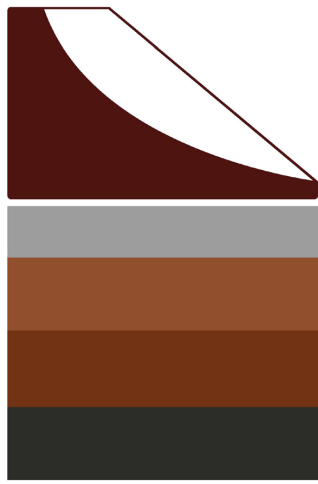


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Soils and Rocks



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18th ECSMGE, Lisbon 2024

Invited Editors:

António Alberto Correia, Universidade de Coimbra, Portugal

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Editorial to the Special Issue: 18th ECSMGE, Lisbon 2024

Editorial

The XVIII European Conference of ISSMGE 2024, the first that took place in Portugal, was a recognized successful organization of the Portuguese Geotechnical Society. Its outstanding success reflects the maturity that geotechnics in Portugal has achieved in the last 40 years within the European Union, as demonstrated by numerous remarkable engineering projects driven by strong Portuguese geotechnical expertise.

Among the most prominent themes of the conference were environmental topics and concerns, climate change mitigation, developments in water and energy infrastructures, risk assessment and safety evaluation, as well as innovative construction methods. Key subthemes included ground characterization, energy piles, life cycle assessment, offshore wind turbines foundation, ground improvement, monitoring and second generation of Eurocode 7.

The conference gathered an impressive number of contributions of remarkable scientific and technical quality. In recognition of this, the Editorial Board of Soils & Rocks decided to launch a Special Issue featuring a selection of representative technical papers, with clear potential to foster meaningful progress in geotechnics.

With the aim of providing a broad perspective on the main themes of the conference, fifteen papers were selected, highlighting innovative solutions from Portuguese, European and International authors, attesting the vitality and maturity of geotechnical engineering and its value to Society.

The Guest Editors of this Special Issue gratefully acknowledge the dedication and effort of the authors in the preparation of the extended version of their selected papers, and the collaboration of the Editorial Board of Soils & Rocks in the successful production of this special issue.

It is our expectation that the diversity and depth of the topics addressed in this Special Issue will make it a reference volume for the Geotechnical community.

António Alberto Correia

José Mateus de Brito

Cristiana Ferreira

Guest Editors for this Special Issue of Soils & Rocks



ARTICLES

Soils and Rocks
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The influence of the construction methodology on the modelled response of shafts

António M. G. Pedro^{1#} , David M. G. Taborda² , Lucas M. Repsold³ ,
Jorge Almeida e Sousa³ 

Article

Keywords

Shaft excavation
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Abstract

Shaft excavation is essential in modern cities, allowing for quick and direct access to the underground, where most transportation networks and utilities are being installed to reduce surface congestion. Selecting the appropriate construction methodology is critical to minimize ground movements, while ensuring structural stability and construction efficiency. This study assesses the performance of three typical construction methodologies – Excavation Before Support (EBS), Support Before Excavation (SBE) and Dual-Lined Shafts (DLS) – through a comprehensive numerical study. The validation of the adopted modelling approach for each methodology is performed by simulating three case studies in close proximity to each other. Several aspects of numerical modelling are discussed, such as the simulation of the hardening behavior of the sprayed concrete, the modelling the wall installation and their stiffness anisotropy. For each methodology, the influence of key variables is assessed through parametric studies, highlighting the importance of the excavation step height, the lining thickness and the embedded length of the wall. A final study, where all methodologies are compared for the same ground conditions, is carried out for two shaft diameters. Results indicate that SBE produces the smallest ground movements but induces the highest lining forces. In contrast, EBS originates higher ground movements due to significant soil decompression but smaller lining forces. DLS methodology exhibits an intermediate behavior, although more similar to that observed in EBS. These findings emphasize the importance of selecting an adequate shaft construction methodology and provide valuable information regarding the appropriate numerical simulation of each technique.

1. Introduction

The sustainable development of urban centers is directly interconnected with the exploration of their underground. Many cities are currently developing and implementing plans to minimize the impact of traffic congestion, lack of green spaces and affordable housing at ground surface by relocating and installing more efficient transportation networks and utilities in the subsurface (Admiraal & Cornaro, 2016; Bobylev, 2016; Lin et al., 2022; Debrock et al., 2023; Kuchler et al., 2024). Since the most effective way to access underground is through the construction of shafts it is unsurprising that several case studies reporting construction details and monitoring data have been published in recent years (Schwamb & Soga, 2015; Faustin et al., 2018; Walton et al., 2018; Quigg, 2019;

Falco et al., 2022; Royston et al., 2022). In contrast with earlier developments, where the typical shape of the shafts was rectangular, the vast majority of recently constructed shafts have a circular or ellipsoid shape (Pedro et al., 2019). These configurations are more efficient, minimizing lining thickness, since it takes advantage of the circumferential soil arching developed during the excavation, while reducing the associated ground movements (Muramatsu & Abe, 1996). Excavation methodologies have also improved significantly, making the excavation and support operations more efficient and safe (Doig, 2012). According to Faustin et al. (2018), shaft excavation techniques can be divided into two main categories. In the Excavation Before Support (EBS) method, excavation is performed prior to the installation of the support, while the opposite occurs in the Support Before Excavation

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(SBE) category, with the excavation only being executed after the support is in place. Naturally, the adequate selection of the shaft excavation technique depends on different factors, including ground and groundwater conditions, the shaft purpose and its dimensions, and the vulnerability of the surrounding structures. As demonstrated by Pedro et al. (2024), the construction technique and even the type of lining employed can have a considerable impact in the carbon dioxide emissions associated with the shaft construction, raising issues associated with the need to evaluate the sustainability of the chosen solution during the design process.

