strižno odpornost vzdolž zgornjih in spodnjih delov pilota ter povzroča pomembne spremembe nosilnosti – ta se pri segrevanju lahko poveča za približno 8,4–12,3 %, medtem ko se pri ohlajanju lahko zmanjša za okoli 8,7 % [14], [15], [16].

Je pa treba poudariti, da deformacije zaradi vpliva spremembe temperature predstavljajo pretežno reverzibilne deformacije, pri čemer se največji del pojavi že med začetnimi toplotnimi cikli, nato pa se postopoma kopičijo skozi daljša časovna obdobja [17], [18]. Odnos med pomikom in temperaturo prav tako kaže reverzibilno obnašanje, podobno krivuljam toplotnega raztezanja pri zmernih mehanskih obremenitvah. Vendar pa ob mehanskih obremenitvah, ki presegajo 40 % končne nosilnosti pilota, pride do ireverzibilnih posedkov na konicah pilotov v peščenih tleh, kar predstavlja ključno mejo, ki jo je treba upoštevati pri načrtovanju [19].

Za izračun teh dodatnih posedkov in napetosti se največkrat uporabljata dva metodološka pristopa, in sicer, analitične metode prenosa obremenitve (delovni diagrami) in numerični pristopi. Metode prenosa obremenitve (t-z metode) razdelijo pilote na toge elemente, povezane z vzmetmi, kar omogoča preprostejše računske pristope za praktično načrtovanje [20], [21]. Kljub temu imajo te metode pomembne omejitve – predpostavljajo konstantno spremembo temperature v tleh in ne upoštevajo vpliva temperature na mehanske lastnosti tal, kar je lahko še posebej problematično v glinenih tleh, kjer so termične obremenitve tudi merodajne in kjer lahko prihaja do toplotne konsolidacije [22]. Modeli prav tako predpostavljajo, da je temperaturna sprememba za ekstrakcijo plitve geotermalne energije običajno omejena na območje 0–40 °C [23], [24]. V računskih modelih se za namene pridobitve zasnov uporabljajo bolj konservativne vrednosti temperaturnih razlik, in sicer do $\Delta T = 20^\circ$ [25], kar pa iz geotehničnega vidika že predstavlja precejšnjo spremembo temperature in posledično vpliv na konstrukcijo.

Primerjalne analize različnih t-z pristopov so pokazale tudi precejšnje razlike v rezultatih, čeprav so bile uporabljene enake lastnosti tal in pilotov. Zaradi tega je potrebno izbrati ustrezne in preverjene t-z funkcije, saj lahko drugače podcenimo posedke zaradi vpliva spremembe temperature [26].

Celovito numerično modeliranje, ki zajema termo-hidro-mehanske procese, omogoča podrobnejše analize, saj vključuje mehanske robne pogoje, specifično vedenje posameznih elementov, vpliv podtalnice in spremembo materialnih lastnosti. Ti pristopi pa terjajo več časa za pridobitev rezultatov in so tudi zelo odvisni od pravilnosti vhodnih podatkov, saj je vpliv podtalnice in spremembe materialnih lastnosti treba modelirati z obzirom, saj napačne predpostavke lahko privedejo do skrajno konservativnih zasnov. S tem pa seveda do ekonomske neupravičenosti konstrukcije [27], [28].

Zaradi ekonomske upravičenosti so bile že izvedene optimizacijske študije geotermalnih pilotov, ki so ob svojih predpostavkah (homogena tla, konstantna sprememba temperature) pokazale, da so kritične zemljine tiste, ki imajo modul elastičnosti manjši kot $E_{soil} < 15$ MPa in kjer je prisotna vertikalna mehanska obremenitev vsaj 1 MN, kar je razvidno iz večjih posedkov geotermalnih pilotov v primerjavi z geotermalnimi piloti v zemljinah z večjim modulom elastičnosti. Kadar imamo horizontalno obremenjene pilote dimenzionirane v skladu z Evrokod 7, dobimo bolj robustne zasnove, saj morajo prenesti obremenitev z momentom. Na tako zasnovi potem sprememba temperature nima bistvenega vpliva. Ugotovitve teh optimizacijskih študij sovpadajo z ugotovitvami parametričnih analiz, ki kažejo na to, da je sprememba temperature kritična pri optimalni zasnovi, kadar imamo tla z nizkim modulom elastičnosti [29], [30], [31].

Tudi ekonomsko upravičenost so raziskovalci analizirali pri že obstoječih objektih. Obsežna analiza v Avstraliji, ki je zajemala več različnih ogrevalnih sistemov (plin, olje, biomasa, geotermalna energija) in katerih cene se po podrobni analizi ujemajo ali pa zelo približajo slovenskim, je pokazala, da kljub večji naložbi pri gradnji geotermalnih geokonstrukcij v delovanju stavbe prihranimo od 15–26 % ogrevalnih stroškov, ter naložbo pokrijemo v 5–7 letih [32]. Do podobnih zaključkov so prišli tudi pri rekonstrukciji letališča v Zürichu, Švici. Čeprav je bila gradnja geotermalnih pilotov 10x dražja, kot gradnja konvencionalnih se je naložba povrnila v 8 letih, saj so prihranki pri ogrevanju in hlajenju prostorov odtehtali začetno naložbo [6].

Čeprav obstaja že več izpeljanih projektov, ki kažejo ekonomsko upravičenost geotermalnih geokonstrukcij, je še vedno pomembno dodatno raziskovanje optimalnih zasnov in razvoj modelov, ki bi z upoštevanjem vseh vplivov privedli do optimalnih zasnov, saj bi lahko s tem še dodatno izboljšali povratno dobo naložbe in izvedbo tovrstnih konstrukcij približali širši javnosti in gradbeni stroki.

2.2 Mehanske lastnosti geotermalnih pilotov

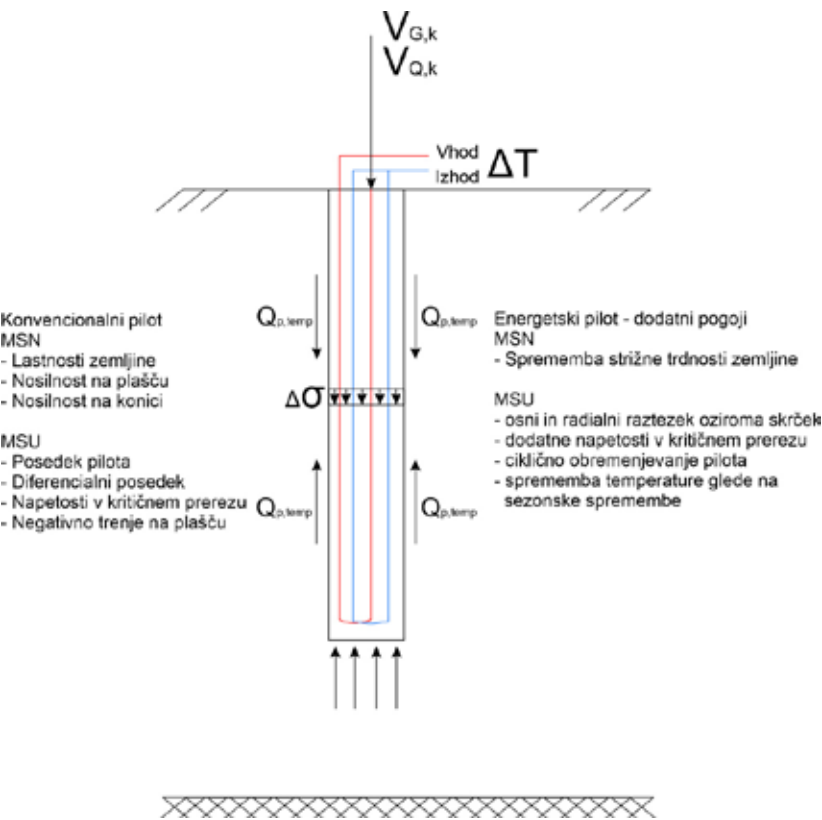
Združeno kraljestvo razpolaga s priporočili, izdanimi s strani GSHP (Ground Source Heat Pump Association) [33], ki so za dimenzioniranje geotermalnih pilotov pripravljena s pomočjo in-situ raziskav. Avtorji poudarjajo, da je treba upoštevati učinek vpliva temperature tako pri MSN kot pri MSU. Čeprav se vpliv mehanske obtežbe in vpliv spremembe temperature obravnavata ločeno, je potrebno končne napetosti in vertikalne pomike upoštevati kot celoto in ti ne smejo presegati dovoljenih zahtev. GSHP priporoča, da se ob že predpisanih pogojih klasičnega projektiranja pilotov preverijo še slednji pogoji:

Za mejno stanje nosilnosti (MSN):

- Morebitne spremembe strižne trdnosti zemljine zaradi učinka vpliva temperature (ogrevanje/hlajenje).

Za mejno stanje uporabnosti (MSU):

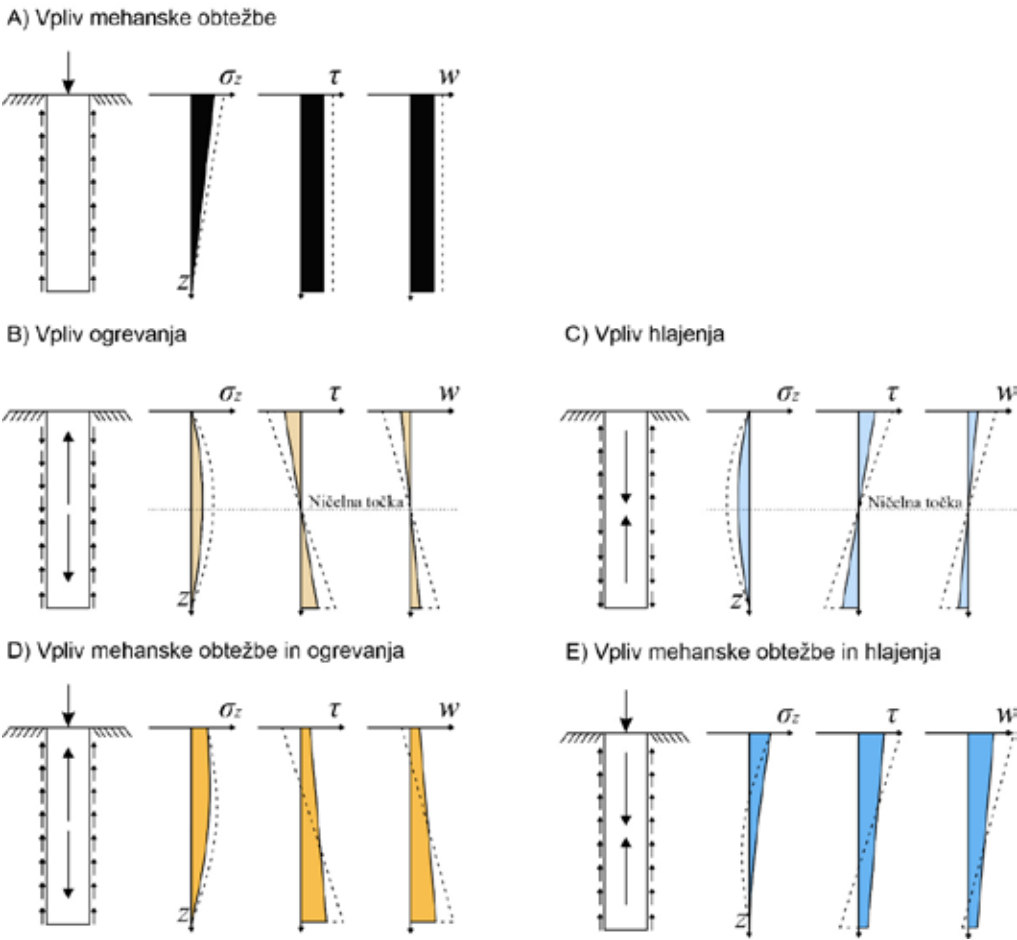
- osni in radialni raztezek ali skrčak,
- dodatne napetosti v armiranobetonskem prerezu zaradi vpliva spremembe temperature,
- upoštevanje cikličnega obremenjevanja in
- spremembo temperature na stiku zemljine in geotermalnega pilota glede na sezonska nihanja.



Slika 2: Dodatna priporočila, ki jih je treba upoštevati pri zasnovi geotermalnih pilotih.

Projektiranje geotermalnih konstrukcij tako zahteva razumevanje vpliva spremembe temperature na geotermalno konstrukcijo, v našem primeru pilota. Za lažjo oceno vpliva se lahko uporabijo sheme, ki prikazujejo vpliv spremembe temperature na geotermalni pilot. Odziv pilota ob ogrevanju, hlajenju in vplivu mehanske obremenitve so prikazane na spodnjih shemah. Sheme, ki so bile oblikovane na podlagi teorije elastičnosti, prikazujejo vpliv teh obremenitev na napetosti in vertikalne pomike geotermalnega pilota za različne robne pogoje vpetja [34]. Trenjski geotermalni pilot (konica in glava pilota nista toga vpeti) je prikazan na spodnji sliki 3. Izraz tega vpetja se v teh primerih razume kot osna togost.

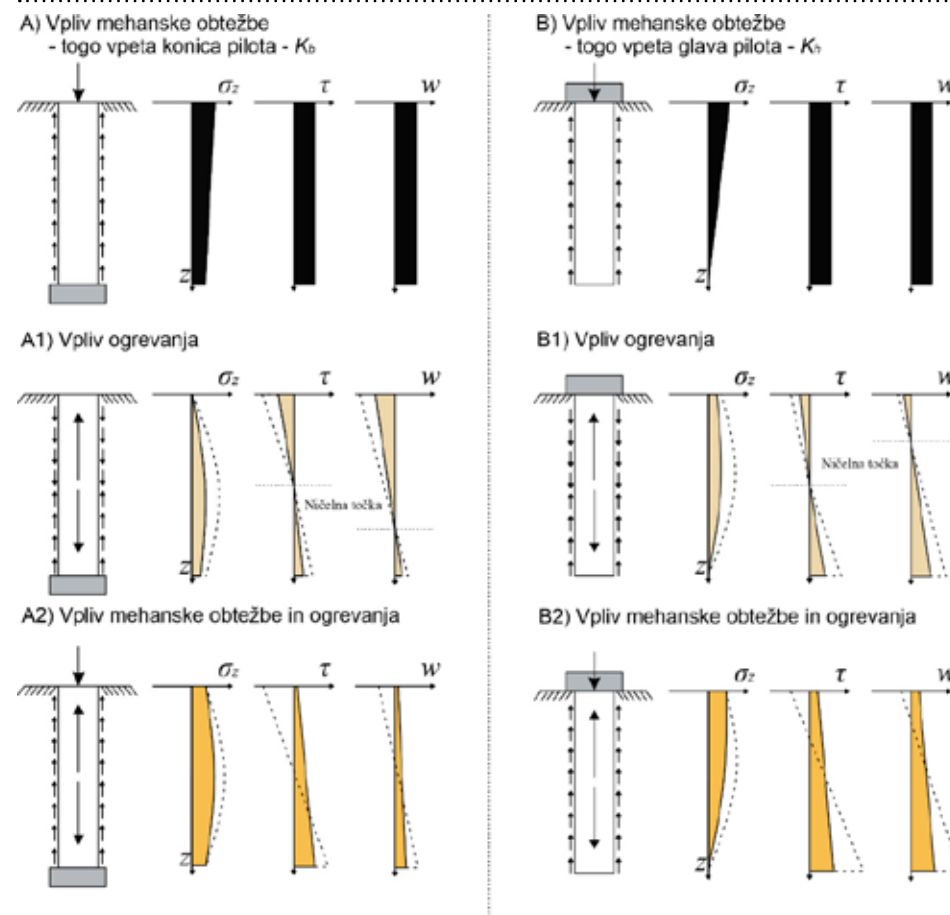
Termo-mehanične sheme za plavajoči pilot



Slika 3: Shematski prikaz vpliva temperaturne spremembe in mehanske obremenitve na potek napetosti in vertikalnih pomikov pri trenjskem geotermalnem pilotu [34].

Iz sheme je razvidno, da učinek vpliva mehanske obtežbe povzroči padajočo linearno porazdelitev vertikalnih napetosti vzdolž geotermalnega pilota ter približno enakomerno porazdelitev strižnih napetosti vzdolž pilota, ki delujejo v nasprotni smeri kot tlačna vertikalna napetost, saj delujejo kot odpor na plašču. Vertikalni pomik je konstanten vzdolž plašča. Bolj kot je geotermalni pilot tog, bolj so te veličine enakomerno in linearno porazdeljene, seveda pa pride do manj enakomernih porazdelitev v primeru različne slojevitosti tal. V primeru učinka vpliva spremembe temperature pa pride pri ogrevanju ali hlajenju do nelinearne porazdelitve osne napetosti, katere največja vrednost je na sredini dolžine trenjskega pilota. Na tej točki pa je prav tako ničelna točka strižnih napetosti in vertikalnih pomikov.

Ničelna točka se pojavi, ker se v primeru ogrevanja pilota ($\Delta T > 0^\circ\text{C}$) v zgornji polovici geotermalnega pilota mobilizirajo strižne napetosti, ki delujejo v nasprotni smeri kot osne napetosti in povzročijo dvig geotermalnega pilota, medtem ko se v spodnji polovici geotermalnega pilota mobilizirajo strižne napetosti, ki delujejo v enaki smeri kot osne napetosti in povzročijo vertikalne posedke. V primeru hlajenja pilota ($\Delta T < 0^\circ\text{C}$) pa je učinek ravno nasproten, kot je razvidno iz slike 3. Na sliki 4 so prikazani geotermalni piloti z vpeto glavo ali konico.



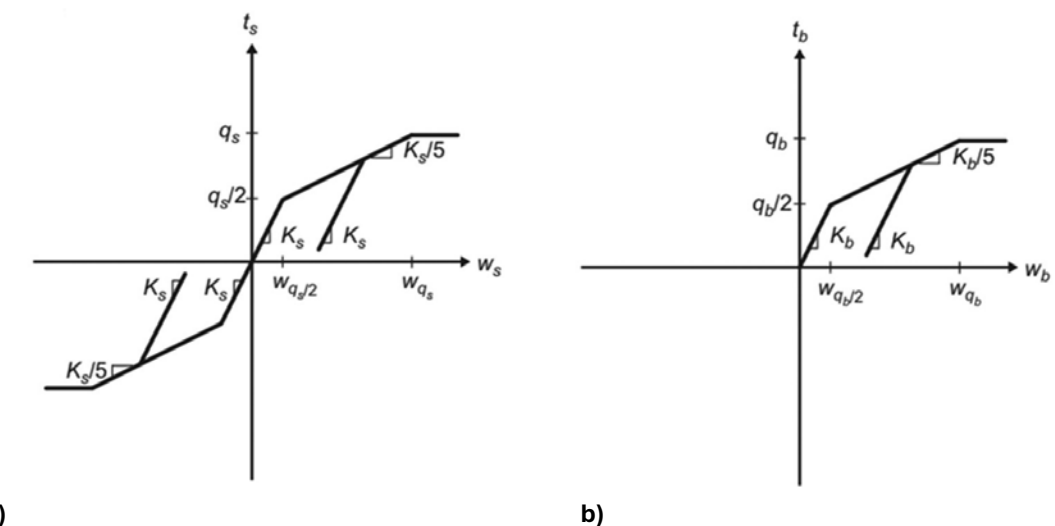
Slika 4: Shematski prikaz vpliva temperature spremembe in mehanske obremenitve na potek napetosti in vertikalnih pomikov pri delno vpetem geotermalnem pilotu [35].

Iz shematskega prikaza (Slika 4) je razvidno, da se v primeru robnega pogoja togo vpete glave pilota vzdolž geotermalnega pilota pojavijo manjše vertikalne napetosti, manjše strižne napetosti in manjši pomiki, saj na primer betonska plošča nad pilotom prevzame del vpliva mehanske obtežbe in prenos sil ni neposreden. Kadar imamo robni pogoj vpetja, se pri vplivu spremembe temperature pojavijo večje napetosti na tistem koncu, kjer je pilot vpet. Torej, če imamo vpeto konico, so večje napetosti pri konici in obratno. Napetosti, ki se pojavijo v stoječem pilotu, so tako večje kot pri primeru, kadar imamo trenjski geotermalni pilot. Tudi ničelna točka zdaj ni več v središču dolžine pilota, ampak je pomaknjena v smeri vpetja, torej če je vpeti glava, je ničelna točka bližje glavi pilota. Posledično se pojavijo večje strižne napetosti in večji pomiki tam, kjer ni vpetja geotermalnega pilota, in manjše na drugi strani geotermalnega pilota. V primeru hlajenja pa so napetosti in pomiki simetrični ogrevanju. Poudariti je treba, da kadar projektiramo geotermalni pilot z vpeto konico in hlajenjem, lahko zaradi zmanjšanja tlačnih napetosti na konici pilota in nezmožnosti zemljine prevzema nateznih napetosti prevzame le vsoto aplicirane obtežbe in lastne teže.

Da bi lahko inženirji pridobili bolj konkretne vrednosti posedkov geotermalnih pilotov zaradi vpliva spremembe temperature, so bili z uporabo metode končnih elementov in integralnih enačb razviti diagrami pomikov, ki inženirju omogočajo oceno pomikov, do katerih pride zaradi vpliva spremembe temperature. Z in-situ preiskavami in mehanskimi shemami so raziskovalci ugotovili, da so za vertikalni posedek geotermalnega pilota ključni štirje dejavniki [33], in sicer:

- modul stisljivosti zemljine in togost geotermalnega pilota,
- geometrijska zasnova geotermalnega pilota,
- robni pogoji vpetosti geotermalnega pilota in
- linearni toplotni raztezek geotermalnega pilota.

Za modeliranje pomikov, ki so lahko kritični pri geotermalnih pilotih, se uporabljajo funkcije prenosa obtežbe. Obstaja več vrst funkcij prenosa obtežbe, kot so hiperbolične, eksponentne, kubične ipd. [35]. Osnovni delovni diagrami, ki predpostavljajo linearne prenose obtežbe po avtorjih Frank in Zao [36] so najbolj razširjene funkcije prenosov, predvsem zaradi lažje implementacije v razne računske in optimizacijske modele.



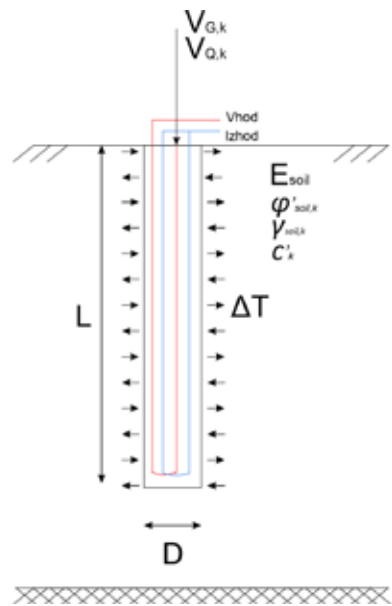
a)

b)

Slika 5: Delovni diagram, ki prikazuje korelacijo med obremenitvijo in pomikom za pilot. a) plašč in b) konica pilota [36].

3 OPTIMIZACIJSKI MODEL GEOTERMALNEGA PILOTA

Geometrijsko zasnovo trenjskega in stoječega geotermalnega pilota prikazuje slika 6, kjer so tudi prikazani delni vhodni podatki optimizacijskega modela, ki je podrobneje opisan v nadaljevanju poglavja.



Slika 6: Geometrijska zasnova geotermalnega trenjskega in stoječega pilota.

Stroškovna funkcija vsebuje stroške materialov, izdelave geotermalnih pilotov in zemeljskih del. Zapis stroškovne funkcije je naslednji:

$$\min: COST = C_{drill} + C_{conc} + C_{rein,main} + C_{rein,shear} + C_{thermal} = L \cdot \left(c_{1,drill} + c_{2,drill} \cdot (D - D_{min}) \right) + L \cdot \left(\frac{\pi \cdot D^2}{4} \right) \cdot c_{conc} + L \cdot \left(\frac{\pi \cdot \phi_{main}^2}{4} \right) \cdot n_{main} \cdot \rho_s \cdot c_{rein} + \left(\frac{\pi \cdot \phi_{shear}^2}{4} \right) \cdot (\pi \cdot D) \cdot \left(\frac{L}{s_{shear}} \right) \cdot \rho_s \cdot c_{rein} + L \cdot c_{thermal} \quad (3.0)$$

kjer $COST$ predstavlja vse stroške gradnje geotermalnega pilota v EUR. Optimalna rešitev so tako minimalni stroški geotermalnega pilota ob pogoju, da geotermalni pilot prenese dane vplive. Namen-ska funkcija je tako podrobneje sestavljena iz naslednjih ekonomskih podatkov: strošek odstranjeva-nja zemljine, ki zajema izkop in deponiranje, s pomočjo vrtanja in dodatni strošek za večje pre-mere pilotov C_{drill} [EUR/m], ki vsebuje $c_{1,drill}$ [EUR/m] in predstavlja strošek za odstranitev zemljine in $c_{2,drill}$ [EUR/m/mm], ki predstavlja dodatni strošek glede na večji premer pilota, ter premer pilota D [mm] in minimalni premer pilota D_{min} [mm], strošek vgradnje betona C_{conc} [EUR/m³], ki vsebuje strošek na enoto c_{conc} [EUR/m³] in dolžino pilota L [m], armaturo $C_{rein,main}$ in strižno armaturo $C_{rein,shear}$, ki vsebuje strošek na enoto za jekleno armaturo c_{rein} [EUR/kg], premer armaturnih palic Φ_{main} [mm], število palic n_{main} , gostota jekla ρ_s [kg/m³] in za strošek strižne armature, premer stremen Φ_{shear} [mm] in razmik le-teh s_{shear} [mm] in strošek vgradnje sistema (cevi in zanke) za ogrevanje oziroma hlajenje z geotermalnim pilotom $C_{thermal}$ [EUR/m], ki je nadalje sestavljena iz stroška na enoto vgradnje sistema $c_{thermal}$ in dolžino pilota L [m]. Glavne spremenljivke optimizacijskega modela so tako dolžina geoter-malnega pilota L , premer geotermalnega pilota D , premer armaturnih palic Φ_{main} , število palic n_{main} , premer stremen Φ_{shear} in razmik stremen s_{shear} .

Temelj za razvoj optimizacijskega modela predstavljata geotehnična in konstrukcijska analiza. Ključni so mejni pogoji skladno z zahtevami Evrokodov [37], [38] saj ti zagotavljajo varnost in dolgoročno stabilnost konstrukcije. Zaradi dodatnega vpliva temperaturnih sprememb smo upoštevali še pripo-ročila Ground Source Heat Pump Association [33] ter usmeritve iz literature avtorjev Laloui in Rotta Loria [34]. Da bi zajeli vse pomembne vidike, smo oblikovali sedem pogojev, ki jih mora zasnova izpolnjevati:

- Prvi pogoj: nosilnost geotermalnega pilota na konici in plašču (MSN).
- Drugi pogoj: posedek geotermalnega pilota zaradi vpliva mehanske obtežbe in vpliva spremembe temperature.
- Tretji pogoj: pojav osne napetosti na kritičnem mestu (odvisno od robnih pogojev) geotermal-nega pilota zaradi vpliva mehanske obtežbe in vpliva spremembe temperature.

V skladu z Evrokod 2 [39] je treba zagotoviti tudi minimalno armaturo zaradi razpok. Potrebna arma-tura je definirana z naslednjimi štirimi pogoji:

- Četrty pogoj: minimalna potrebna armatura mora biti zagotovljena.
- Peti pogoj: razmik med strižno armaturo mora biti v predpisanih mejah.
- Šesti pogoj: zagotovitev minimalnega premera strižne armature.
- Sedmi pogoj: zadostno število armaturnih palic (zgornja in spodnja meja).

Za izvedbo optimizacije je bil uporabljen genetski algoritem z realno kodiranimi števili RCGA [40]. Za uporabo genetskega algoritma namesto linearnih metod smo se odločili zaradi dela z nezveznimi funkcijami, kjer lahko manj robustni algoritmi hitro obtičijo v lokalnih skrajnostih. Genetski algo-ritmi, ki temeljijo na principih naravne selekcije, križanja in mutacije, so zaradi svoje stohastične narave posebej primerni za iskanje globalno optimalnih rešitev v kompleksnih in diskontinuiranih prostorih, saj omogočajo učinkovito preiskovanje širšega nabora možnih rešitev.

3.1 Nosilnost geotermalnega pilota na konici in plašču (MSN)

Za oceno nosilnosti geotermalnega pilota je bila uporabljena metoda, povzeta po Fernandesu [41], ki temelji na klasičnih izhodiščih avtorja Vesića [42]. Ključna prednost te metode je, da pri izračunu odpora pilota upošteva togost zemljine, kar je še posebej pomembno v primeru geotermalnih pilo-tov, kjer poleg mehanskih delujejo tudi temperaturni vplivi. S tem je zagotovljena skladnost z meto-dami za določanje posedkov zaradi temperaturnih obremenitev, ki prav tako vključujejo vpliv togosti zemljine.

Enačbe metode so zbrane v spodnji tabeli (Tabela 1). Dana metoda upošteva vertikalno obremeni-tev pilota, ki je omejena z rezultanto odpora zemljine (enačba 3.1). Vse obremenitve, ki delujejo na pilot, so zajete v skupni enačbi (3.2), medtem ko je skupna nosilnost pilota $R_{c,d}$ določena po enačbi (3.3). Skupni odpor je sestavljen iz odpora na plašču (enačba 3.4) in odpora na konici pilota (enačba 3.5). Pri tem se uporabljajo standardni parametri zemljine, kot so projektne vrednosti kohezije in trenja (c'_d , ϕ'_d), projektni kot trenja med zemljino in pilotom (δ'_d), koeficient zemeljskega pritiska ter vertikalne efektivne napetosti, vse skladno s splošno znanimi enačbami, ki jih v prispevku nismo podrobneje predstavili.

Za razliko od ostalih metod izračuna nosilnosti pilota, kot so Berezantzev, Meyerhof ipd., ta metoda zraven korekcijskih faktorjev za obliko in globino konice vključuje tudi korekcijski faktor za togost zemljine, ki je odvisen od indeksov togosti zemljine (I_R , I_{RC} , I_{RR}), ki ločujejo med različnimi porušitve-nimi mehanizmi (enačbe 3.6 – 3.17).

Izbrana metoda torej nudi celostno in mehansko skladno obravnavo nosilnosti geotermalnega pilota, ki ne upošteva le geotehničnih parametrov zemljine, temveč tudi vpliv njene togosti, kar je ključno za pravilno dimenzioniranje geotermalnih sistemov, pri katerih prihaja do sprememb, ki so odvisne od togosti zemljine.

Tabela 1: Enačbe za izračun nosilnosti geotermalnega pilota

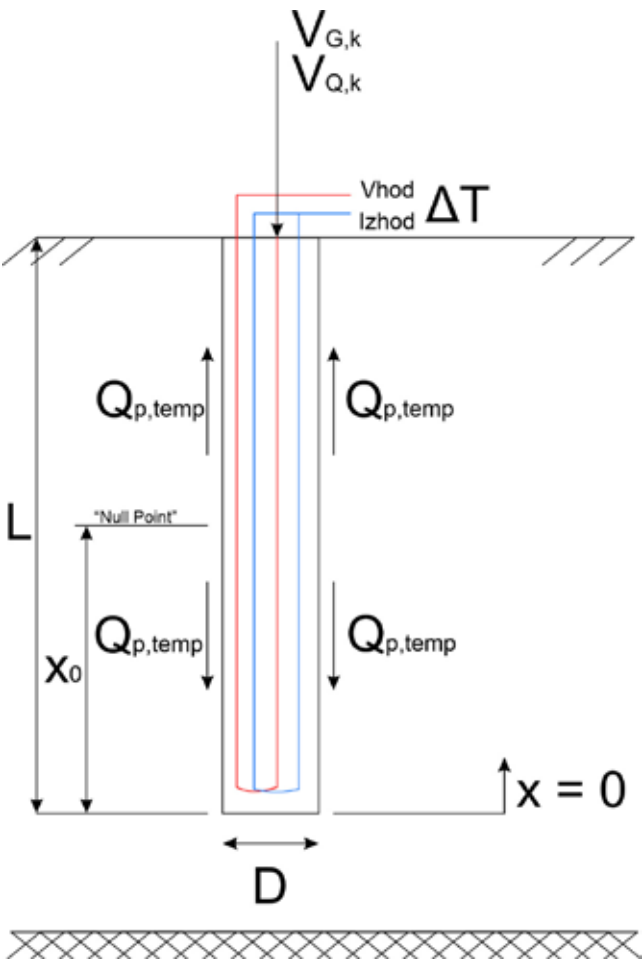
Pogoj 1		
$V_{Ed} \leq R_{c,d}$	(3.1)	$\check{C}e \ I_{RC} \geq I_{RR}$ (3.10)
$V_{Ed} = SF_G \cdot (V_{G,k} + \pi \cdot D^2 / 4 \cdot L \cdot \gamma_{wc}) + SF_Q \cdot V_{Q,k}$	(3.2)	$\xi_{qr} = e^{\left(-3,8 \cdot \tan(\varphi'_d) + \frac{(3,07 \cdot \sin \varphi'_d) \cdot (\log(2 \cdot I_{RR}))}{1 + \sin \varphi'_d} \right)}$ (3.11)
$R_{c,d} = \frac{R_{s,k}}{SF_s \cdot SF_m} + \frac{R_{b,k}}{SF_b \cdot SF_m}$	(3.3)	Sicer (3.12)
$R_{s,k} = q_s \cdot A_{shaft}$	(3.4)	$\xi_{qr} = 1.0$ (3.13)
$R_{b,k} = q_b \cdot A_b$	(3.5)	$I_{RC} = 0,5 \cdot e^{\left(2,85 \cdot \cot \left(45^\circ - \frac{\varphi'_d}{2} \right) \right)}$ (3.14)
$N'_q = N_q \cdot \xi_{qr} \cdot s_q \cdot d_q$	(3.6)	$I_{RR} = \frac{I_R}{1 + I_R \cdot \varepsilon_v}$ (3.15)
$N_q = e^{(\pi \cdot \tan(\varphi'_d))} \tan^2 \left(45 + \frac{\varphi'_d}{2} \right)$	(3.7)	$\varepsilon_v = 0 \rightarrow I_{RR} = I_R$ (3.16)
$s_q = 1 + \tan(\varphi'_d)$	(3.8)	$I_R = \frac{E_{soil}}{2 \cdot (1 + \nu) \cdot (c'_d + \sigma'_{v,base} \cdot \tan \varphi'_d)}$ (3.17)
$d_q = 1 + 2 \cdot \tan(\varphi'_d) \cdot (1 - \sin(\varphi'_d))^2 \cdot \tan^{-1} \left(\frac{L}{D} \right)$	(3.9)	

3.2 Posedek geotermalnega pilota zaradi vpliva mehanske obtežbe in vpliva spremembe temperature

Za oceno posedka zaradi vpliva spremembe temperature in mehanske obtežbe so bile uporabljene analitične enačbe, ki so jih razvili avtorji Perić et. al. [43]. Analitične enačbe temeljijo na predhodnih raziskavah in funkcijah prenosa obtežbe več različnih avtorjev, ki so prišli do podobnih zaključkov [44], [45], [46]. Predvideno obnašanje geotermalnega pilota, ki je povzeto po prej omenjenih avtorjih, smo že prikazali v prejšnjem poglavju (poglavje 3.2). Ocene posedkov z uporabo enačb razvitih s strani avtorjev Perić et. al. tako sovpadajo s predvidenim obnašanjem geotermalnega pilota in so bile tudi večkrat potrjene s primerjavo numeričnih metod in »in situ« testov. Zaradi kompleksnosti vplivov spremembe temperature v večslojnih tleh imajo izrazi za analizo določene omejitve. Izrazi predpostavljajo:

- Eno dimenzionalni vpliv obremenitve (raztezek/skrček v vertikalni smeri),
- predpostavljena konstantna sprememba temperature vzdolž pilota,
- geotermalni pilot se obnaša linearno elastično in
- predpostavljen je homogen sloj zemljine.

Diagram prostega telesa, za lažje razumevanje enačb, ki sledijo, je naslednji:



Slika 7: Diagram prostega telesa pri geotermalnem pilotu.

Z uporabo kinematičnih zvez so bile izpeljane enačbe za izračun posedkov in napetosti zaradi vpliva spremembe temperature in mehanske obremenitve. Enačbe so zbrane v spodnji tabeli (Tabela 2). Pogojna enačba za posedek pilota je enačba (3.18). Ta pogoj omejuje posedek pilota – μ_{total} , ki predstavlja seštevek posedka zaradi mehanske obremenitve (enačba (3.19)) in zaradi vpliva spremembe temperature (enačba (3.20)). Posedek je omejen s spreminjajočo se omejitvijo μ_{lim} (enačba 3.21), ki predstavlja procentualne vrednosti premera (0,1D; 0,05D; 0,01D). Parameter x_0 predstavlja točko, kjer ni vertikalnega posedka in je tako imenovan »null point« in je definiran z enačbo (3.22). Ker nas zanimajo maksimalni pomiki in maksimalne napetosti, ki se pojavijo v prečnem prerezu geotermalnega pilota zaradi vpliva temperature, je parameter x definiran v skladu s shematskimi prikazi. Brezdimenzijski količnik ψ , ki zajema geometrijske lastnosti geotermalnega pilota in razmerje med togostjo pilota in zemljine, je določen z enačbo (3.23). Količnik K_s opisuje nagib elastične veje obremenitve in razbremenitve plašča geotermalnega pilota in je definiran z enačbo (3.24), kjer je količnik n odvisen od vrste zemljine (grobo in fino zrnata zemljina). Ta količnik je določen po priporočilih avtorja Frank in Zhao [36], funkcije prenosa obtežbe so prav tako prikazane na sliki 5. Menardov presiometrični modul stisljivosti E_m je določen z enačbo (3.25), kjer koeficient α_r predstavlja reološke lastnosti zemljine. Edometrski modul se določi z uporabo enačbe (3.26). Površina konice pilota predstavlja enačba (3.27). Obseg pilota je definiran, kot diktira enačba (3.28).

Tabela 2: Enačbe za izračun posedka geotermalnega pilota.

<i>Pogoj 2</i>			
$u_{total} = u_{mechanical} + u_{thermal} \leq u_{lim}$	(3.18)	$K_s = \left(\frac{n \cdot E_m}{D}\right)$	(3.24)
$u_{mechanical} = \frac{V_{SLS} \cdot E_p \cdot \psi \cdot \cosh(\psi \cdot x_{max,\mu}) + K_b \cdot \sinh(\psi \cdot x_{max,\mu})}{A_b \cdot E_p \cdot \psi \cdot \sinh(\psi \cdot L) + K_b \cdot \cosh(\psi \cdot L)}$	(3.19)	$E_m = E_{oed} \cdot \alpha_r$	(3.25)
$u_{thermal} = \frac{\alpha_p \cdot \Delta T \cdot \sinh(\psi \cdot (x_{max,\mu} - x_0))}{\psi \cdot \cosh(\psi \cdot (L - x_0)) + \frac{K_b}{E_p} \cdot \sinh(\psi \cdot (L - x_0))}$	(3.20)	$E_{oed} = \frac{E_{soil} \cdot (1 - \nu)}{(1 + \nu) \cdot (1 - 2 \cdot \nu)}$	(3.26)
$u_{lim} = 0,1 \cdot D; 0,05 \cdot D; 0,01 \cdot D$	(3.21)	$A_b = \frac{\pi \cdot D^2}{4}$	(3.27)
$x_0 = \frac{1}{\psi} \cdot \tanh^{-1}\left(\frac{\psi \cdot \cosh(\psi \cdot L) - \psi + \frac{K_b}{E_p} \cdot \sinh(\psi \cdot L)}{\psi \cdot \sinh(\psi \cdot L) + \frac{K_b}{E_p} \cdot \cosh(\psi \cdot L) + \frac{K_b}{E_p}}\right)$	(3.22)	$p = \pi \cdot D$	(3.28)
$\psi = \sqrt{\frac{p}{A_b} \cdot \frac{K_s}{E_p}}$	(3.23)		

3.3 Pojav osne napetosti zaradi vpliva mehanske obtežbe in zaradi vpliva spremembe temperature

Tretji pogoj (Tabela 3) v optimizacijskem modelu omejuje dovoljene napetosti v kritičnem prerezu geotermalnega pilota (enačba (3.29)) in določa, da mora biti vsota napetosti σ_{total} , ki nastanejo zaradi mehanske obtežbe $\sigma_{mechanical}$ (enačba (3.31)) ter napetosti zaradi temperaturnih sprememb $\sigma_{thermal}$ (enačba (3.32)), manjša od produkta karakteristične tlačne trdnosti betona f_{ck} in faktorja, ki upošteva dolgoročne učinke na tlačno trdnost α_{cc} .

Pri geotermalnih pilotih, kjer skozi armiranobetonski prerez potekajo temperaturne spremembe zaradi izmenjave toplote s tlemi (npr. pri sistemih za ogrevanje in hlajenje), prihaja do razvoja dodatnih notranjih napetosti. Študije znanih avtorjev na tem področju navajajo, da so te napetosti posledica omejenih toplotnih deformacij [47], [48], [49]. Beton se zaradi segrevanja razteza oziroma zaradi ohlajanja krči, vendar to gibanje omejuje trenje in odpornost okoliške zemljine, kar povzroča nastanek dodatnih osnih napetosti v pilotu. Če te napetosti niso ustrezno upoštewane, lahko vodijo do razpok v betonu ali do dolgoročnih poškodb konstrukcije, zlasti pri ponavljajočih se temperaturnih ciklih. Zato je vključitev napetosti zaradi vpliva spremembe temperature v projektiranje ključnega pomena za zagotavljanje varnosti in trajnosti konstrukcije.

Tabela 3: Enačbe za izračun napetosti v armiranobetonskem prerezu geotermalnega pilota.

<i>Pogoj 3</i>			
$\sigma_{total} = \sigma_{mechanical} + \sigma_{thermal} \leq \sigma_{lim}$	(3.29)	$\sigma_{mechanical} = \frac{V_{SLS} \cdot E_p \cdot \psi \cdot \sinh(\psi \cdot x_{max,\sigma}) + K_b \cdot \cosh(\psi \cdot x_{max,\sigma})}{A_b \cdot E_p \cdot \psi \cdot \sinh(\psi \cdot L) + K_b \cdot \cosh(\psi \cdot L)}$	(3.31)
$\sigma_{lim} = \alpha_{cc} \cdot f_{ck}$	(3.30)	$\sigma_{thermal} = E_p \cdot \alpha_p \cdot \Delta T \left(\frac{\psi \cdot \cosh(\psi \cdot (x_{max,\sigma} - x_0))}{\psi \cdot \cosh(\psi \cdot (L - x_0)) + \frac{K_b}{E_p} \cdot \sinh(\psi \cdot (L - x_0))} - 1 \right)$	(3.32)

3.4 Minimalna zahtevana armatura v skladu z Evrokodom 2

Za namene optimizacije je prav tako bila upoštevana minimalna potrebna armatura v skladu z Evrokodom 2. Z neenačbo (3.33) je omejena minimalna potrebna površina armature v prečnem prerezu geotermalnega pilota $A_{main,min}$, ki je izračunana z enačbo (3.34). Zagotovljena minimalna površina armature $A_{main,prov}$ pa je določena z enačbo (3.35), kjer Φ_{main} predstavlja nominalni premer armature in n_{main} število armaturnih palic. Oba parametra sta diskretni alternativni v optimizacijskem modelu, katerih vrednosti so definirane v tabeli 4. Omejitve, ki pa diktirajo optimalne vrednosti, so zapisane v pogoju 6 in pogoju 7, ter omejene z enačbami (3.36–3.46).

Tabela 4: Konstrukcijski pogoji geotermalnega pilota, obremenjenega z vertikalno silo.

<i>Pogoj 4</i>			
$A_{main,prov} \geq A_{main,min}$			(3.33)
kjer:			
$A_{main,min} = \frac{0,004 \cdot \pi \cdot D^2}{4}$	(3.34)	$A_{main,prov} = \frac{\pi \cdot \Phi_{main}^2}{4} \cdot n_{main}$	(3.35)
<i>Pogoj 5</i>			
$s_{shear} \leq s_{shear,max}$			(3.36)
kjer:			
$s_{shear,max} = 12 \cdot \Phi_{main}$			(3.37)
<i>Pogoj 6</i>			
$\Phi_{shear}^{LO} \leq \Phi_{shear} \leq \Phi_{shear}^{UP}$			(3.38)
kjer:			
$\Phi_{shear}^{LO} \geq \text{Max}(0,25 \cdot \Phi_{main}; 6 \text{ mm})$			(3.39)
<i>Pogoj 7</i>			
$n_{main}^{LO} \leq n_{main} \leq n_{main}^{UP}$			(3.40)
kjer:			
$n_{main}^{UP} > \frac{\pi \cdot (D - 2 \cdot c_{nom} - 2 \cdot \Phi_{shear})}{3 \cdot \Phi_{main}}$	(3.41)	$n_{main}^{LO} > \frac{\pi \cdot (D - 2 \cdot c_{nom} - 2 \cdot \Phi_{shear})}{s_{r,max}}$	(3.42)
<i>Omejitve vrednosti spremenljivk</i>			
$\Phi_{main}^{LO} \leq \Phi_{main} \leq \Phi_{main}^{UP}$	(3.43)	$s_{shear}^{LO} \leq s_{shear} \leq s_{shear}^{UP}$	(3.44)
$L^{LO} \leq L \leq L^{UP}$	(3.45)	$D^{LO} \leq D \leq D^{UP}$	(3.46)

Tabela 5: Najmanjše in največje vrednosti diskretnih spremenljivk v optimizacijskem modelu.

Spremenljivka	Diskretne vrednosti spremenljivk	
	Najmanjša vrednost	Največja vrednost
L (m)	5	65
D (m)	0,6	1,5
Φ_{main} (mm)	10	30
n_{main} (-)	6	40
Φ_{shear} (mm)	8	14
s_{shear} (mm)	100	250

4 APLIKACIJA OPTIMIZACIJSKEGA MODELA NA VERTIKALNO OBREMENJENEM GEOTERMALNEM PILOTU V RAZLIČNIH ZEMLJINAH – ŠTUDIJA PRIMERA

Na podlagi prej definiranih enačb za analizo geotermalnega pilota smo izračunali optimalne zasnove geotermalnih in konvencionalnih pilotov za različne vrste zemljin. Zanimalo nas je, kako omejitev posedka in različne togosti zemljine vplivajo na optimalno zasnovo. Vhodni podatki za študijo primera so zbrani v spodnji tabeli (Tabela 6).

Tabela 6: Vhodni podatki za optimizacijski model – študija primera

Simbol	Opis	Vrednost
$V_{G,k}$	Stalna vertikalna obtežba	1000 / 4000 kN
$V_{Q,k}$	Spremenljiva vertikalna obtežba	0 kN
ΔT	Sprememba temperature	0 °C / 20 °C
$h_{nosilni}$	Globina nosilnega sloja (za stoječi geotermalni pilot)	17 m
D_{min}	Minimalni premer geotermalnega pilota	600 mm
k_{adh}	Adhezijski koeficient za trenje na plašču	0,5
ν_{soil}	Poissonovo razmerje	0,3
α_{cc}	Koeficienta dolgotrajnih učinkov obtežbe na tlačno trdnost betona	0,6
α_r	Reološki koeficient zemljine	0,33/0,5/0,66
n	Koeficient za izračun K_s (grobo in fino zrnata zemljina)	0,8/2
α_{pile}	Temperaturni koeficient linearnega raztezka	°C ⁻¹
ϕ'_k	Strižni kot zemljine	40/34/28/22/19°
c'_k	Kohezija	5/12/10 kPa
$\gamma_{soil,k}$	Prostorninska teža zemljine	23,5/23/22/20/19 kN/m ³
E_{soil}	Modul elastičnosti zemljine	400/40/15/8/4 MPa
E_{pile}	Modul elastičnosti geotermalnega pilota	30000 MPa
f_{ck}	Karakteristična vrednost tlačne trdnosti betona	30 MPa
c_{nom}	Krovni sloj betona	50 mm
$s_{r,max}$	Največji dovoljen razmik med palicami	200 mm
γ_{wc}	Prostorninska teža betona	24 kN/m ³
$c_{1,drill}$	Strošek za odstranitev zemljine	40 €/m
$c_{2,drill}$	Strošek zaradi večjega premera pilota	0,07 €/mm/m
c_{conc}	Strošek betona	100 €/m ³
c_{rein}	Strošek armature	1,1 €/kg
$c_{thermal}$	Strošek vgradnje sistema (cevi in zanke) za ogrevanje oziroma hlajenje	20 €/m
ρ_s	Gostota jekla	7850 kg/m ³

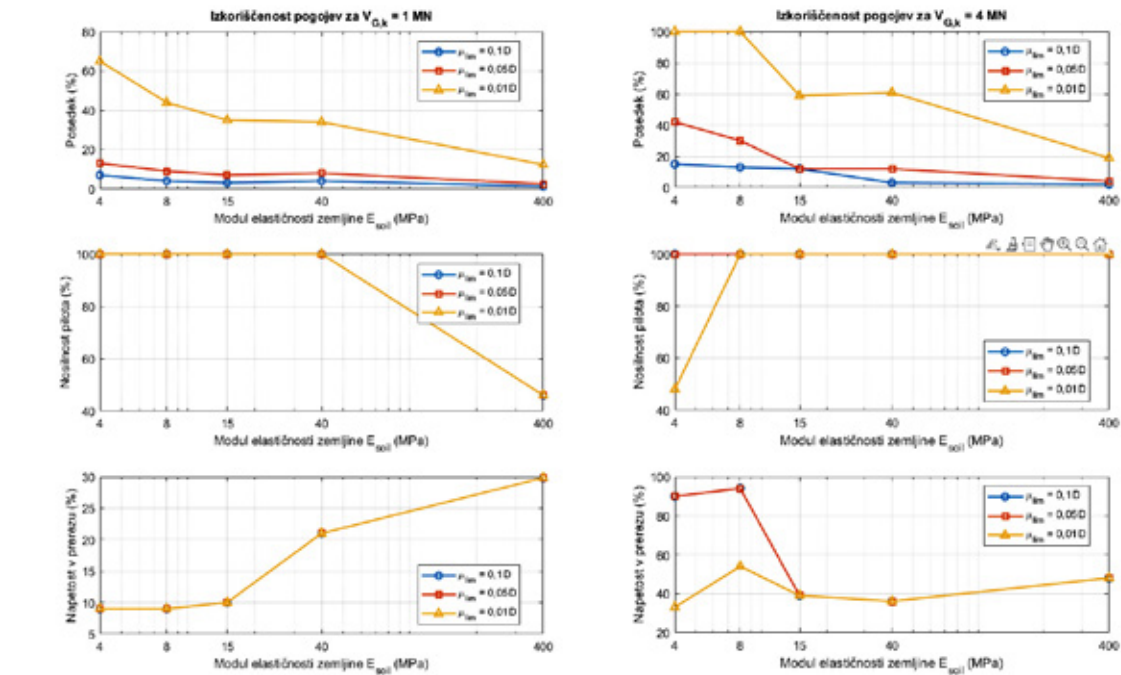
Simbol	Opis	Vrednost
	Delni količniki varnosti za PP2	PP2
SF_G	Delni količnik varnosti za stalne vplive	1,35
SF_φ	Delni količnik varnosti za strižni kot zemljine	1,0
SF_c	Delni količnik varnosti za kohezijo zemljine	1,0
SF_b	Delni količnik varnosti za odpornost konice pilota	1,1
SF_s	Delni količnik varnosti za odpornost ob plašču pilota	1,1
SF_m	Delni količnik varnosti za materialno lastnost zemljine	1,4

Optimizacije smo se lotili postopoma za različne vrednosti modula elastičnosti zemljine (E_{soil}) v kombinaciji z vplivom stalne vertikalne obtežbe ($V_{G,k}$) vrednosti 1 MN in 4 MN. Pri tem nas je zanimalo, kateri geotehnični in konstrukcijski pogoji so merodajni pri različnih vrstah zemljin, različnih vplivih sil in še omejitvijo posedkov geotermalnih in konvencionalnih pilotov z 0,1*D*; 0,05*D* in 0,01*D*. Omejitev posedkov smo uveljavili predvsem zato, ker – kot opozarja tuja literatura in priporočila za dimenzioniranje geotermalnih geokonstrukcij lahko posedek zaradi vpliva spremembe temperature bistveno vpliva na preverjanje mejnega stanja uporabnosti.

Največji posedki pri glavi geotermalnega pilota se pojavijo, kadar pride do krčenja, to pa se zgodi pri hlajenju. Takrat krčenje zaradi vpliva vertikalne obtežbe in vpliva temperature povzroči maksimalne posedke konstrukcije. To stanje smo potem analizirali pri trenjskem geotermalnem pilotu in pri stoječem geotermalnem pilotu. Prav tako je treba poudariti, da se za namene analize upošteva absolutna globina nosilnega sloja 17 m. Ko je v dobro nosilni zemljini dosežena nosilnost pilota že prej (zaradi trenja na plašču), se na tisti globini predpostavi, da je pilot osno togo vpet in se obravnava kot stoječi geotermalni pilot.

4.1 Analiza trenjskega geotermalnega pilota (Floating geothermal pile)

Na spodnji sliki (Slika 8) je prikazana izkoriščenost ključnih geotehničnih pogojev, med katerimi so posedek geotermalnega pilota, nosilnost na konici in vzdolž plašča ter osna napetost v armiranobetonskem prerezu pilota. Na podlagi analiz optimalnih zasnov in izkoriščenosti posameznih pogojev je razviden primerljiv trend pri obremenitvi z vertikalno silo 1 MN (Slika 8a) ter pri obremenitvi z večjo vertikalno silo 4 MN (Slika 8b). Pri obremenitvi 1 MN se izkoriščenost pogoja posedka giblje okrog 50 %, zlasti pri strožjih omejitvah posedka, medtem ko z naraščajočo togostjo tal (do 40 MPa) hitro upada. Pri večji obremenitvi (4 MN) doseže izkoriščenost pogoja posedka 100 % in postane merodajen pogoj, saj je pri togosti tal 4 MPa izkoriščenost nosilnosti manjša od 50 %. Napetosti, ki nastajajo v betonskem prerezu trenjskega geotermalnega pilota, v nobenem analiziranem primeru ne dosežejo kritičnih vrednosti.

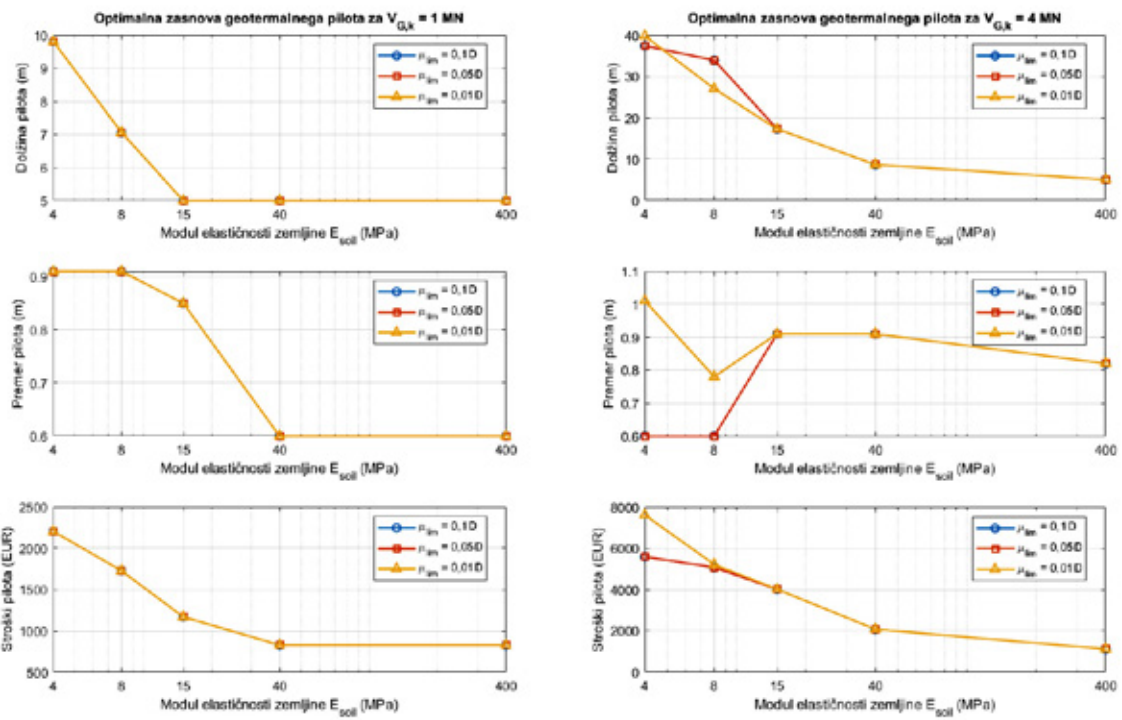


a)

b)

Slika 8: Izkoriščenost pogojev trenjskega geotermalnega pilota za a) obremenitev z vertikalno silo 1MN in b) obremenitev z vertikalno silo 4 MN.

Pripadajoče optimalne zasnove za obremenitvi 1 MN (Slika 9a) in 4 MN (Slika 9b) prav tako kažejo podoben trend, ki je pričakovano skladen z izkoriščenostjo posameznih pogojev. V zemljinah z modulom elastičnosti pod 15 MPa je optimalna zasnova daljši pilot, saj je za doseg zadostne nosilnosti pri mehanski obremenitvi treba povečati globino. To pa hkrati povzroči večje posedke, predvsem zaradi vpliva spremembe temperature, saj so ti posedki neposredno odvisni od dolžine pilota. Pri večji obremenitvi (4 MN) in majhnih vrednostih modula elastičnosti je opazen trend, da zaradi strožjega pogoja omejitve posedkov (0,01D) optimalna zasnova vključuje pilote z večjimi premeri, saj ti omogočajo doseganje večje nosilnosti ob dopustnem posedku. Iz tega logično sledi, da se stroški trenjskih geotermalnih pilotov pri večjih obremenitvah občutno povečajo. Ker v analizi večslojna tla niso bila upoštevana, kar bi bilo za natančnejšo oceno zasnove bolj ustrezno, so rezultati pokazali tudi na povečane optimalne dolžine pilotov. Za podrobnejšo analizo bi bilo tako smiselno izvesti modeliranje večslojnih tal, da bi se zmanjšale dolžine pilotov.

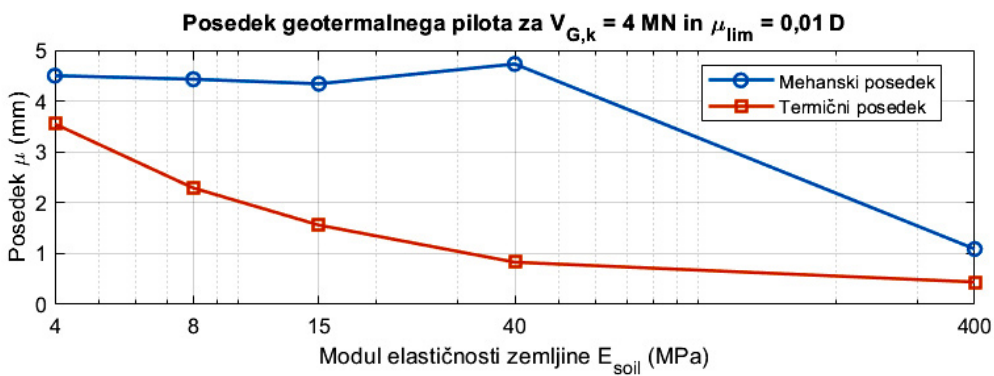


a)

b)

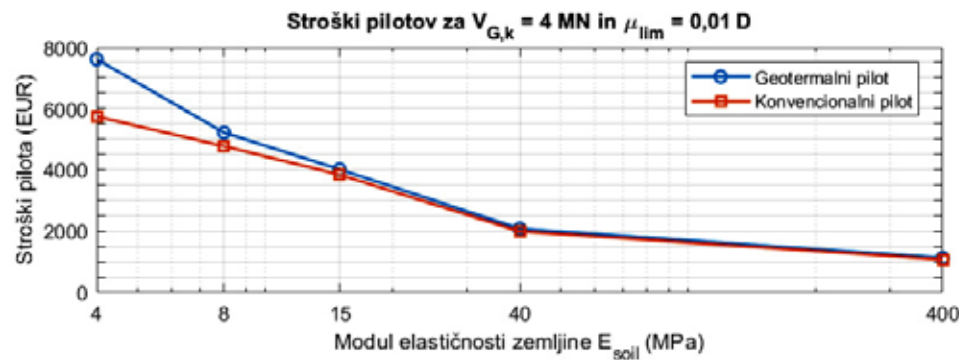
Slika 9: Optimalne zasnove trenjskega geotermalnega pilota za a) obremenitev z vertikalno silo 1MN in b) obremenitev z vertikalno silo 4 MN.

Spodnja slika (Slika 10) prikazuje razmerje med posedkom, ki nastane zaradi temperaturnih vplivov, in posedkom zaradi mehanske obremenitve pri omejitvi posedka na 0,01D. Iz dosedanjih analiz je bilo ugotovljeno, da je prav strožja omejitev posedkov kritična za zasnovo geotermalnih pilotov. Iz slike 10 je razvidno, da pri optimalnih zasnovah in izbranih vhodnih podatkih pri nizkem modulu elastičnosti tal (4 MPa) posedek zaradi temperaturnih vplivov predstavlja 55 % celotnega posedka. Zanimivo je tudi to, da v dobro nosilni zemljini z modulom elastičnosti 400 MPa posedek zaradi temperature še vedno dosega približno 40 % celotnega posedka, kljub manjši dimenziji pilota.



Slika 10: Razmerje med mehanskim in termičnim posedkom za trenjski geotermalni pilot.

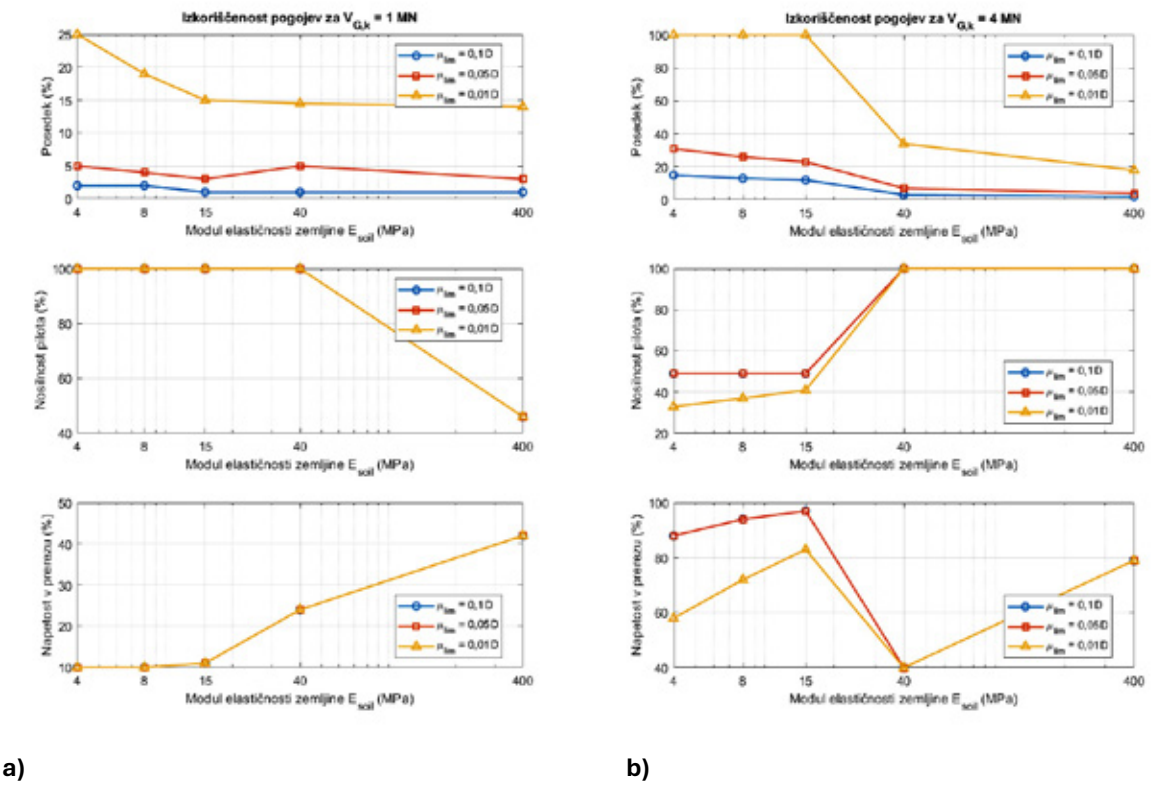
Slika 11 prikazuje primerjavo stroškov izgradnje konvencionalnega in geotermalnega pilota. V območju dobro nosilnih zemljin je dodatni strošek vgradnje geotermalnega sistema zanemarljiv, medtem ko je pri slabo nosilnih tleh (4 MPa) potreben podrobnejši premislek o geotermalnih potrebah in pričakovani izkoriščenosti sistema. V takšnih primerih je namreč smiselno oceniti, ali je dodatni strošek, ki znaša približno 1.900 €/pilot, upravičen glede na predvideno povratno dobo investicije in izgradnje ločenega sistema toplotne črpalke.



Slika 11: Stroški izgradnje optimalno zasnovanega geotermalnega in konvencionalnega trenjskega pilota.

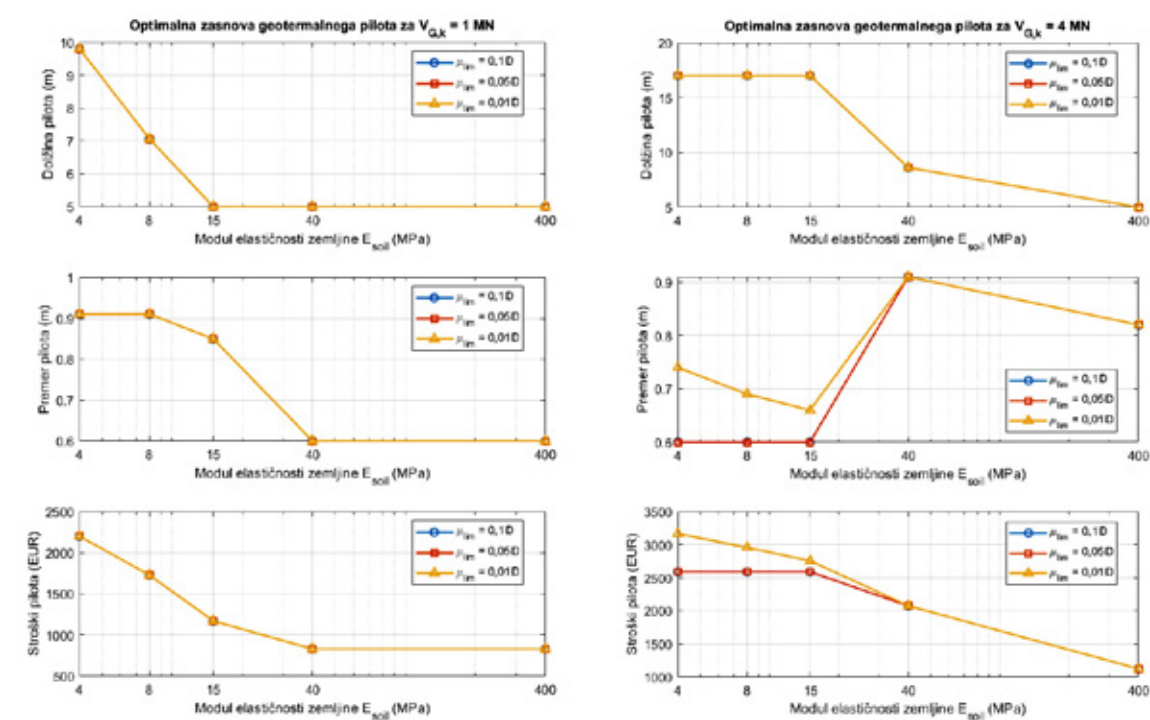
4.2 Analiza stoječega geotermalnega pilota (End bearing geothermal pile)

Na slikah (Slika 12a in Slika 12b) je prikazana izkoriščenost ključnih geotehničnih pogojev za primer stoječega pilota, po enaki metodologiji kot pri trenjskem pilotu. Na podlagi analiz optimalnih zasnov in izkoriščenosti posameznih pogojev je razvidno, da se pri obremenitvi z vertikalno silo 1 MN (Slika 12a) ter pri obremenitvi z večjo vertikalno silo 4 MN (Slika 12b) pojavlja bistveno večja razlika v izkoriščenosti pogoja posedka kot pri trenjskem pilotu. Pri obremenitvi 1 MN se izkoriščenost pogoja posedka pri manjših vrednostih elastičnega modula zemljine v povprečju giblje okoli 20 %, zlasti pri strožjih omejitvah posedka, pri čemer z naraščajočo togostjo tal (do 40 MPa) ne upade bistveno. Pri večji obremenitvi (4 MN) izkoriščenost pogoja posedka doseže 100 % in postane merodajen pogoj, saj je pri togosti tal 4 MPa izkoriščenost nosilnosti manjša od 30 %. Napetosti, ki nastajajo v betonskem prerezu geotermalnega pilota, pa v nasprotju s trenjskim pilotom dosegajo višje vrednosti in kažejo na skoraj merodajen pogoj, kar je glede na robne pogoje vpetja pilota pričakovan rezultat.



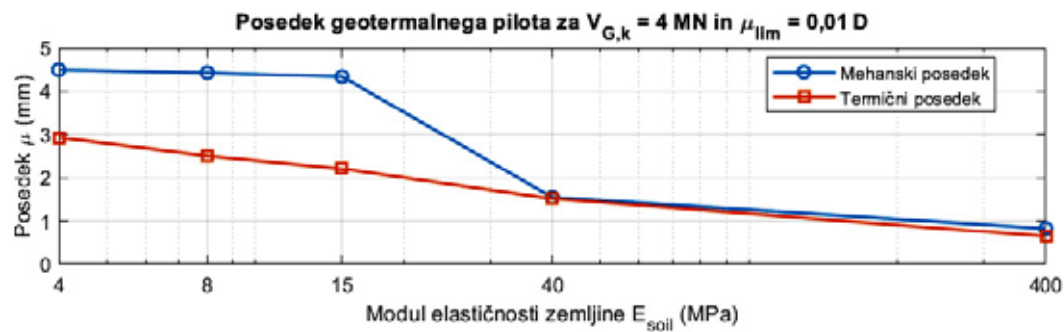
Slika 12: Izkoriščenost pogojev stoječega geotermalnega pilota za a) obremenitev z vertikalno silo 1MN in b) obremenitev z vertikalno silo 4 MN.

Pripadajoče optimalne zasnove na sliki 13a za obremenitev 1MN in sliki 13b za obremenitev 4 MN kažejo ujemanje s trendom izkoriščenosti pogojev. Prav tako je razvidno, da pri večjih obremenitev nosilnost na plašču ne zadošča in je zato pilot pri manjših vrednostih modula elastičnosti dolg 17 m, kar sovпада z zgornjo mejo nosilnega sloja. Enako kot pri trenjskem pilotu, vendar je v tem primeru to bolj vidno in izrazito, pri modulu elastičnosti 40 MPa, optimalna zasnova vsebuje večjo dimenzijo širine pilota in manjšo globino, kar je za dane vhodne podatke optimalni rezultat in ni indikator vpliva spremembe temperature, saj pogoj posedka v tem primeru ni kritičen. Ker se nosilnost pilota doseže prej, so stroški izgradnje pilota manjši, saj so potrebne manjše dimenzije (zlasti dolžine) pilotov za doseganje nosilnosti.



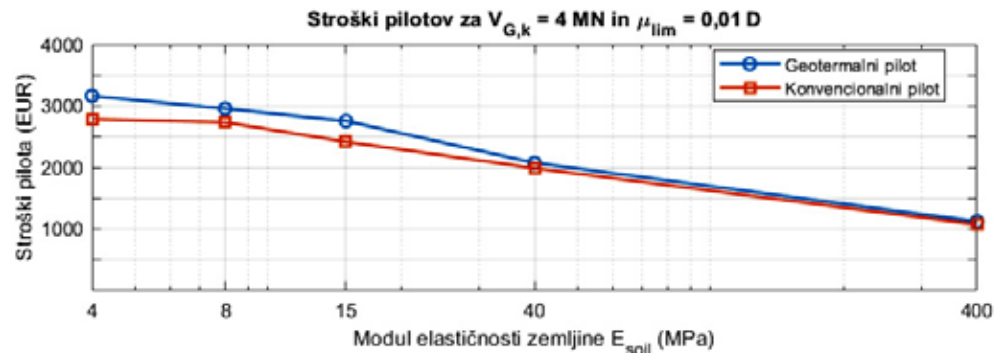
a) b) **Slika 13:** Optimalne zasnove stoječega geotermalnega pilota za a) obremenitev z vertikalno silo 1MN in b) obremenitev z vertikalno silo 4 MN.

Spodnja slika 14 prikazuje razmerje med posedkom, ki nastane zaradi temperaturnih vplivov, in posedkom zaradi mehanske obremenitve pri omejitvi posedka na 0,01D. Iz slike je razvidno, da pri optimalnih zasnovah in izbranih vhodnih podatkih pri nizkem modulu elastičnosti tal (4 MPa) posedek, povzročen s temperaturnimi vplivi, predstavlja kar 65 % celotnega posedka. Zanimivo je tudi, da v dobro nosilni zemljini z modulom elastičnosti 400 MPa termični posedek še vedno dosega približno 80 % celotnega posedka. Na splošno so večji posedki zaradi temperature značilni za pilote s togim vpetjem, in sicer zato, ker se t. i. »ničelna točka« pomika proti delu pilota, kjer je osna togost vpetja večja. Kar pri stojećem pilotu pomeni proti konici in s tem pride do večjega posedka pri glavi geotermalnega pilota.



Slika 14: Razmerje med mehanskim in termičnim posedkom za stoječi geotermalni pilot.

Ker je pri tej vrsti pilota razlika v dimenzijah med termično aktivnimi in neaktivnimi piloti manjša – nosilnost se namreč doseže hitreje, kar omogoča krajšo dolžino pilota in posledično manjši vpliv temperature – spremembe v tleh z nizkim modulom elastičnosti ne vplivajo bistveno na dimensioniranje. Zato je cenovna razlika med geotermalnimi in konvencionalnimi piloti zgolj približno 12 % pri nizkih modulih (4 MPa) in pada z naraščanjem togosti zemljine, kar kaže na njihovo smiselno uporabo tudi pri manjših geotermalnih potrebah. Vse skupaj je razvidno iz slike 15.



Slika 15: Stroški optimalno zasnovanega geotermalnega in konvencionalnega stoječega pilota.

5 ZAKLJUČEK

Analiza rezultatov je za dane predpostavke in omejitve modela pokazala določene razlike v obnašanju trenjskih pilotov in stoječih pilotov, tako z vidika posedkov kot nosilnosti in stroškov izgradnje. Pri trenjskih pilotih je izkoriščenost pogoja posedka pogosto merodajna, zlasti pri večjih obremenitvah in v zemljinah z nizkim modulom elastičnosti (4–15 MPa). Pri obremenitvi 4 MN je pogoj posedka kritičen, saj doseže 100 % izkoriščenost, medtem ko nosilnost na plašču v takšnih pogojih ne preseže 50 %. Termični posedki pri trenjskih pilotih predstavljajo znaten delež celotnega posedka – do 65 % v plastičnih tleh, pri čemer presenetljivo tudi v togih zemljinah (400 MPa) termični posedek dosega okoli 70–80 % celotnega posedka, vendar pa je treba poudariti, da posedek pri teh primerih ni kritičen in je izkoriščenost le približno 20 %. Vseeno pa to jasno potrjuje, da termika pomembno vpliva na obnašanje trenjskih geotermalnih pilotov in je ključna pri preverjanju mejnega stanja uporabnosti.

Optimalne zasnove trenjskih pilotov v slabše nosilnih tleh pri danih omejitvah, torej homogen sloj zemljine, zahtevajo večje dolžine (npr. 35 m), kar povzroča višje stroške, ki lahko narastejo za več kot 1.900 €/pilot, če imamo geotermalni pilot. To kaže na potrebo po natančnejšem modeliranju večslojnih tal, saj bi lahko upoštevanje realne geološke sestave omogočilo krajše in ekonomičnejše zasnove. V dobro nosilnih zemljinah pa razlika v stroških med geotermalnimi in konvencionalnimi piloti postane zanemarljiva, zaradi česa bi kljub geotermalnemu izračunu (potrebe/izkoristek energije) lahko predpostavili njihovo uporabnost.

Pri stojećih pilotih so trendi nekoliko drugačni. Zaradi prenosa obtežbe na nosilni sloj je posedek pri enakih pogojih bistveno manjši, izkoriščenost pogoja posedka pa se pri manjših obremenitvah (1 MN) giblje okoli 20 %. Napetosti v prerezu pilota so višje kot pri trenjskem pilotu in se približujejo merodajnim vrednostim, kar je pričakovano zaradi togih robnih pogojev na konici. Optimalne zasnove pri višjih moduli elastičnosti (40 MPa) vključujejo povečanje premera pilota in zmanjšanje dolžine, saj se nosilnost doseže pri manjših dimenzijah, kar vodi v nižje stroške.

Pomembno je poudariti, da sta pri obeh vrstah pilotov v zemljinah z nizkim modulom elastičnosti tako posedek zaradi mehanske obremenitve kot tudi posedek zaradi vpliva temperature kritična pogoja. Ob tem pa se pojavi velika razlika v stroških, saj trenjski pilot zaradi večjih dolžin zahteva tudi povečanje premera, da se zagotovi pogoj posedka (0,01D; 0,05D; 0,1D), kar vodi do približno 25 % višje cene geotermalnega pilota v primerjavi s konvencionalnim. Pri stojećem pilotu je ta cenovna razlika bistveno manjša, zgolj okoli 12 %.

Analiza je pokazala, da vpliv večjih temperaturnih sprememb ($\Delta T = 15^\circ\text{--}20^\circ$, v našem primeru 20°) nikakor ni zanemarljiv pri dimenzioniranju geotermalnih geokonstrukcij, saj se njihov vpliv povečuje z večanjem mehanske obremenitve in strožimi omejitvami posedkov. V obeh primerih posedek zaradi spremembe temperature predstavlja vsaj 45 % skupnega posedka pri nizkih vrednostih modula elastičnosti, kar pomembno vpliva na dimenzioniranje geotermalnih pilotov. Poleg tega lahko pri pilotih s togim vpetjem (osna togost) pride do opaznega povečanja osnih sil v armiranobetonskem prerezu, kar zahteva dodatno preverjanje konstrukcijske varnosti.

Stroškovna analiza je pokazala, da kljub povečanim dimenzijam ob upoštevanju temperaturnih vplivov cenovna razlika v nekaterih primerih ni velika. Z dodatnimi analizami potreb po geotermalni oskrbi in izkoristku sistema bi lahko bila izgradnja trenjskih geotermalnih pilotov upravičena tudi v zahtevnejših primerih ($E_{\text{soil}} \leq 10 \text{ MPa}$, $V \geq 4 \text{ MN}$, $u_{\text{lim}} \leq 0,03D$).

LITERATURA IN REFERENCE

- [1] Council of the European Union | European Parliament, “Directive (EU) 2023/2413 of the European Parliament and of the Council of 18 October 2023 amending Directive (EU) 2018/2001, Regulation (EU) 2018/1999 and Directive 98/70/EC as regards the promotion of energy from renewable sources, and repealing Council Directive (EU) 2015/652,” *Official Journal of the European Union*, Oct. 2023.
- [2] A. Kagel, D. Bates, and K. Gawell, “A guide to geothermal energy and the environment, geothermal energy association,” *Pennsylvania Avenue SE, Washington, DC*, no. April, 2005.
- [3] Joerg Prestor, D. Rajver, S. Pestotnik, and R. Benček, “Smernice za vrtanje v plitvi geotermiji (do 300m),” May 2021.
- [4] Ministrstvo za gospodarstvo, “Zakon o rudarstvu (ZRud-1),” Jan. 2011.
- [5] V. Phil, “Capturing the heat from below,” The green village. [Online]. Available: <https://thegreenvillage.org/project/energy-pile/>
- [6] D. Pahud and M. Hubbuch, “Measured Thermal Performances of the Energy Pile System of the Dock Midfield at Zürich Airport,” *Proceeding European Geothermal Congress 2007*, vol. 2, no. June, pp. 1–7, 2007.
- [7] S. Frodl, J. N. Franzius, and T. Bartl, “Design and construction of the tunnel geothermal system in Jenbach I,” *Geomechanics and Tunneling*, vol. 3, no. 5, pp. 658–668, Oct. 2010, doi: 10.1002/geot.201000037.
- [8] P. Bourne-Webb, S. Burlon, S. Javed, S. Kürten, and F. Loveridge, “Analysis and design methods for energy geostructures,” *Renewable and Sustainable Energy Reviews*, vol. 65, pp. 402–419, Nov. 2016, doi: 10.1016/j.rser.2016.06.046.
- [9] J. Fadejev, R. Simson, J. Kurnitski, and F. Haghighat, “A review on energy piles design, sizing and modelling,” *Energy*, vol. 122, pp. 390–407, Mar. 2017, doi: 10.1016/j.energy.2017.01.097.
- [10] R. Zagorščak, and R. Thomas, “A Review on Performance of Energy Piles and Effects on Surrounding Ground A review on performance of energy piles... A REVIEW ON PERFORMANCE OF ENERGY PILES AND EFFECTS ON SURROUNDING GROUND PREGLED NAČINA RADA ENERGETSKIH PILOTA I UTJECAJA NA OKOLNO TLO,” 2016. [Online]. Available: <https://www.researchgate.net/publication/307883481>
- [11] M. de Moel, P. M. Bach, A. Bouazza, R. M. Singh, and J. O. Sun, “Technological advances and applications of geothermal energy pile foundations and their feasibility in Australia,” *Renewable and Sustainable Energy Reviews*, vol. 14, no. 9, pp. 2683–2696, Dec. 2010, doi: 10.1016/j.rser.2010.07.027.
- [12] F. Loveridge and W. Powrie, “Pile heat exchangers: thermal behaviour and interactions,” *Proceedings of the Institution of Civil Engineers – Geotechnical Engineering*, vol. 166, no. 2, pp. 178–196, Apr. 2013, doi: 10.1680/geng.11.00042.
- [13] C. Knellwolf, H. Peron, and L. Laloui, “Geotechnical Analysis of Heat Exchanger Piles,” *Journal of Geotechnical and Geoenvironmental Engineering*, vol. 137, no. 10, pp. 890–902, 2011, doi: 10.1061/(asce)gt.1943-5606.0000513.
- [14] X. Huang, Y. Wu, H. Peng, Y. Hao, and C. Lu, “Thermomechanical Behavior of Energy Pile Embedded in Sandy Soil,” *Math Probl Eng*, vol. 2018, pp. 1–11, 2018, doi: 10.1155/2018/5341642.
- [15] P. Fang, A. Bouazza, Z. Wang, and X. Xie, “Bearing Performance of Geothermal Energy Pile Subjected to Thermal Loading,” in *Proceedings of GeoShanghai 2018 International Conference: Tunneling and Underground Construction*, Singapore: Springer Singapore, 2018, pp. 710–717, doi: 10.1007/978-981-13-0017-2_71.
- [16] D. Wang, L. Lu, and P. Cui, “Simulation of thermo-mechanical performance of pile geothermal heat exchanger (PGHE) considering temperature-depend interface behavior,” *Appl Therm Eng*, vol. 139, pp. 356–366, Jul. 2018, doi: 10.1016/j.applthermaleng.2018.02.020.

- [17] J. Luo, H. Zhao, S. Gui, W. Xiang, and J. Rohn, "Study of thermal migration and induced mechanical effects in double U-tube energy piles," *Comput Geotech*, vol. 91, pp. 1–11, Nov. 2017, doi: 10.1016/j.compgeo.2017.06.015.
- [18] X. H. Bao, Y. L. Xiong, H. Y. Mingi, H. Z. Cui, G. B. Liu, and R. Y. Zheng, "Investigation on the Thermo-Mechanical Behavior of an Energy Pile and the Surrounding Soil by Model Test and 2D Finite Element-Finite Difference Method," in *Proceedings of GeoShanghai 2018 International Conference: Tunnelling and Underground Construction*, Singapore: Springer Singapore, 2018, pp. 696–709. doi: 10.1007/978-981-13-0017-2_70.
- [19] N. Yavari, A. M. Tang, J.-M. Pereira, and G. Hassen, "Experimental study on the mechanical behaviour of a heat exchanger pile using physical modelling," *Acta Geotech*, vol. 9, no. 3, pp. 385–398, Jun. 2014, doi: 10.1007/s11440-014-0310-7.
- [20] P. J. Bourne-Webb and T. M. Bodas Freitas, "Thermally-activated piles and pile groups under monotonic and cyclic thermal loading—A review," *Renew Energy*, vol. 147, pp. 2572–2581, Mar. 2020, doi: 10.1016/j.renene.2018.11.025.
- [21] P. J. Bourne-Webb, B. Amatya, K. Soga, T. Amis, C. Davidson, and P. Payne, "Energy pile test at Lambeth College, London: geotechnical and thermodynamic aspects of pile response to heat cycles," *Géotechnique*, vol. 59, no. 3, pp. 237–248, Apr. 2009, doi: 10.1680/geot.2009.59.3.237.
- [22] G. Dalla Santa *et al.*, "An updated ground thermal properties database for GSHP applications," *Geothermics*, vol. 85, p. 101758, May 2020, doi: 10.1016/j.geothermics.2019.101758.
- [23] H. Brandl, "Energy foundations and other thermo-active ground structures," *Géotechnique*, vol. 56, no. 2, pp. 81–122, Mar. 2006, doi: 10.1680/geot.2006.56.2.81.
- [24] L. Laloui and A. Di Donna, "Understanding the behaviour of energy geo-structures," *Proceedings of the Institution of Civil Engineers – Civil Engineering*, vol. 164, no. 4, pp. 184–191, Nov. 2011, doi: 10.1680/cien.2011.164.4.184.
- [25] F. Dupray, L. Laloui, and A. Kazangba, "Numerical analysis of seasonal heat storage in an energy pile foundation," *Comput Geotech*, vol. 55, pp. 67–77, Jan. 2014, doi: 10.1016/j.compgeo.2013.08.004.
- [26] M. Sutman, C. G. Olgun, and L. Laloui, "Cyclic Load-Transfer Approach for the Analysis of Energy Piles," *Journal of Geotechnical and Geoenvironmental Engineering*, vol. 145, no. 1, Jan. 2019, doi: 10.1061/(ASCE)GT.1943-5606.0001992.
- [27] R. Saggu, "Strain Distribution in Geothermal Energy Piles: A Parametric Study," 2019, pp. 129–137. doi: 10.1007/978-981-13-2227-3_16.
- [28] M. Alberdi-Pagola, S. Madsen, R. Jensen, and S. Poulsen, "Numerical Investigation on the Thermo-mechanical Behavior of a Quadratic Cross Section Pile Heat Exchanger," in *Proceedings of the IGSHA Technical/Research Conference and Expo 2017*, International Ground Source Heat Pump Association, 2017. doi: 10.22488/okstate.17.000520.
- [29] P. Jelušič and B. Žlender, "Determining optimal designs for conventional and geothermal energy piles," *Renew Energy*, vol. 147, pp. 2633–2642, Mar. 2020, doi: 10.1016/j.renene.2018.08.016.
- [30] R. Varga, B. Žlender, and P. Jelušič, "Optimising energy piles: a multi-objective approach to cost and failure probability," *Georisk: Assessment and Management of Risk for Engineered Systems and Geohazards*, pp. 1–18, Apr. 2025, doi: 10.1080/17499518.2025.2485484.
- [31] P. Bayer, M. de Paly, and M. Beck, "Strategic optimization of borehole heat exchanger field for seasonal geothermal heating and cooling," *Appl Energy*, vol. 136, pp. 445–453, Dec. 2014, doi: 10.1016/j.apenergy.2014.09.029.
- [32] G. A. Narsilio, "Cost Effectiveness of Energy Piles in Residential Dwellings in Australia," *Current Trends in Civil & Structural Engineering*, vol. 3, no. 3, pp. 1–6, 2019, doi: 10.33552/ctcse.2019.03.000564.
- [33] GSHP Association, "Thermal pile design, installation and materials standards," *Ground Source Heat Pump Association, Milton Keynes*, no. 1, p. 82, 2012.
- [34] L. Laloui and A. F. Rotta Loria, *Analysis and Design of Energy Geostructures*. Elsevier, 2020. doi: 10.1016/C2017-0-04166-6.
- [35] C. Bohn, A. Lopes dos Santos, and R. Frank, "Development of Axial Pile Load Transfer Curves Based on Instrumented Load Tests," *Journal of Geotechnical and Geoenvironmental Engineering*, vol. 143, no. 1, Jan. 2017, doi: 10.1061/(asce)gt.1943-5606.0001579.
- [36] R. Frank and S. R. Zhao, "Estimation par les paramètres pressiométriques de l'enfoncement sous charge axiale de pieux forés dans des sols fins," vol. 119, Nov. 1982.
- [37] Slovenski inštitut za standardizacijo, "SIST EN 1997-1-1:2005, Evrokod 7, Geotehnično projektiranje 1 del, Splošna pravila," 2005, *SIST, Ljubljana*.
- [38] Slovenski inštitut za standardizacijo, "SIST_EN_1991-1-1_2004, Evrokod 1, Vplivi na konstrukcije," 2004.
- [39] "EN 1992-1-1: Eurocode 2: Design of concrete structures – Part 1-1: General rules and rules for buildings," 2004.
- [40] K. Deep, K. P. Singh, M. L. Kansal, and C. Mohan, "A real coded genetic algorithm for solving integer and mixed integer optimization problems," *Appl Math Comput*, vol. 212, no. 2, pp. 505–518, 2009, doi: 10.1016/j.amc.2009.02.044.
- [41] M. M. Fernandes, *Analysis and design of geotechnical structures*. First edition. | Abingdon, Oxon ; Boca Raton, FL : CRC Press, 2020.: CRC Press, 2020. doi: 10.1201/9780429398452.
- [42] A. S. Vesić, "DESIGN OF PILE FOUNDATIONS," *NCHRP Synthesis of Highway Practice*, 1977.
- [43] D. Perić, A. E. Cossel, and S. A. Sarna, "Analytical Solutions for Thermomechanical Soil Structure Interaction in End-Bearing Energy Piles," *Journal of Geotechnical and Geoenvironmental Engineering*, vol. 146, no. 7, Jul. 2020, doi: 10.1061/(asce)gt.1943-5606.0002269.
- [44] P. J. Bourne-Webb, B. Amatya, and K. Soga, "A framework for understanding energy pile behaviour," *Proceedings of the Institution of Civil Engineers: Geotechnical Engineering*, vol. 166, no. 2, pp. 170–177, 2013, doi: 10.1680/geng.10.00098.
- [45] P. J. Bourne-Webb, B. Amatya, K. Soga, T. Amis, C. Davidson, and P. Payne, "Energy pile test at Lambeth College, London: geotechnical and thermodynamic aspects of pile response to heat cycles," *Géotechnique*, vol. 59, no. 3, pp. 237–248, Apr. 2009, doi: 10.1680/geot.2009.59.3.237.
- [46] A. F. R. Loria and L. Laloui, "Thermo-mechanical Schemes for Energy Piles," *Springer Series in Geomechanics and Geoengineering*, 2018, [Online]. Available: <https://api.semanticscholar.org/CorpusID:117379884>
- [47] B. L. AMATYA, K. SOGA, P. J. BOURNE-WEBB, T. AMIS, and L. LALOUI, "Thermo-mechanical behaviour of energy piles," *Géotechnique*, vol. 62, no. 6, pp. 503–519, Jun. 2012, doi: 10.1680/geot.10.P116.
- [48] P. J. Bourne-Webb, B. Amatya, K. Soga, T. Amis, C. Davidson, and P. Payne, "Energy pile test at Lambeth College, London: geotechnical and thermodynamic aspects of pile response to heat cycles," *Géotechnique*, vol. 59, no. 3, pp. 237–248, Apr. 2009, doi: 10.1680/geot.2009.59.3.237.
- [49] L. Laloui, M. Nuth, and L. Vulliet, "Experimental and numerical investigations of the behaviour of a heat exchanger pile," *Int J Numer Anal Methods Geomech*, vol. 30, no. 8, pp. 763–781, Jul. 2006, doi: 10.1002/nag.499.



GALENA DEBEVEC JORDANOVA

Dr. Galena Debevec Jordanova se je že kot magistrska študentka in pozneje kot mlada raziskovalka na Naravoslovnotehniški fakulteti Univerze v Ljubljani usmerila v raziskovanje pobočnih masnih premikov. Sodelovala je pri nadgradnji sistema za zgodnje opozarjanje pred plitvimi zemeljskimi plazovi MASPREM ter pri raziskavah z domačimi in tujimi sodelavci, s katerimi je objavila več znanstvenih in strokovnih prispevkov.

Med študijem se je dodatno izobraževala na področju tehnologije InSAR na tečajih Akademije ESA in na Univerzi v Aachnu, za področje kamninskih plazov in podorov pa se je udeležila mednarodne poletne šole v Kirgiziji. O uporabi tehnologije InSAR v geologiji za opazovanje pobočnih premikov je predavala na številnih konferencah in simpozijih, med drugim na konferenci evropskega projekta TransCPEarlyWarning v Varaždinu.

Na temo uporabe tehnologije InSAR v geologiji—natančneje njene uporabe za spremljanje plazenja—je leta 2025 doktorirala na študijskem programu Grajeno okolje na Fakulteti za gradbeništvo in geodezijo Univerze v Ljubljani. Ob redni zaposlitvi na Ministrstvu za naravne vire in prostor, kjer deluje na področju rudarstva, sodeluje tudi pri projektih s svojega raziskovalnega področja. Trenutno sodeluje pri projektu monitoringa plazov Slano blato z uporabo tehnologije InSAR.

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UPORABA TEHNOLOGIJE INSAR ZA SPREMLJANJE STABILNOSTI POBOČIJ IN OBJEKTOV

Povzetek

V članku sta predstavljena dva primera uporabe metode SBAS InSAR za spremljanje sprememb površja na plazovih Slano blato nad Lokavcem in Šumljak na Rebrnicah v Vipavski dolini. Analiza površinskih sprememb je omogočila vpogled v prostorsko aktivnost plazov ter v časovni razvoj površinskih sprememb v obdobju med januarjem 2017 in avgustom 2022 (za Šumljak) oz. marcem 2023 (za Slano blato). Pri plazu Slano blato je iz časovne analize razviden vpliv intenzivnejših padavinskih dogodkov na spremembe površja. Nasprotno pa je bila analiza površja plazov Šumljak otežena zaradi prisotnosti šuma (signala), ki je posledica goste vegetacije na območju plazov. Kljub temu podatki za viadukt Šumljak na hitri cesti, ki leži na plazu, kažejo na izrazit vpliv temperaturnih sprememb na konstrukcijo (ekspanzija in kontrakcija), hkrati pa je zaznan tudi trend posedanja, najverjetneje povezan s premiki plazov. Rezultati za plaz Slano blato so bili preverjeni s terenskim delom, kjer so bile opažene sledi aktivnega plazenja na robovih plazov (pojav usadov in plitvih zdrsov) ter izrazita površinska erozija, na nekaterih delih plazov pa so potekala tudi sanacijska dela. Rezultati za viadukt Šumljak pa bodo dodatno preverjeni z rezultati geodetskih izmer.

Ključne besede:

SBAS InSAR, analiza časovnih vrst, pobočni premiki, viadukt, Slano blato, Rebrnice

USING INSAR TECHNOLOGY FOR MONITORING THE STABILITY OF SLOPES AND STRUCTURES

Abstract

This paper presents two case studies employing the SBAS InSAR technique to investigate surface displacement associated with the Slano blato and Šumljak landslides in the Vipava Valley, SW Slovenia. The surface deformation analysis provides insight into both the spatial distribution of landslide activity and the temporal evolution of surface displacement between January 2017 and August 2022 (for Šumljak) and March 2023 (for Slano blato). In the case of the Slano blato landslide, the temporal deformation patterns clearly reflect the influence of more intense rainfall events. Conversely, analyzing surface displacement at the Šumljak site proved challenging due to signal noise caused by dense vegetation. However, the SBAS InSAR data for the nearby highway viaduct revealed patterns consistent with thermal expansion and contraction of the structure, along with a gradual subsidence trend most likely related to underlying landslide movement. Field surveys validated the SBAS InSAR results for the Slano blato landslide, confirming visible signs of sliding activity and significant surface erosion, with some areas undergoing active rehabilitation related construction work. Validation of the Šumljak viaduct displacement data is still pending and will be validated by results of other geodetic measurements.

Key words:

SBAS InSAR, time-series analysis, slope movements, viaduct, Slano blato, Rebrnice

1 UVOD

Opazovanje (ne)stabilnosti pobočij in objektov je kompleksna naloga, ki zahteva natančne in pogoste meritve za ugotavljanje kinematike deformacij ter vpliva zunanjih faktorjev na morebitne premike, zlasti kadar je ogrožena varnost prebivalcev in/ali infrastrukture. Pri opazovanju pobočnih masnih premikov se praktično zmeraj pojavi vprašanje, kako čim bolj učinkovito pridobiti čim več podatkov za čim večjo površino, medtem ko je za objekte prav tako pomembna tudi natančnost meritev.

V tujini se je za različne namene uveljavila tehnologija satelitske radarske interferometrije (*Interferometric SAR*, InSAR), ki omogoča natančnost meritev v mm/leto. Za opazovanje kinematike pobočnih masnih premikov se pogosto uporablja kot samostojna metoda [1], [2], [3], [4] ali v kombinaciji z drugimi metodami, kot so npr. GNSS meritve [5], [6], [7], [8], ALS [9], fotogrametrija z brezpilotnim letalnikom [10], [11], [12] in druge metode.

V Sloveniji je bil InSAR v preteklosti uporabljen v nekaj primerih opazovanja deformacij površja zaradi pobočnih premikov ali potresov, npr. pri določanju posledic plazov Stože v Logu pod Mangartom [13], za ugotavljanje posredkov v Julijskih Alpah zaradi tektonskega dogajanja [14], za opazovanje posredkov ob potresu v Posočju leta 1998 [15], za ugotavljanje premikov plazov na odlagališču jalovine Boršt na Žirovskem vrhu [16], za ugotavljanje plazljivih območij v Škofjeloško-Cerkljanskem hribovju [17] ter na plazov Potoška planina [5], za namen dolgoročnega spremljanja dinamike plazov pa je bila uporabljena pri opazovanju plazljivih območij na Rebrnicah, Čemšeniku in Potoški planini [18]. V vseh primerih opazovanja plazov so bili satelitski posnetki obdelani po metodi permanentnih sipalcev (*Permanent Scatterer InSAR*, PS InSAR) in le v dveh primerih je bila uporabljena metoda diferencialnega InSAR (*Differential InSAR*, DInSAR).

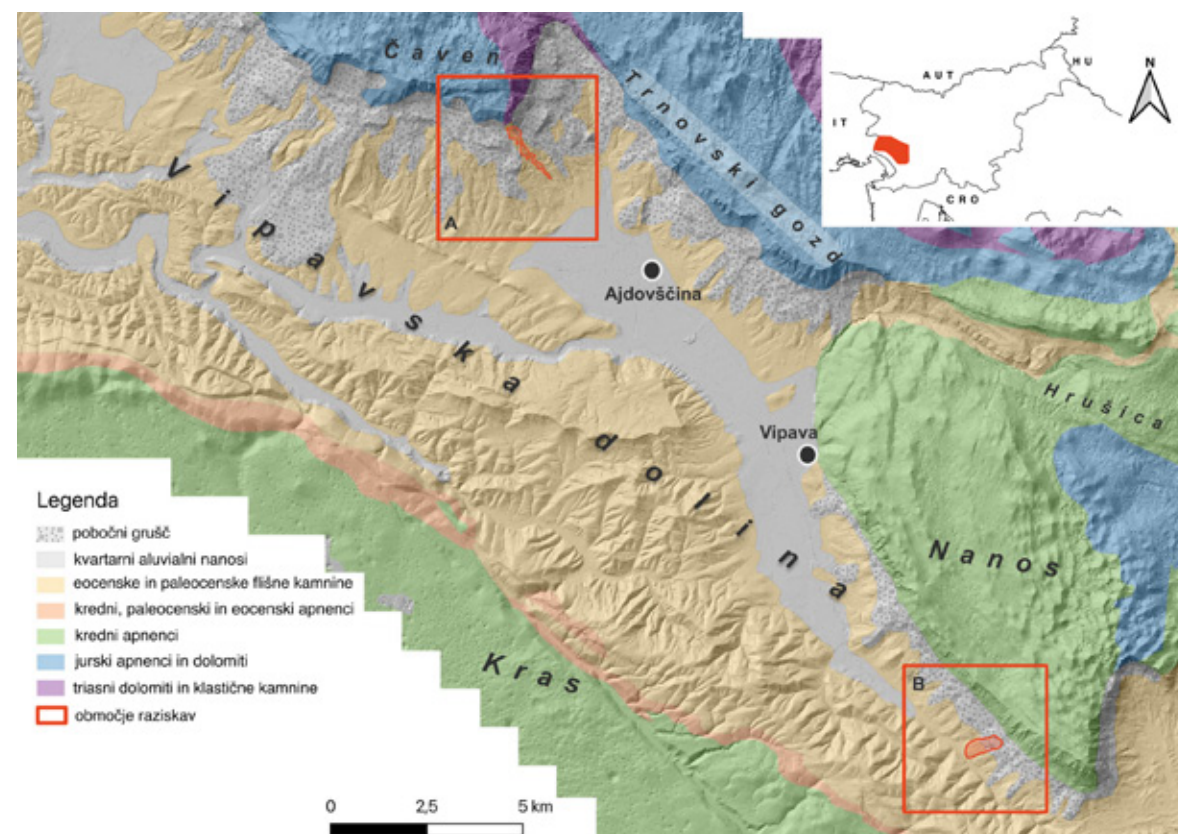
V nadaljevanju tega članka sta predstavljena dva primera uporabe tehnologije InSAR za opazovanje pobočnih masnih premikov in stabilnosti objektov, in sicer na primeru plazov Slano blato nad Lokavcem in plazov Šumljak na Rebrnicah v Vipavski dolini. Rezultati predstavljajo pomembnejše izsledke doktorske disertacije Debevec Jordanove [12], v kateri je bil uporabljen InSAR za dolgoročni monitoring pobočnih premikov na omenjenih pilotnih območjih ob uporabi spremljevalnih metod. Predstavljeni so rezultati obdelave podatkov po metodi SBAS InSAR (diferencialna metoda z majhnimi bazami), ki je bila prvič uporabljena za opazovanje pobočnih premikov v Sloveniji.

2 OBMOČJE RAZISKAV

Geologijo Vipavske doline so zaznamovali večji regionalni dogodki, ki so površje oblikovali v današnji značilni relief (slika 1). Eocenske flišne kamnine, ki se pojavljajo v Vipavski dolini, pripadajo Snežniški narivni grudi, na katero so narinjeni mezozojski karbonati Trnovskega pokrova in Nanosa ter mezozojske, paleocenske in eocenske kamnine Hrušiskega pokrova. Terciarno narivanje karbonatov na fliš v dinarski smeri je tako izoblikovalo današnji relief, kjer lahko opazujemo mlajše fliše v nižjih nadmorskih legah z bolj položnim naklonom, nad katerimi se dvigajo strma pobočja starejših mezozojskih karbonatnih kamnin [19], [20].

Geološka zgradba Vipavske doline je iz vidika pobočnih masnih premikov problematična zaradi več razlogov:

- narinjene karbonatne kamnine nad flišnimi pobočji predstavljajo izvor materiala, ki se kopiči pod ostenji karbonatnih pokrovov in nato transportira proti dnu doline z različnimi mehanizmi masnega transporta;
- iz hidrogeološkega vidika predstavljajo karbonatni pokrovi velike zalogovnike podzemne vode, saj so karbonatne kamnine značilno precej zakrasele, zajemajo meteorne vode na površju in jih nato po kraških sistemih preusmerjajo v več smeri;
- flišne kamnine hitro preperevajo, preperina pa je zemljina s slabimi geomehanskimi lastnostmi, kar v kombinaciji s podzemno vodo iz kraškega zaledja ustvarja idealne pogoje za pobočne masne premike.



Slika 1: Geografska lokacija in poenostavljena litostratigrafska karta Vipavske doline (prirejeno po [21] in [22]) na lidarskem digitalnem modelu reliefa z označenima območjema InSAR analize (rdeča kvadrata; A – plaz Slano blato, B – plaz Šumljak).

2.1 Plaz Slano blato

Plaz Slano blato je eden večjih plazov v Sloveniji. Njegova ocenjena prostornina je med 700.000 in 1.000.000 m³, dolžina okoli 1,5 km in širina med 50 in 300 m. Globina drsne ploskve je okoli 10 do 13 m znotraj fliša, kjer pa se pojavljajo karbonatni gruščji, je tam globina drsne ploskve manjša, od 3 do 8 m. Slano blato je dokaj specifičen plaz v širši okolici zaradi svoje geološke zgradbe in tudi pojavov relativno redkih mineralov.

Plazina je sicer sestavljena praktično v celoti iz fliša oz. njegove preperine, specifična težava pa nastopa zaradi regionalne geološke zgradbe. Na severnem delu plazu, nad vodnjaki namreč poteka narivna ploskev, kjer so apnenci in podrejeno dolomiti Trnovskega gozda narinjeni na eocenski fliš. Poleg tega predstavlja območje Male Gore oz. južnega dela Čavna večji karbonatni blok, ki je z globoko razpoko ločen od matičnega dela zaledja Čavna. Posledično je zaradi kombinacije posedlega bloka in narivnice prišlo do t.i. strukturne kadunje [20], kjer se padavinska voda, ki se infiltrira v zaledju Čavna, pretaka skozi razpokane karbonate in se ujame v to strukturno kadunjo, nato pa počasi izteka po globlje ležečih plasteh razpokanih peščenjakov v zgornjem delu plazu nad AB vodnjaki. Plaz so intenzivno raziskovali v letih po reaktivaciji novembra 2000 [23], [24], takrat se je začela tudi sanacija z večjimi gradbenimi deli (druga večja faza sanacije se zaključuje letos), nikoli pa ni bila vzpostavljena sistematična spremljava premikov na plazu.

2.2 Plaz Šumljak (Rebrnice)

Plaz Šumljak je večinoma sestavljen iz flišnega deluvija (glinasti in meljasti grušč) ter karbonatnega grušča s klasti dimenzij 10 cm, ki so delno sprijeti v breče. Matična podlaga je eocenski fliš s plastmi vpada 10° do 40°, lokalno tudi do 60° proti SV. Plasti, zlasti glinene in meljaste plasti, so lokalno zelo tektonsko deformirane. Drsna ploskev je na kontaktu med gruščem in deluvijem na globini 15 do 30 m. Karbonatni grušč je bil transportiran in odložen v t.i. paleograpo, ki je bila vrezana v flišno podlago [25], [26], [27], posledično pa je globina grušča največja vzdolž centralne osi plazu. Paleograpa služi tudi kot indikator pretakanja podzemne vode in predstavlja glavno območje toka podzemne vode [28].

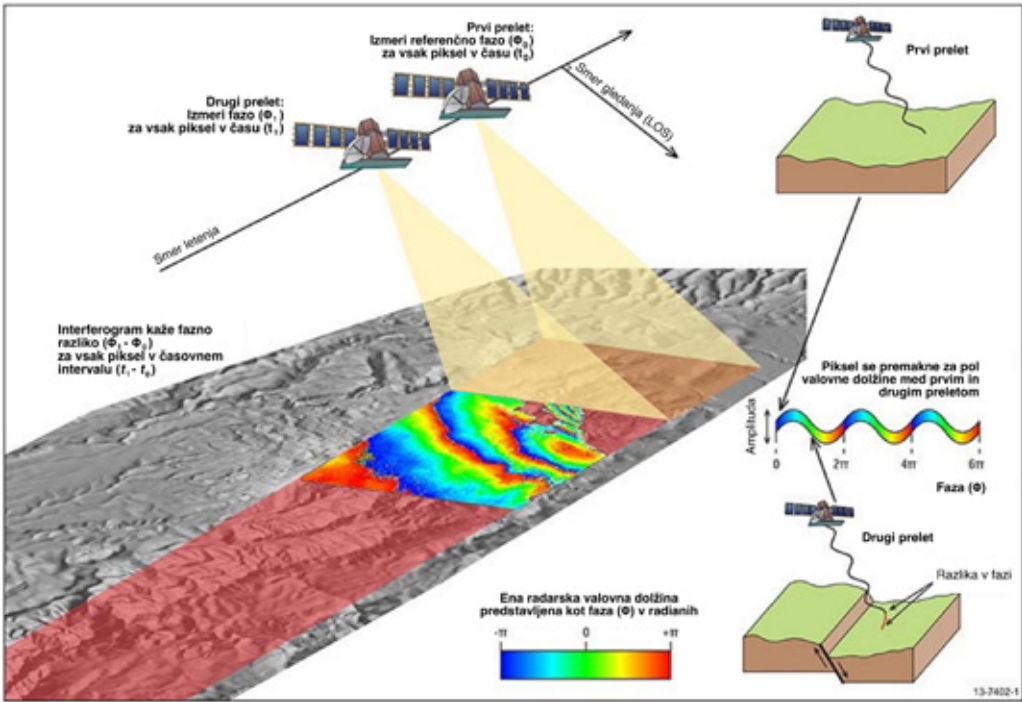
3 METODOLOGIJA

3.1 Osnovni princip posnetkov SAR in interferometrije

Radarska interferometrija (*Interferometric SAR*, InSAR) je metoda, s katero pridobivamo visoko-ločljivostne podatke o Zemljinem površju in se uspešno uporablja na številnih področjih.

Posnetek SAR je sestavljen kot raster, pri katerem vsak piksel (celica) predstavlja en del Zemljinega površja (resolucijski piksel) in poda kompleksno število, ki vsebuje podatke o amplitudi (intenziteti) in fazi odbitega valovanja mikrovalov, ki so se odbili od različnih sipalcev na površju – skal, vegetacije, stavb, cest ipd. Sipanje mikrovalov na površju se odvija različno v odvisnosti od materiala in kota odboja [29], [30]. Jakost odbitega elektromagnetnega (EM) valovanja je odvisna od več faktorjev, najpomembnejši pa so krajevni vpadni kot, razgibanost terena ter prevodnost in dielektričnost površja. Vpadni kot je odvisen od naklona terena in zornega kota satelita [29].

V primeru dveh preletov, ko isti radarski sistem posname isto območje dvakrat, interferogram predstavlja razliko razdalj med anteno in tarčo (slika 2). Faza (ang. *phase*) pomeni potovalni čas radarskega žarka od antene do opazovanega površja in nazaj do sprejemnika. V primeru dveh posnetkov zaporednih preletov tako opazujemo fazno razliko obeh posnetkov in jo lahko povežemo z višinami na površju [29]. Interferogram se izdela z razvojem faze z natančno koregistracijo posnetkov in skozi zapleten postopek množenja vrednosti pikslov prvega posnetka s konjugirano vrednostjo pikslov drugega posnetka [15]. Posnetka morata ustrezati pogojem za interferometrično obdelavo, t.j. da se faza med obema posnetkoma ni bistveno spremenila [29]. Faza, ki je prikazana na interferogramu, je modul absolutne faze. Proces določanja absolutne faze iz izmerjene (relativne) faze je fazni razvoj – ang. *phase unwrapping* [15].



Slika 2: Radarska interferometrija – fazna razlika pri dveh preletih (prirejeno po [31]).

Vsak kolobar (ang. *fringe*) na interferogramu predstavlja fazo v pasu od $-\pi$ do $+\pi$ ali en polni barvni krog (slika 2), kar je polovica valovne dolžine radarskega žarka [32]. Pri določanju celotnih premikov se kolobarje prešteje in sešteje. Fazna razlika se določa le v smeri gledanja satelita (ang. *Line of Sight*, LOS). Konstelacija Sentinel-1 snema iz dveh orbit – dvigajoče (ang. *ascending*) in spuščajoče (ang. *descending*).

3.2 Obdelava podatkov SAR

Obdelava podatkov je potekala v specializiranem programskem orodju ENVI SARscape 5.6.3 (Exelis Visual Information Solutions, Boulder, Colorado).

Za analizo smo uporabili prosto dostopne podatke satelitov Sentinel-1 programa Copernicus, ki jih preko Evropske vesoljske agencije (ESA) zagotavlja Evropska Unija. Posnetki so prosto dostopni za nalaganje na strani Aljaškega satelitskega oddelka (*Alaska Satellite Facility*, ASF) na povezavi <https://asf.alaska.edu>. Uporabljeni so bili posnetki v obliki *Single Look Complex* (SLC). Posnetki pokrivajo zelo veliko območje zaradi širokopasovnega snemanja satelitov, pri čemer je pas širine 250 km in producira posnetke z ločljivostjo $5\text{ m} \times 20\text{ m}$ (v radarskem koordinatnem sistemu). Velikost posameznega posnetka je iz tega razloga med 4 in 8 GB, kar za daljše obdobje lahko zahteva tudi do 1 TB prostora za shranjevanje, za obdelavo pa še vsaj 1 TB. Zaradi velike količine podatkov je obdelava lahko izjemno zamudna, v kolikor strojna oprema ni ustrezna. V tem primeru je bila uporabljena delovna postaja s 128 GB RAM delovnega spomina, procesorjem 13th Gen IntelCore i9-13900K 3.00 GHz ter 13 TB SSD notranjega spomina, kar je omogočilo obdelavo v relativno kratkem času (nekaj dni).

Tabela 1: Osnovni podatki o programski opremi in posnetkih SAR.

Območje	Plaz Slano blato	Plaz Šumljak
Obdobje analize	januar 2017 – marec 2023	januar 2017 – avgust 2022
Različica programske opreme	SARscape 5.6.3	
Konstelacija	Sentinel-1 (ESA)	
Kanal snemanja satelita	C-kanal, 5,6 GHz ($\lambda \gg 5,6\text{ cm}$)	
Oblika posnetkov	Single Look Complex (SLC)	
Velikost posnetkov (GB)	od 4 do 8 GB	
Relativna orbita in št. okvirja za ASC* orbito	147. okvir 44. relativne orbite	
Relativna orbita in št. okvirja za DESC** orbito	441. okvir 22. relativne orbite	
Število posnetkov za ASC* orbito	332	317
Število posnetkov za DESC** orbito	333	229
Časovna ločljivost	6 dni (do dec 2021), nato 12 dni	
Prostorska ločljivost	15 m	

*ASC orbita = dvigajoča orbita (ang. *ascending orbit*)

**DESC orbita = spuščajoča orbita (ang. *descending orbit*)

Podatki satelita Sentinel-1A so postali širši javnosti dostopni od 8. oktobra 2014 naprej, dostopnost podatkov satelita Sentinel-1B pa od konca leta 2016 do 23. decembra 2021. Sentinel-1B je 23. decembra 2021 prenehal pošiljati podatke zaradi napake v električni napeljavi, 3. avgusta 2022 pa je bil razglašen konec misije tega satelita, saj satelita ni bilo mogoče popraviti [33]. Povratna doba satelitov je bila za obdobje med 1. januarjem 2017 in 20. decembrom 2021 6 dni, za obdobje od decembra 2021 do marca 2023 pa 12 dni.

Ves postopek procesiranja se izvaja v radarskem koordinatnem sistemu z geometrijo poševnega razpona (ang. *slant range geometry*). Razpoložljivost večjega števila primernih interferogramov je osnova za zanesljivost pridobljenih produktov. Za analizo je potrebnih najmanj 15 do 20 posnetkov, ki pokrivajo časovni interval enega ali več let. Večja kot je kopica uporabljenih posnetkov, večja je natančnost končnih produktov.

3.2.1 Small BAseline Subset InSAR (SBAS InSAR)

Metoda *Small BAseline Subset InSAR* (SBAS InSAR) temelji na uporabi podskupin interferogramov z manjšimi bazami. Interferometrična baza je razdalja med dvema satelitoma (ali orbitama) v ravnini, ki je pravokotna na orbito [30]. Pri metodi SBAS InSAR se uporabijo prav vsi posnetki, ki so na voljo za obdelavo, se jih med seboj »poveže« in iz parov posnetkov izdelava različne interferograme z različnimi bazami. Slednje se nato prav tako poveže med seboj, pri čemer se za zmanjšanje vpliva prostorske dekorelacije in napak zaradi napak v DMR-ju izločijo interferogrami s prevelikimi bazami ter nato izračuna razlike v fazi med interferogrami. V postopku je prav tako tudi proces zmanjševanja efekta dekorelacije zaradi šuma, s čimer se prepreči negativni vpliv na izračune premikov v smeri LOS.

Metoda SBAS InSAR je zelo močno orodje za določanje površinskih sprememb tako v velikem merilu kot za manjša lokalna območja [34]. Glavna značilnost te metode je sposobnost zmanjšanja faznega šuma in napak številnih interferogramov ter sposobnost, da izlušči faze, ki so dejansko rezultat premikov [35]. Glavne omejitve SBAS InSAR metode še naprej ostajajo meritve površinskih sprememb na območjih z nizko koherenco (vegetacija), ocena atmosferskih napak ter procesiranje velikih količin podatkov – *big data* [36].

SBAS InSAR metoda omogoča natančnost 1–2 mm/leto za območja z dobro časovno in prostorsko koherenco, kar je primerljivo z GNSS meritvami. Natančnost rezultatov pada z manjšanjem časovne in prostorske koherence območja, kot npr. na poraščenih ali zasneženih območjih.

4 REZULTATI IN DISKUSIJA

4.1 Plaz Slano blato

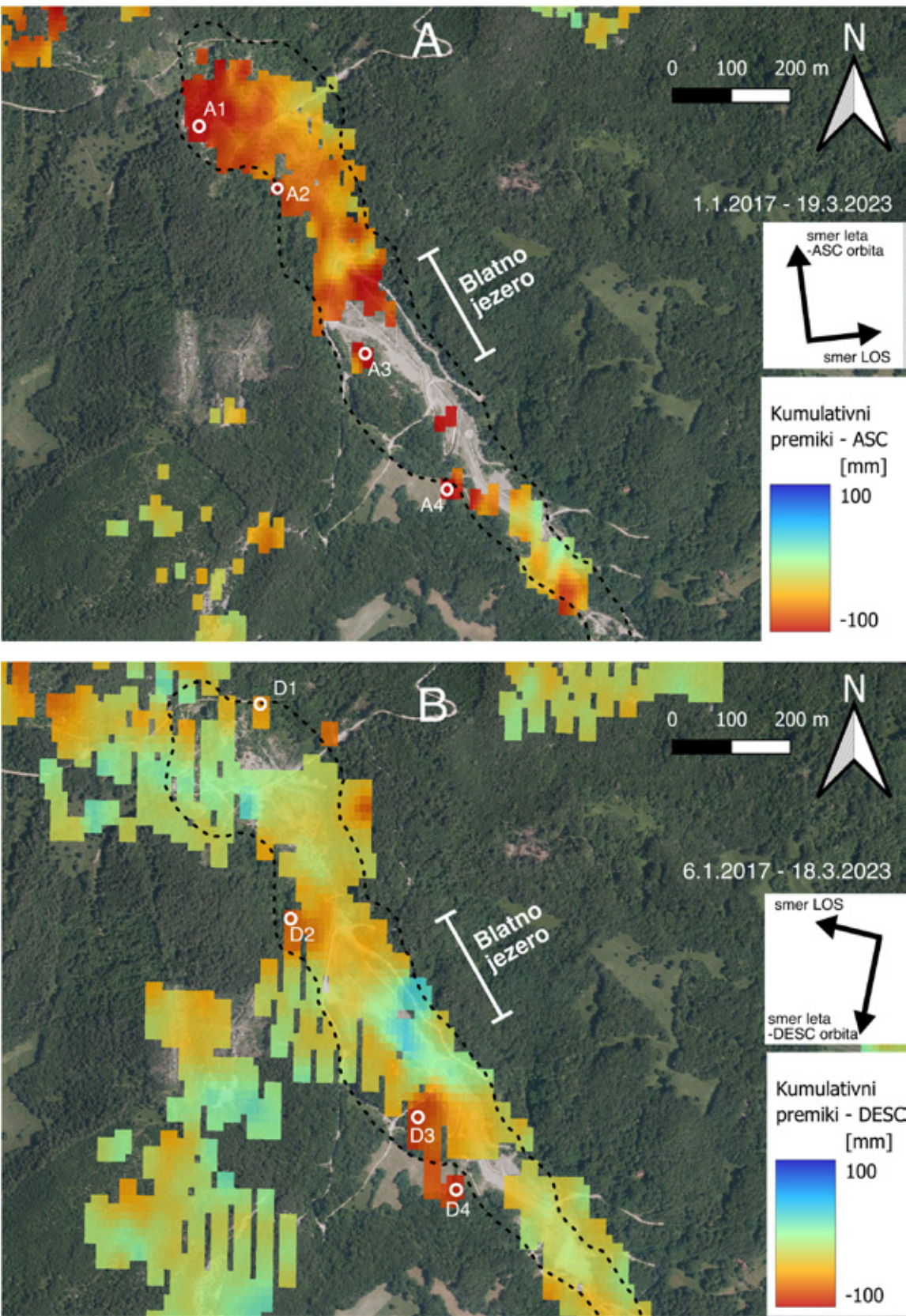
Kumulativne površinske spremembe oz. premiki plaz u so bili pridobljeni za obdobje med 6. januarjem 2017 in 18. marcem 2023 za DESC (spuščajočo) orbito ter med 1. januarjem 2017 in 19. marcem 2023 za ASC (dvigajočo) orbito. Kumulativni premiki predstavljajo skupne premike za celotno obdobje, pri čemer je referenčni posnetek prvi posnetek v nizu. Pri prvem posnetku so premiki ničelni, končni rezultat predstavlja razliko v topografiji med prvim in zadnjim posnetkom v smeri LOS, premiki pa so podani v mm.

Na sliki 3 so prikazani rezultati za obe orbiti. Razlika med orbitama ASC in DESC je smer potovanja in smer snemanja satelita, ki različno pokriva območje plaz u. Modra barva pomeni premik proti satelitu (kompresija), rdeča pa stran od satelita (ekstenzija) v smeri snemanja satelita (ang. *Line of Sight*, LOS), območja brez premikov (do 0 ± 1 mm) so rumeno-zelene barve. Satelit iz ASC orbite (Slika 3) snema pobočje prečno na plaz (prikazano na grafiki). Območje »Blatno jezero« na posnetkih ni vidno zaradi značilnosti topografije, zato so rezultati na tem območju pomanjkljivi. Satelit iz DESC orbite ima praktično pravokotno smer LOS na pobočje plaz u, zato rezultati bolje pokrivajo plaz. Odmikanje od satelita v smeri LOS pomeni pogrezanje ali posedanje, medtem ko premik proti satelitu pomeni akumulacijo materiala.

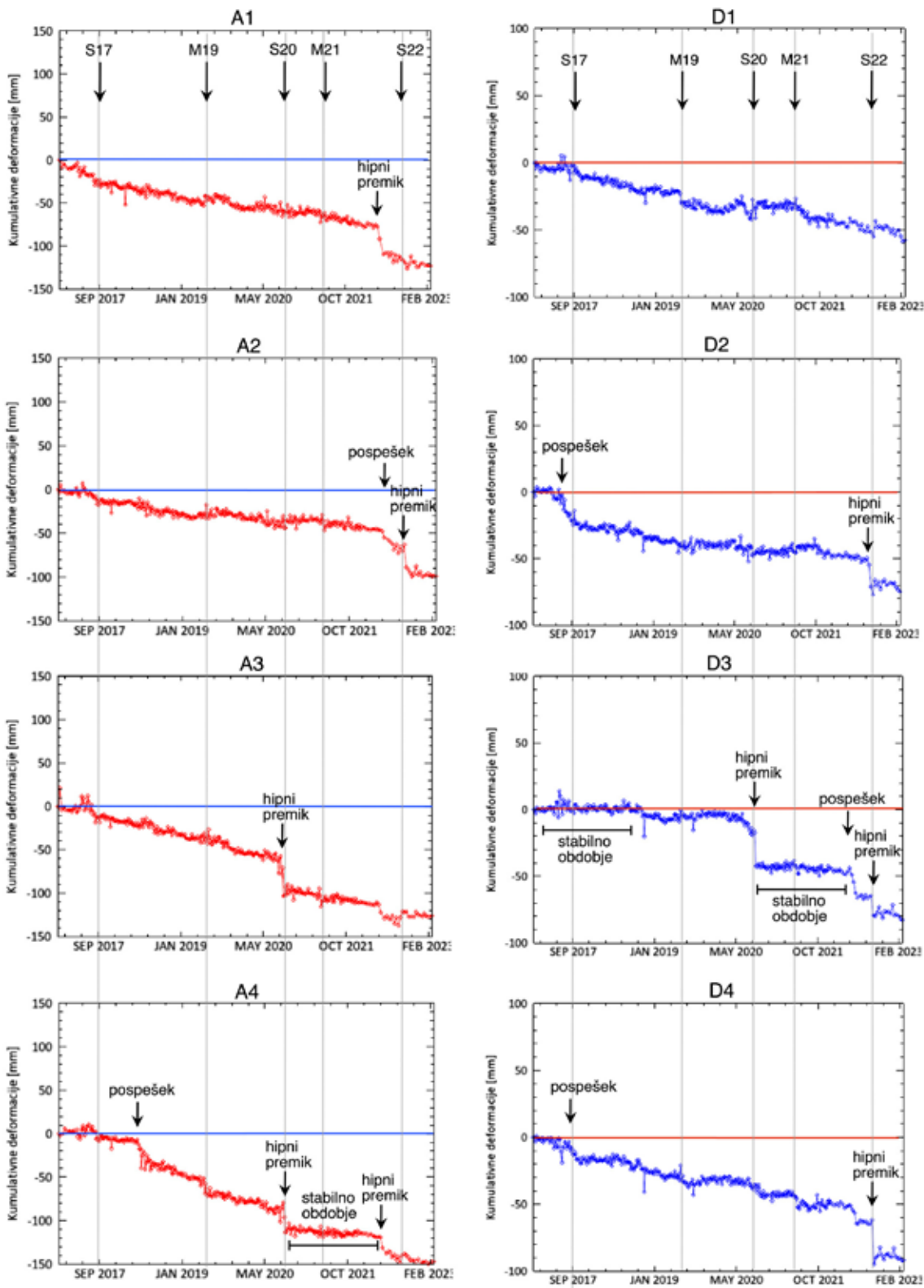
Primerjava rezultatov obeh orbit kaže na pogrezanje delov pobočja. Tudi ob terenskem ogledu plaz u se je pokazalo, da se na plaz u dogajajo spremembe v obliki manjših plaz u in akumulacije ter površinske erozije. Na ASC karti (slika 3a) je vidna ekstenzija vzdolž celotnega plaz u (z izjemo območja »Blatnega jezera«), pri čemer so žarišča v desnem zgornjem delu pri odlomnem robu ter vzdolž stranskih robov plaz u. Rezultatov izven območja plaz u ni, najverjetneje zaradi goste vegetacije in strmega pobočja. Na DESC karti (slika 3b) so žarišča premikov drugačna, vendar se še vedno pojavljajo na robovih plaz u, pojavljajo pa se tudi manjša lokalna območja akumulacije materiala. Na območju »Blatnega jezera«, kjer je vidna akumulacija (modra barva) iz DESC orbite, se je med letoma 2019 in 2022 izvajala sanacija pobočja. DESC orbita podaja tudi nekaj rezultatov izven območja plaz u, najverjetneje zaradi bolj ugodnega kota snemanja satelita, kar je omogočilo prodiranje skozi nižjo vegetacijo.

Površinski premiki plaz u Slano blato so bili validirani s terenskimi ogledi in »žarišča« sovpadajo s terenskim opazovanjem površinskih sprememb – proženje usadov ob robovih plaz u, površinska erozija ter območja izvedbe sanacijskih del.

Za podrobnejšo analizo premikov so bile izbrane različne interesne točke, za katere so bili izdelani grafi časovne razporeditve zaznanih premikov (slika 4). Na sliki 3 označene točke A predstavljajo časovne nize premikov za ASC orbito, točke D pa za DESC orbito. Točke so bile izbrane z namenom razlage različnih tipov krivulj premikov.



Slika 3: Rezultati analize SBAS InSAR za plaz Slano blato za (a) dvigajočo orbito (ASC) in (b) spuščajočo orbito (DESC) za obdobje med januarjem 2017 in marcem 2023 na DOF podlagi (Geodetska uprava Republike Slovenije, DOF 2024).



Slika 4: Grafi časovnih nizov premikov za določene točke, označene na sliki 3. Vertikalne črte predstavljajo večje padavinske dogodke, ki so se zgodili tekom septembra 2017 (S17), maja 2019 (M19), septembra 2020 (S20), maja 2021 (M21) in septembra 2022 (S22).

Na grafih časovnih nizov premikov je opaznih več tipov premikov in sicer: (i) enakomerne, (ii) pospešene in (iii) »stopničaste« (hipne). V veliko primerih je kombinacija različnih, kar nakazuje na zelo heterogeno dogajanje na plazu. Primer »stopničastih« oz. hipnih premikov je točka D3, kjer imamo izrazito mirovanje – stabilnost točke – med posameznimi dogodki hitrih in izrazitih premikov. Do neke mere se podobno dogaja tudi na točkah A4 in D4. Pri A4 je opazen tudi trend pospeška, ki se razvije iz stabilnega stanja, ki je trajalo od septembra 2017 do približno konca leta 2018. Sledilo je premikanje in pospešeno premikanje, nato pa ustalitev hitrosti in sčasoma skoraj ustavitev premikanja v maju 2020. Podobni pospeški so se pojavili tudi na A2, D2 in D4 točkah, pri čemer sta na D2 in D4 točkah ob enakem času. Na grafih A1, D2 in A3 je dobro izraženo enakomerno deformiranje, ki je bilo prekinjeno zaradi večjih hipnih premikov. Zlasti A3 kaže trend konstantne hitrosti tudi po tem, ko se je na lokaciji zgodil večji enkratni premik.

Hipne premike večinoma lahko razložimo na podlagi padavinskih dogodkov. Na grafih so z vertikalnimi sivimi črtami označeni nekateri izmed bolj deževnih mesecev v obdobju opazovanja. V septembru 2017 (S17) je padlo 418 mm padavin v obliki dolgotrajnih vsakodnevnih padavin. Maja 2019 (M19) je padlo 391 mm padavin prav tako v obliki dolgotrajnega deževja. Septembra 2020 (S20) je v dveh dneh padlo 290 mm padavin, v enem tednu 418 mm, v celem mesecu pa 466 mm. Maj 2021 (M21) je bil konstantno deževen z enim izrazitim padavinskim dnem, ko je padlo 100 mm v 24 urah, vse skupaj pa je bilo 522 mm padavin v celem mesecu. September 2022 (S22) je bil po izrazito sušnem obdobju precej deževen mesec, ko je v enem dnevu padlo 127 mm, v dveh 220 mm, v enem tednu 290 mm, vse skupaj pa 601 mm padavin. Podatki so bili pridobljeni za padavinsko postajo Otlica, ki se nahaja približno 4,5 km vzhodno od plazu.

Vsaka lokacija ima različen odziv na padavinske dogodke. Največ premikov so povzročile padavine v septembru 2020 ter septembru 2022. Pri določenih točkah (A1, A2, A3, D3, A4 in D4) je opazen hipni premik tudi v maju 2022, ko je bilo zelo malo padavin. Tega »padca« na grafu, tako kot nekaterih drugih zaznanih premikov, ki jih je analiza InSAR zaznala, ni mogoče razložiti s padavinskimi dogodki. Razlog za to je morda oddaljenost padavinske postaje od lokacije plazu. Lokalni padavinski dogodki so se lahko zgodili le na plazu in ne na lokaciji postaje, kar onemogoča natančne korelacije premikov s padavinskimi dogodki.

Preostala območja, ki se premikajo, nakazujejo na manjše premike, ki so v rangi do nekaj cm za celotno obdobje opazovanja, kar lahko predstavlja povprečno erozijo.

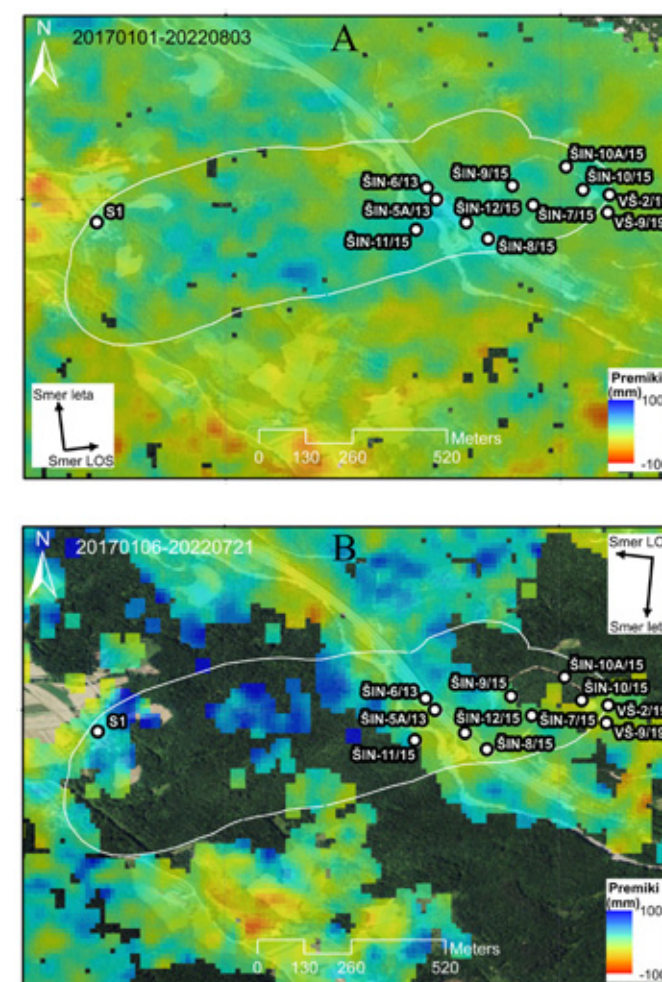
4.2 Plaz Šumljak

Kumulativni premiki površja plazu Šumljak, pridobljeni z analizo SBAS InSAR, so predstavljeni na sliki 5. Rezultati so predvsem za dvigajočo (DESC) orbito pomanjkljivi, najverjetneje zaradi izgube koherence zaradi vegetacije. Označene točke na plazu predstavljajo lokacije inklinometriških vrtin; slednje v tem članku niso obravnavane, so pa lokacije uporabljene kot interesne točke.

Rezultati iz ASC orbite (slika 5a) kažejo manj izrazite premike kot pri DESC orbiti (slika 5b). Zaznane so bile spremembe površja v rangi med -20 in 20 mm (rdeča barva – ekstenzija oz. premik stran od satelita, modra barva – kompresija oz. premik proti satelitu). Na levem boku v osrednjem delu plazu je opazno žarišče kompresije. Pri rezultatih iz DESC orbite (slika 5b) je opazna izrazita kompresija (temno modra barva) v osrednjem in spodnjem delu plazu, kar pomeni dvig ozemlja ali akumulacijo materiala. Posedanje ozemlja bi bilo obarvano z rdečo barvo, kar pa na območju plazu ni bilo zaznано. Posedanje je bilo pričakovano v zgornjem delu plazu, kjer se nahaja območje praznjenja. V zgornjem delu plazu (nad hitro cesto) je ponekod opazna kompresija, vendar so rezultati pomanjkljivi. Splošno gledano za DESC orbito so značilna območja kompresije oz. premika proti satelitu

na več delih plazu, manjša območja pa kažejo premike med -20 in 20 mm. Vzdlž hitre ceste je bila zaznana rahla kompresija pri ASC orbiti ter rahla ekstenzija pri DESC orbiti.

Časovni razvoj premikov vzdolž viadukta kaže kompresijo iz vidika ASC orbite s kumulativnimi premiki 1–2 cm v 5 letih (slika 5a). Na drugi strani, premiki iz vidika DESC orbite kažejo ekstenzijo (negativno vrednost) za območje točk ŠIN-12 in ŠIN-8 ter zelo malo ali nič premikov za točki ŠIN-5 in ŠIN-6 (slika 5b).

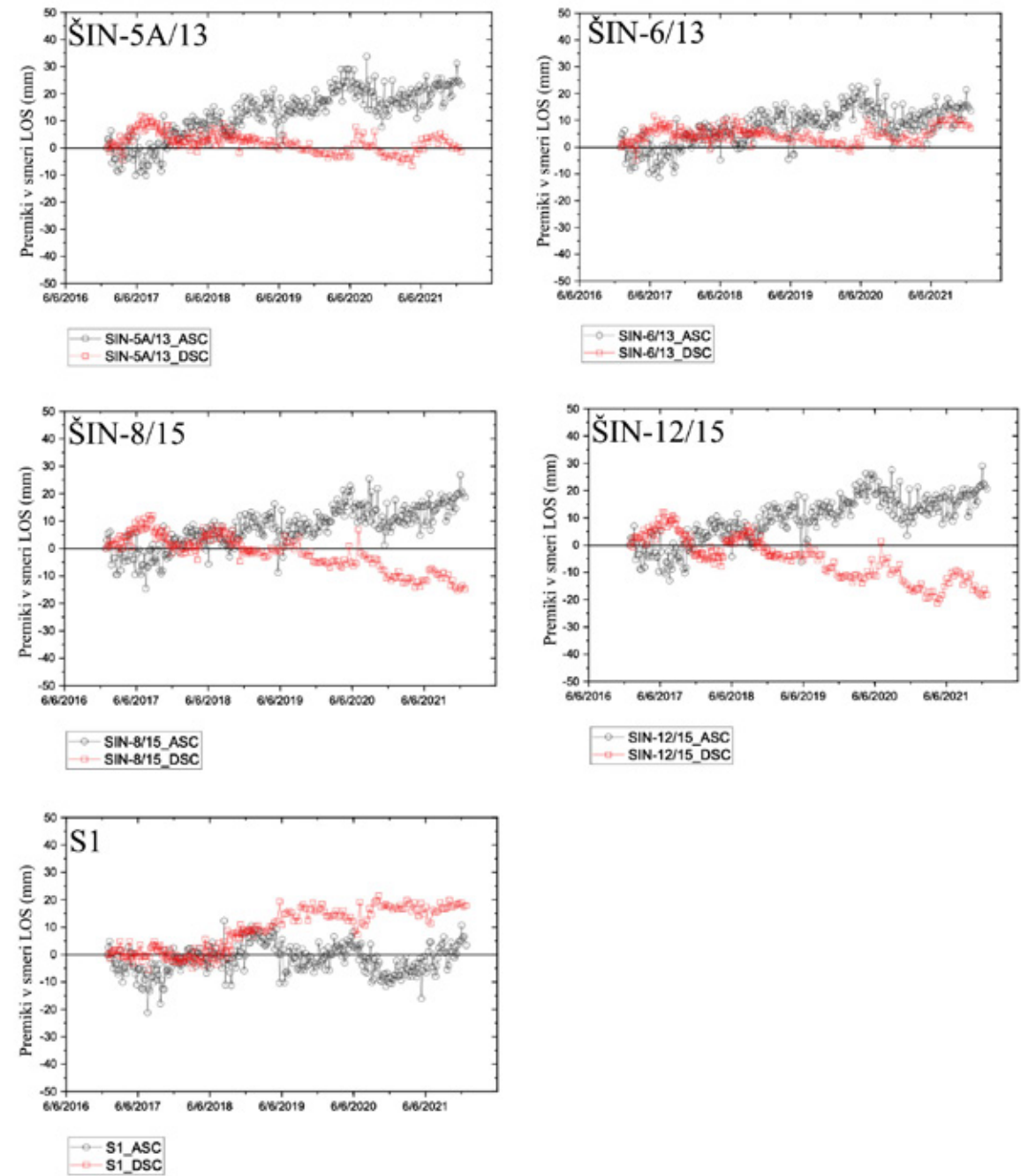


Slika 5: Kumulativni premiki območja plazu Šumljak za (a) dvigajočo (ASC) in (b) spuščajočo (DESC) orbito za obdobje med januarjem 2017 in avgustom 2022. Točke označujejo lokacije inklinometriških vrtin, ki v tem članku niso obravnavane.

Časovni razvoj premikov za točke vzdolž viadukta in za lokacijo na peti plazu, kjer se nahaja majhen zaselek z nekaj hišami, je predstavljen na sliki 6. Na grafih je za DESC orbito (rdeča krivulja) viden trend sinusoidnega nihanja z vrhom v poletnih mesecih. To je najverjetneje posledica temperaturne ekspanzije in kontrakcije viadukta z vrhom v poletnih mesecih in minimumom v zimskih mesecih, kar InSAR lahko zaznava [37]. Kljub temu trend za območje točk ŠIN-8 in ŠIN-12 (leva stran plazu) kaže posedanje viadukta za okoli 2 cm/5 let, medtem ko sta točki ŠIN-5A in ŠIN-6 (desna stran plazu) bolj stabilni ter z manjšim nihanjem zaradi temperaturnih sprememb viadukta. Točka S1 kaže trend dviga iz smeri DESC orbite, medtem ko iz ASC orbite kaže predvsem sinusoidno nihanje in bolj

stabilno krivuljo s sezonskim nihanjem med -10 in 10 mm. ASC orbita (črna) za ostale točke podaja trend dviga z rahlim sinusoidnim gibanjem zaradi temperaturne ekspanzije materiala viadukta. Splošno, kumulativni premiki kažejo heterogenost površinskih sprememb plaz.

Rezultati za viadukt bodo dodatno preverjeni z rezultati geodetskih izmer deformacij viadukta.



Slika 6: Grafi časovnih nizov premikov, pridobljenih iz SBAS InSAR analize, na lokacijah inklinometrov.

5 ZAKLJUČKI

- **Tehnologija InSAR** je ustrezna za opazovanje sprememb površja na pobočjih, zlasti kjer območje ni pokrito z gosto vegetacijo in omogoča dober odboj satelitovega signala od površja.
- **Analiza po metodi SBAS InSAR** je podala ustrezne rezultate za ugotavljanje površinskih sprememb na pilotnih območjih. Rezultati pokrivajo skoraj celotno površino plazov, vključno z objekti, ki se nahajajo na njih.
- Podatki iz spuščajoče (DESC) orbite so podali dober vpogled v dogajanje na plazu Slano blato. Dvigajoča (ASC) orbita je manj ustrezna za analizo zaradi usmerjenosti pobočja in nastanka topografske sence, kar vodi v pomanjkljive rezultate. Na nekaterih delih plaz, ki so pokriti z gosto vegetacijo, so prav tako pomanjkljivi rezultati zaradi kanala, v katerem snema satelit (C kanal z valovno dolžino okoli 5,6 cm, ki prodre do 6 m v vegetacijo).
- Podatki iz dvigajoče (ASC) orbite so podali zadovoljiv vpogled v dogajanje na plazu Šumljak, medtem ko so rezultati iz spuščajoče (DESC) orbite pomanjkljivi zaradi kota snemanja in vegetacije (zmanjšana koherenca). Najboljši odboj je vzdolž hitre ceste, kjer je odboj signala konstanten in omogoča kakovostno analizo časovnega razvoja premikov viadukta.
- **Analiza časovnih vrst** je za plaz Slano blato podala dober vpogled v vpliv padavinskega režima na površinske spremembe plaz, medtem ko je za plaz Šumljak podala vpogled v časovni razvoj premikov viadukta, za katerega je značilno nihanje zaradi sezonskih temperatur (temperaturna ekspanzija in kontrakcija objekta) ter posedanje (najverjetneje zaradi počasnega plazenja).
- Rezultati za viadukt Šumljak bodo dodatno preverjeni z rezultati geodetskih izmer.
- **Daljši kot je obdobje opazovanja, boljši so rezultati in možnosti za opazovanje trendov premikov.**

ZAHVALA

Raziskave je podprla Javna agencija za znanstvenoraziskovalno in inovacijsko dejavnost Republike Slovenije (programski skupini P1-0195 »Geookolje in geomateriali« ter P2-0406 »Opazovanje Zemlje in geoinformatika«). Zahvaljujemo se tudi sodelavcema prof. dr. Norikazu Shimizu in dr. Putu Edi Yastika ter DARS d.d. in Terras s.p. za sodelovanje.

LITERATURA IN REFERENCE

- [1] Frattini, P., Crosta, G.B., et al., Activity and Kinematic Behaviour of Deep-Seated Landslides from PS-InSAR Displacement Rate Measurements. *Landslides* 15 (6): 1053–70, 2018. doi:10.1007/s10346-017-0940-6
- [2] Lu, P., Bai, S. et al., Landslides detection through optimized hotspot analysis on persistent scatterers and distributed scatterers. *ISPRS Journal of Photogrammetry and Remote Sensing* 156: 147-159, 2019. doi:10.1016/j.isprsjprs.2019.08.004
- [3] Liu, Y., Lu, Z. et al., Characterization of the Kinematics of Three Bears Landslide in Northern California Using L- band InSAR Observations. *Remote Sensing* 11, 23: 2726, 2019. doi:10.3390/rs11232726
- [4] Solari, L., Del Soldato, M., et al., A Sentinel-1 based hot-spot analysis: landslide mapping in north-western Italy. *International Journal of Remote Sensing* 40, 20: 7898-7921, 2019. doi:10.1080/01431161.2019.1607612
- [5] Komac, M., Holley, R. et al., Coupling of GPS/GNSS and radar interferometric data for a 3D surface displacement monitoring of landslides. *Landslides* 12: 241-257, 2015. doi:10.1007/s10346-014-0482-0

- [6] Mirzaee, S., Motagh, M. et al., Landslide monitoring using InSAR time-series and GPS observations, case study: Shabkola landslide in Northern Iran. *The International Archives of Photogrammetry, Remote Sensing and Spatial Information Sciences XLII-1/W1*: 487–492, 2017. doi:10.5194/isprs-archives-XLII-1-W1-487-201
- [7] Carlà, T., Tofani, V. et al., Combination of GNSS, satellite InSAR, and GBInSAR remote sensing monitoring to improve the understanding of a large landslide in high alpine environment. *Geomorphology* 355: 62–75, 2019. doi:10.1016/j.geomorph.2019.03.014
- [8] Samsonov, S., Dille, A. et al., Satellite Interferometry for Mapping Surface Deformation Time Series in One, Two and Three Dimensions: A New Method Illustrated on a Slow-Moving Landslide. *Engineering Geology* 266: 105471, 2020. doi:10.1016/j.enggeo.2019.105471
- [9] Vick, L.M., Böhme, M. et al., Structurally controlled rock slope deformation in northern Norway. *Landslides* 17: 1745–1776, 2020. doi:10.1007/s10346-020-01421-7
- [10] Razi, P., Sumantyo, J.T.S. et al., 3D Land Mapping and Land Deformation Monitoring Using Persistent Scatterer Interferometry (PSI) ALOS PALSAR: Validated by Geodetic GPS and UAV. *IEEE Access* 6: 12395–12404, 2018. doi:10.1109/ACCESS.2018.2804899
- [11] Eker, R., Aydın, A., Long-term retrospective investigation of a large, deep-seated, and slow-moving landslide using InSAR time series, historical aerial photographs, and UAV data: The case of Devrek landslide (NW Turkey). *Catena* 196: 104895, 2021. doi:10.1016/j.catena.2020.104895
- [12] Debevec Jordanova, G., Spremljanje dinamike pobočnih masnih premikov v Vipavski dolini z daljinskim zaznavanjem in in-situ meritvami. Doktorska disertacija. Ljubljana, Univerza v Ljubljani, Naravoslovnotehniška fakulteta, 2025.
- [13] Oštir, K., Veljanovski, T., Application of satellite remote sensing in natural hazard management: The Mount Mangart landslide case study. *International Journal of Remote Sensing* 24, 20: 3983–4002, 2003. doi:10.1080/0143116031000103826
- [14] Komac, M., & Bavec, M., Application of PSInSAR for observing the vertical component of the recent surface displacements in Julian Alps. *Geologija* 50, 1: 97–110, 2007. doi:10.5474/geologija.2007.008
- [15] Oštir, K., Komac, M., PSInSAR and DInSAR methodology comparison and their applicability in the field of surface deformations—A case of NW Slovenia. *Geologija* 50, 1: 77–96, 2007. doi:10.5474/geologija.2007.007
- [16] Čarman, M., Jemec Auflič, M. et al., Landslides at a uranium mill tailing deposit site Boršt (Slovenia) detected by radar interferometry. *Landslides* 11: 527–536, 2014. doi:10.1007/s10346-013-0454-9
- [17] Jemec Auflič, M., Uporaba metode permanentnih sipalcev v geologiji kot podpora pri proučevanju plazljivih območij. Doktorska disertacija. Ljubljana, Fakulteta za gradbeništvo in geodezijo (samozaložba M. Jemec Auflič), 2012.
- [18] Jemec Auflič, M., Jež, J. et al., Landslide Events in Slovenia 2023: Causes and Consequences. V: Marjanović, M., Đurić, U. (ur.) *Proceedings of the 6th Regional Symposium on Landslides in the Adriatic-Balkan Region, ReSyLAB2024, Belgrade, Serbia 15–18th May 2024*, 6: p. 199–202, 2024.
- [19] Placer, L., Geološka zgradba jugozahodne Slovenije = Geologic structure of southwestern Slovenia. *Geologija* 24, 1: 27–60, 1981.
- [20] Placer, L., Jež, J. et al., Strukturni pogled na plaz Slano blato = Structural aspect on the Slano blato landslide (Slovenia). *Geologija* 51, 2: 229–234, 2008. doi:10.5474/geologija.2008.023
- [21] Buser, S., Grad, K. et al., Osnovna geološka karta SFRJ 1 : 100.000, list Postojna. Zvezni geološki zavod, Beograd, 1967.
- [22] Buser, S. Osnovna geološka karta Jugoslavije 1 : 100.000, list Gorica. Zvezni geološki zavod, Beograd, 1968.
- [23] Ribičič, M., Kočevar, M., Končna sanacija plazu Slano Blato nad Lokavcem pri Ajdovščini = The final remediation of the landslide Slano blato above settlement Lokavec at Ajdovščina. *Geologija* 45, 2: 525–530, 2002. doi:10.5474/geologija.2002.059
- [24] Logar, J., Fifer Bizjak, K., et al., History and present state of the Slano Blato landslide. *Natural Hazards and Earth System Sciences* 5: 447–457, 2005.
- [25] Habič, P., Kraški svet nad Idrijco in Vipavo. Prispevek k poznavanju razvoja kraškega relief = The Karstic region between the Idrijca and Vipava rivers: a contribution to the study of development of the Karst relief. V: Dela. Ljubljana : Slovenska akademija znanosti in umetnosti, Razred za prirodoslovne in medicinske vede, 1968.
- [26] Popit, T., Rožič, B. et al., A LIDAR, GIS and basic spatial statistic application for the study of ravine and palaeo-ravine evolution in the upper Vipava valley, SW Slovenia. *Geomorphology* 204: 638–645, 2014. doi:10.1016/j.geomorph.2013.09.010
- [27] Popit, T., Mehanizmi transporta in sedimentacijski procesi kvartarnih pobočnih sedimentov na območju Rebrnic. Doktorska disertacija. Ljubljana, Fakulteta za gradbeništvo in geodezijo (samozaložba T. Popit), 2016.
- [28] Popović, Z., Letno poročilo z interpretacijo rezultatov geotehničnih meritev, 8. cikel viadukta Šumljak, HC Razdrto-Vipava, leto 2020. Terras s.p., 2021.
- [29] Oštir, K., Daljinsko zaznavanje. Ljubljana, Založba ZRC, ZRC SAZU, 2006.
- [30] Ferretti, A., Monti-Guarnieri, A., et al., *InSAR Principles: Guidelines for SAR Interferometry processing and interpretation*. ESTEC Noordwijk: ESA Publications, 2007.
- [31] <https://www.ga.gov.au/scientific-topics/positioning-navigation/geodesy/geodetic-techniques/interferometric-synthetic-aperture-radar> (Pridobljeno 25. 02. 2025).
- [32] Massonnet, D., Briole, P., et al., Deflation of Mount Etna monitored by spaceborne radar interferometry. *Nature* 375: 567–70, 1995.
- [33] https://www.esa.int/Applications/Observing_the_Earth/Copernicus/Sentinel-1/Mission_ends_for_Copernicus_Sentinel-1B_satellite (Pridobljeno 2. 6. 2023).
- [34] Lanari, R., Casu, F. et al., An overview of the Small Baseline Subset Algorithm: A DInSAR Technique for Surface Deformation Analysis. V: Wolf, D., Fernández, J. (ur.) *Deformation and Gravity Change: Indicators of Isostasy, Tectonics, Volcanism, and Climate Change*. Pageoph Topical Volumes. Birkhäuser Basel: p. 637–661, 2007. doi:10.1007/978-3-7643-8417-3_2
- [35] Yastika, P. E., Shimizu, N. et al., Monitoring of long-term land subsidence from 2003 to 2017 in coastal area of Semarang, Indonesia by SBAS InSAR analyses using Envisat-ASAR, ALOS-PALSAR, and Sentinel-1A SAR data. *Advances in Space Research* 63, 5: 1719–1736, 2019. doi:10.1016/j.asr.2018.11.008
- [36] Li, S., Wenbin, X. et al., Review of the SBAS InSAR Time-series algorithms, applications, and challenges. *Geodesy and Geodynamics* 13, 2: 114–126, 2022. doi:10.1016/j.geog.2021.09.007
- [37] Selvakumaran, S., Sadeghi, Z. et al., Comparison of in situ and interferometric synthetic aperture radar monitoring to assess bridge thermal expansion. *Proceedings of the Institution of Civil Engineers – Smart Infrastructure and Construction* 175(2): 73–91, 2022. doi:10.1680/jsmic.21.00008



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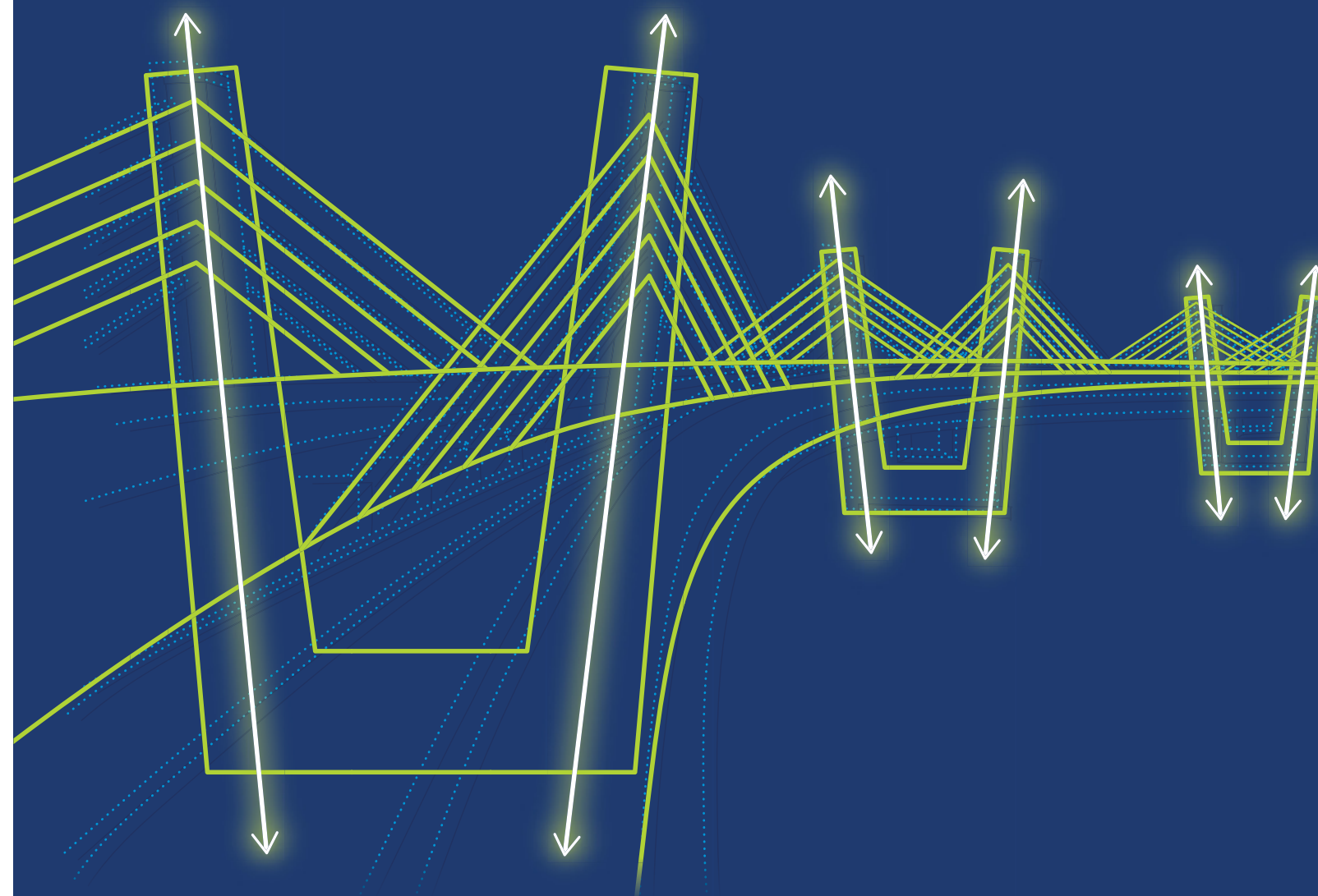


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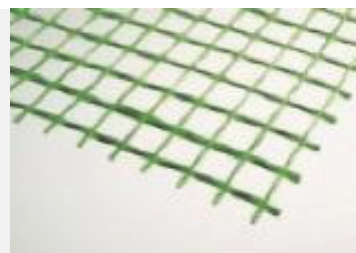
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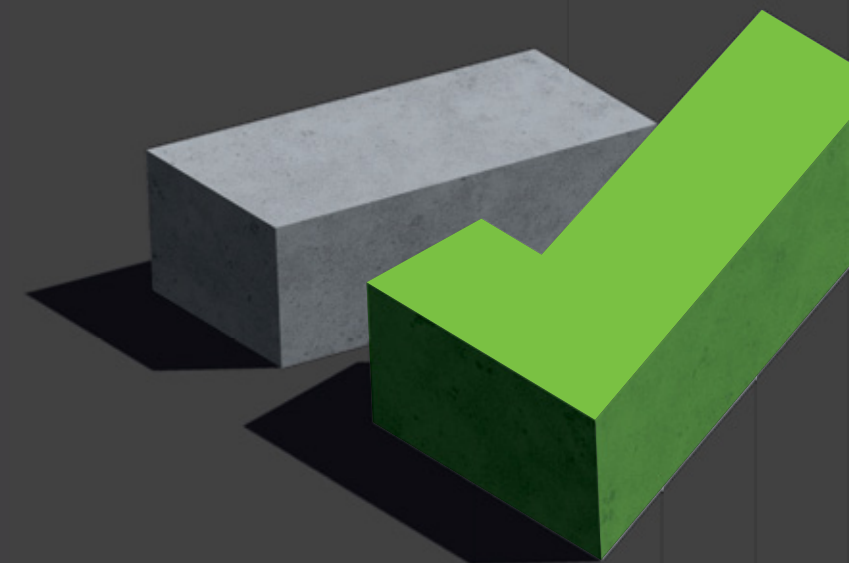
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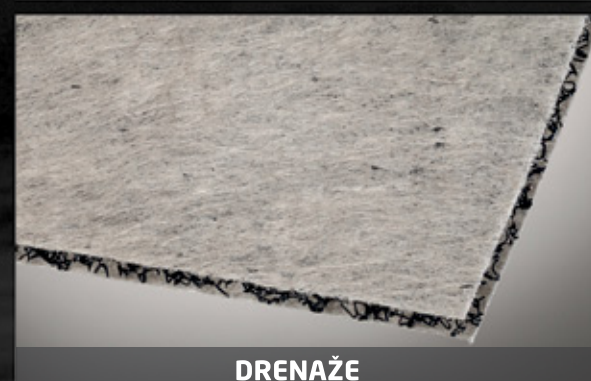
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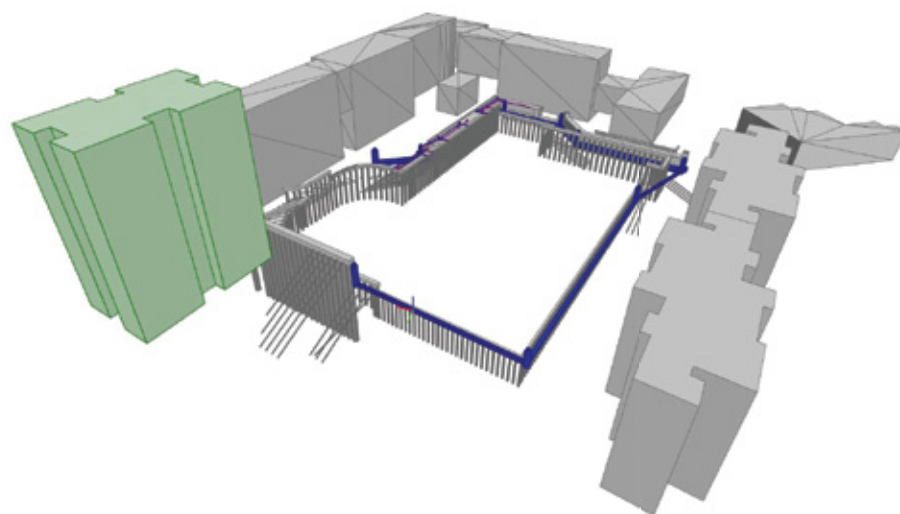
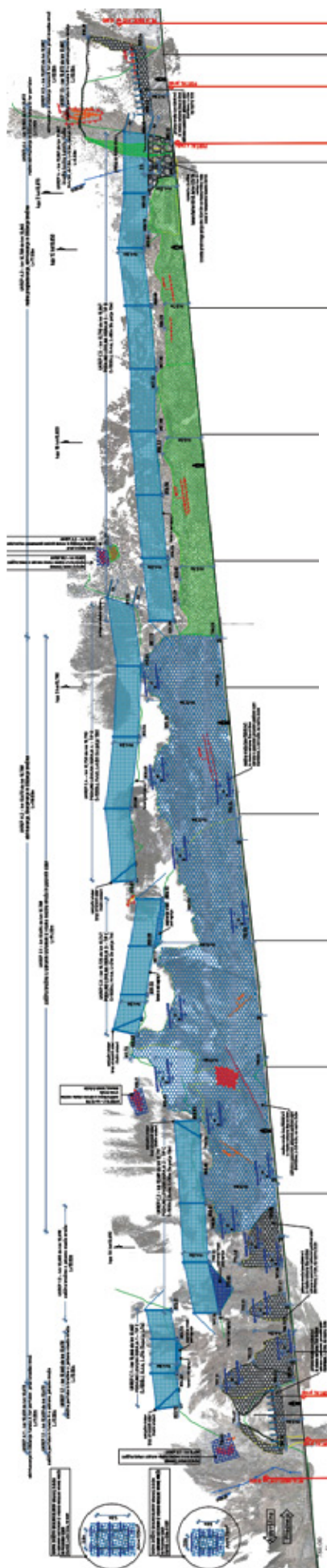


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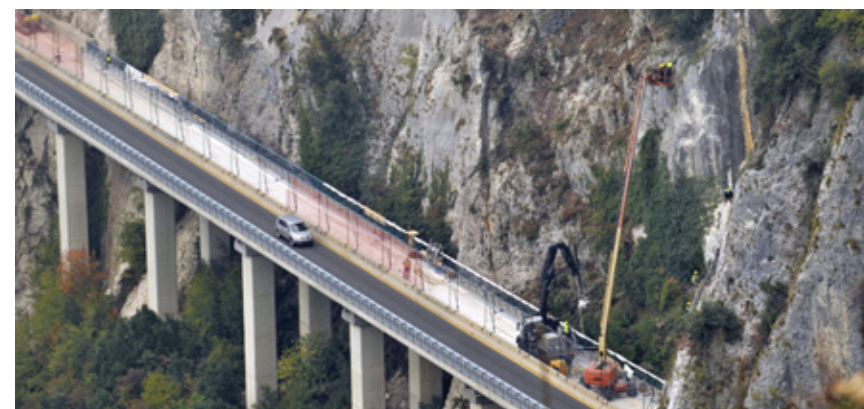
geomehanske raziskave
stabilnostne analize
varovanje gradbenih jam
sanacije plazov in usadov
zaščita pred padajočim kamenjem
zahtevno temeljenje objektov
kontrolirano zniževanje talne vode v gradbenih jamah

 **corus**
inženirji

www.c-i.si | info@c-i.si | +386(0)5 3002020

Skrb za okolje po elementarnih nesrečah.

EHO
PROJEKT



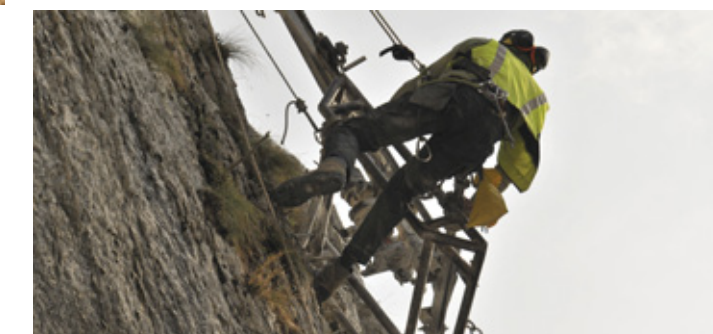
*Stabilizacije brežin
Skalni podori
Zemeljski plazovi
Snežni plazovi
Protipoplavna zaščita*

Izzive s katerimi se spopadamo predstavljajo skalni podori, poplave ter zemeljski in snežni plazovi.

Pri podjetju EHO Projekt d.o.o. skrbno načrtujemo, razvijamo in izvajamo učinkovito zaščito pred takimi pogostimi katastrofami. Prav tako raziskujemo okolje, da bi se izognili nesrečam, preden se zgodijo.

Imamo številne uspešne reference privatnih investorjev ter državnih inštitucij. Ponosni smo na kar nekaj uspešnih nujnih intervencij v katerih smo dokazali možnost hitrih in uspešnih odločitev v inženirskem in izvedbenem smislu.

Imamo širok nabor izkušenj na področju načrtovanja in izvedbe podajno lovilnih ograj kakor tudi prekrivnih mrež nizkih in visokih nateznih trdnosti. Izvajali in načrtovali smo sanacije zemeljskih plazov ter zaščite za varovanje pred snežnimi plazovi. Hkrati pa smo intenzivno vključeni tudi v načrtovanje in izvedbo projektov protipoplavne zaščite ter vodnogospodarskih ureditev.



EHO PROJEKT d.o.o.
Dunajska cesta 51
1000 Ljubljana, Slovenia
tel.: +386 (0)64 296 750
e-mail: info@ehoprojekt.si

www.ehoprojekt.si



FENIKS + d.o.o.
Koroška c. 90
4290 Tržič
Tel.: 041 300 501

www.feniks.biz



Premikamo meje, ustvarjamo poti

projektiranje - svetovanje - inženiring - nadzor



projektiranje - svetovanje - inženiring - nadzor

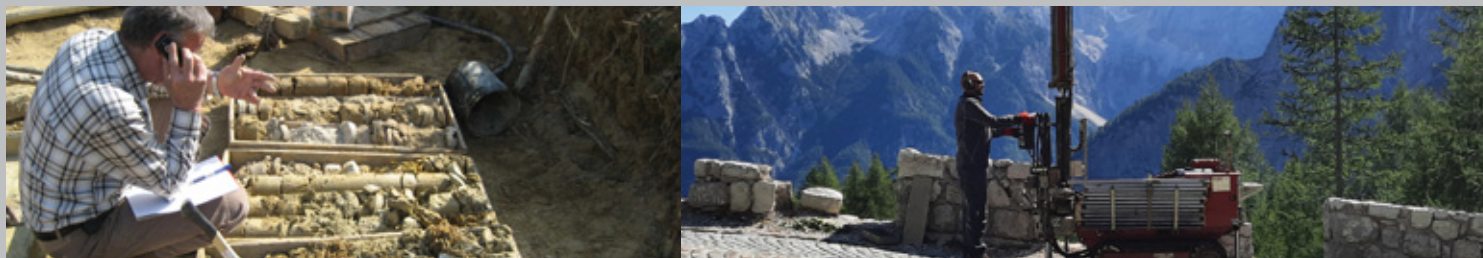
- predori in veliki podzemni prostori
- geologija in geotehnika
- gradbene jame
- sanacija plazov
- deponije
- visoke gradnje
- luške konstrukcije
- zastopstvo za programsko opremo GEO5 (FINE Ltd.)





CENTER ZA GEOTEHNIKO IN GEOLOGIJO

Geomehanika; inženirska geologija; projektiranje inženirskih objektov; raziskave in sanacije plazov; geološko-geotehnične analize in elaborati, kontrola kakovosti in geotehnični monitoring; terenske meritve karakteristik tal, geofizikalne meritve - georadar, geoelektrika.



CENTER ZA PROMETNICE IN INFRASTRUKTURO

Zunanja (ZKK) in notranja (NKK) kontrola izvedbe del pri gradnji prometnic in javne infrastrukture; izdelava izvedbenih načrtov za obnove vozišč, dimenzioniranje voziščnih konstrukcij; preiskave zemljin, kamenih agregatov, asfaltov, betonov in talnih označb.



CENTER ZA MATERIALE IN KONSTRUKCIJE

Ugotavljanje konstrukcijskih in materialnih lastnosti objektov; izvajanje statičnih in seizmičnih analiz objektov s predlogi ojačitve; računske ocene dejanske potresne odpornosti objektov; reševanje zahtevnih konstrukcijskih detajlov; recenzije in revizije projektne dokumentacije; pregledi in opazovanje objektov.



CENTER ZA BIVALNO OKOLJE, GRADBENO FIZIKO IN ENERGIJO

Sodelovanje pri pripravi strateških dokumentov in predpisov s področja gradbene fizike, učinkovite rabe energije v stavbah; razvoj na področju trajnostne gradnje, predinvesticijsko svetovanje za skoraj nič-energetske stavbe in celovito energetske prenove; strokovna mnenja in meritve mikroklima – temperatura, vlaga, CO₂ ter zrakotesnosti in termografija; energetske pregledi stavb in energetske izkaznice stavb.



Terenske in laboratorijske preiskave gradbenih materialov, proizvodov in konstrukcij

Detajlni pregledi materialno-tehničnega stanja inženirskih objektov in vozišč

Monitoringi objektov v vplivnem območju delovišč

Raziskovalno delo ter razvoj novih preiskav gradbenih materialov

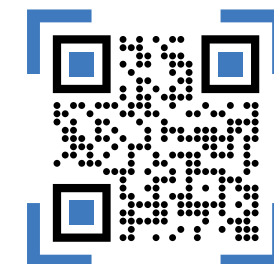
Certificiranje

Akreditacija s področja preskušanja in certificiranja

igmat

d.o.o.

inštitut za gradbene materiale



projektiranje
podpornih/opornih
konstrukcij
projekti sanacij plazov

dimenzioniranje voziščnih konstrukcij
hidrološko – hidravlične analize
analize tveganja onesnaženja podzemne vode
elaborati podnebnih sprememb

načrti temeljenja
načrti zaščite gradbenih jam
načrti zaščite in stabilizacije brežin
geomehanski nadzori

LAM BIRO

inklinometerske meritve
PET test - test zveznosti AB pilotov
meritve nosilnosti tamponskega nasutja
geološko – geomehanska poročila

GRADBENO – GEOTEHNIČNE STORITVE
težki dinamični penetrometer (DPSH-B)
krožna plošča z lahko padajočo utežjo
dinamični penetrometer (DPM)
geotehnične vrtine
dinamično jedrovanje

| LAM BIRO d.o.o. | Gradbeno projektiranje in nadzor | Šmartno ob Paki 134, 3327 Šmartno ob Paki |
| Pokličite nas: 031-776-280 | Pišite nam: info@lambiro.si | Sledite nam: www.lambiro.si |  profil |

Učinkovita prometna infrastruktura

Z znanjem, izkušnjami ter po najvišjih standardih kakovosti izboljšujemo in razvijamo učinkovito in za uporabnika varno prometno infrastrukturo.

Spoznajte naša ključna znanja in kompetence na
www.riko.si.



Specializirano podjetje za temeljenje objektov

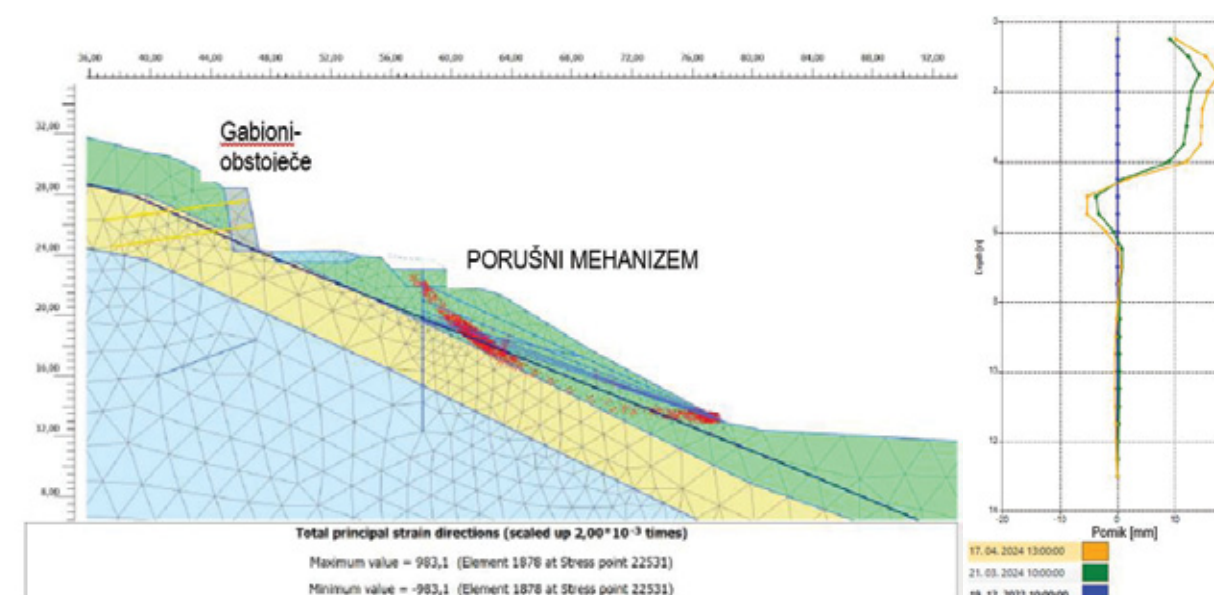


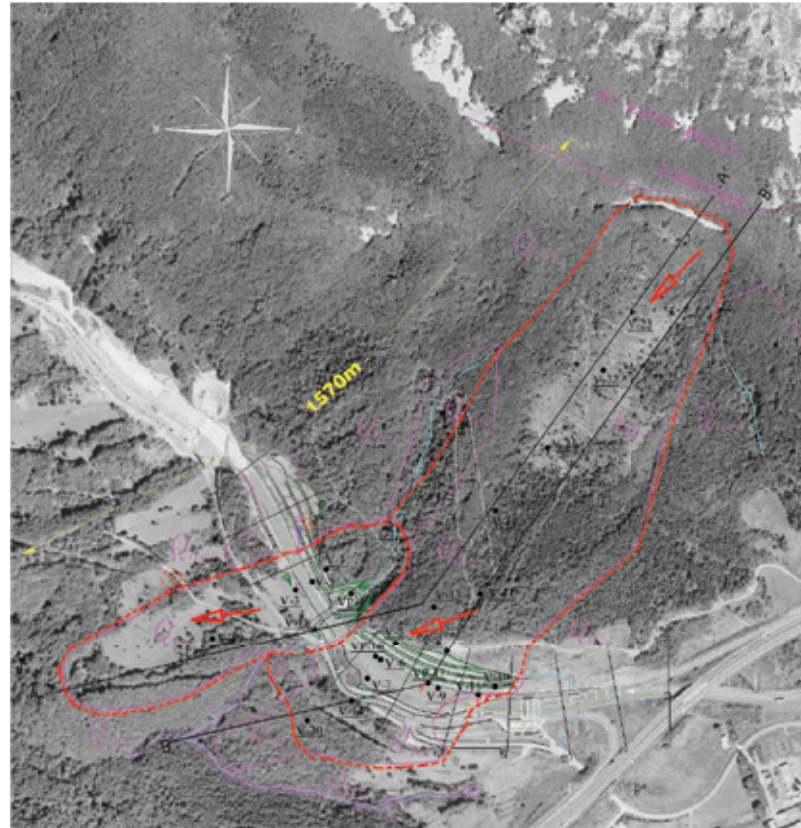
- Meritve nosilnosti pilotov
(dinamični in statični obremenilni test)
- Meritve zveznosti pilotov
(PIT, Cross-hole, TIP)
- Geotehnične preiskave in poročila
(CPT, DPSH, SDMT)
- Projektiranje, nadzor in svetovanje



- 35+ let izkušenj
- 3000+ uspešnih projektov
- 25+ držav
- Mednarodni certifikat za dinamične meritve "PDCA Expert"

Sanacija plazu na državni cesti R3-656/3650 (Dragarji) Zgornji Čačič – Osilnica s sidrano pilotno steno
- geološko geomehanske raziskave, projektiranje in nadzor -





Raziskave in monitoring globokega plazu »Razdrto«



Globoki plaz »Čemšenik«

25 let izkušenj, danes modrost v inženirski geologiji in geotehniki



Sanacija skalne brežine »Grad Borl«



**Zaščita državne ceste Sp. Idrija - Godovič (Zala)
pred podori**



24.
ŠUKLJETOV DAN