The EBS technique resembles the sequential excavation of a tunnel but in the vertical direction. The soil is sequentially excavated in steps, typically of 1 to 1.5 m in height, around the shaft perimeter followed by the installation of the lining, which typically consists of either pre-cast concrete segments (PCS) or sprayed concrete (SCL) (Gomes, 2008). In contrast, the SBE methodology involves pre-installation of the lining – typically diaphragm walls or bored piles – forming a circular retaining structure, after which the enclosed soil is excavated. The construction of shafts using jacked pre-cast segments is generally included within the SBE category, as the segments provide support before soil excavation (Faustin et al., 2018). However, to facilitate the jacking-in process, some soil is usually removed within the shaft causing its decompression. Depending on the amount of soil removed the technique can resemble either SBE (for minimal removal) or EBS (for significant removal). Faustin et al. (2018) also reported cases where both SBE and EBS methodologies were combined, termed dual-lined shafts (DLS). In these cases, the upper section of the shaft, where poorer materials were found, consisted of a diaphragm wall, while in the lower section the excavation followed a sequential approach with SCL applied.

Despite the significant differences between the shaft construction techniques, there is a lack of published studies assessing their relative impact, with most of the research focusing on evaluating the influence of shaft dimensions and soil properties on the ground deformations for a specific excavation technique (Schwamb & Soga, 2015; Le et al., 2019; Taborda et al., 2025b). The study performed by Pedro et al. (2024), although primarily focusing on evaluating the sustainability of construction methods, showed that in EBS the unsupported excavation caused considerable soil decompression, resulting in significant settlements and lateral wall movements. In contrast, in SBE, ground movements were smaller, and heave was inclusively measured at the top of the wall due to the soil decompression at the excavation base. Pedro et al. (2024) also observed that the pre-installed lining in SBE constrained the surrounding soil, leading to higher forces in the lining compared to those observed when adopting an EBS methodology. In the latter case, the forces acting on the lining were determined by the height of each excavation step, as pressures are transferred to the lining above and the soil beneath through the development of a vertical arching effect (Pedro et al., 2024; Repsold et al., 2024).

Through a comprehensive set of numerical analyses, this paper aims to compare the behavior of a shaft excavated employing three different construction sequences – EBS, SBE and DLS. The study compares key aspects of each methodology and evaluates their impact on the ground movements, pressures and forces acting on the lining. The validation of the numerical model is performed by comparing the modelled behavior against monitoring data gathered from three case studies, each employing one of these methodologies. In the validation process, various aspects related to the modelling procedures that are often neglected are analyzed in order to determine the most suitable approaches for simulating accurately each type of construction methodology.

2. Numerical modelling of shafts

2.1 General considerations

With the development of major tunnelling projects in London a large database was compiled on the construction of shafts using different techniques. Despite the differences in construction methods and shaft geometries the results presented by Faustin et al. (2018) show that a good performance was achieved in all cases. From these, three shafts were selected in this paper for analysis, each representing a distinct construction methodology, as they were all excavated in the same materials, reached similar depths, and had detailed information and measurements of settlements for the key excavation stages. The geometry and ground conditions at the three site locations are illustrated in Figure 1.

In the London Power Tunnels project, the EBS technique was used in the construction of the Channel Gate Road (CGR) shaft. This shaft had an outer diameter (D) of 13.1 m and reached a depth (H) of 32.9 m, with the upper 22.2 m supported by PCS with a thickness of 0.325 m, while, in lower part of the shaft, 0.312 m thick SCL was placed. The Whitechapel Cambridge Heath (WCH) shaft, built as part of the Crossrail project, had an outer diameter of 31.2 m and a depth of 32 m. The SBE technique was applied in this shaft, with the installation of a 1.5 m thick and 40.5 m deep diaphragm wall, before excavating the enclosed soil. In the Northern Line Extension project, Kennington Green (KG) shaft, with an outer diameter of 16.2 m and 26.1 m deep, was constructed using the DLS methodology. Initially, secant bored piles with a diameter of 0.60 m and a depth of 13.6 m were installed, followed by the excavation of the enclosed soil. In the second phase, the remaining soil was excavated sequentially in steps and 0.6 m thick SCL was applied along the contour of the excavation, following the traditional EBS approach. Faustin et al. (2018) also presented results of settlements induced by jacked shafts, but the information about the construction sequence was incomplete, preventing the simulation of its excavation without significant assumptions

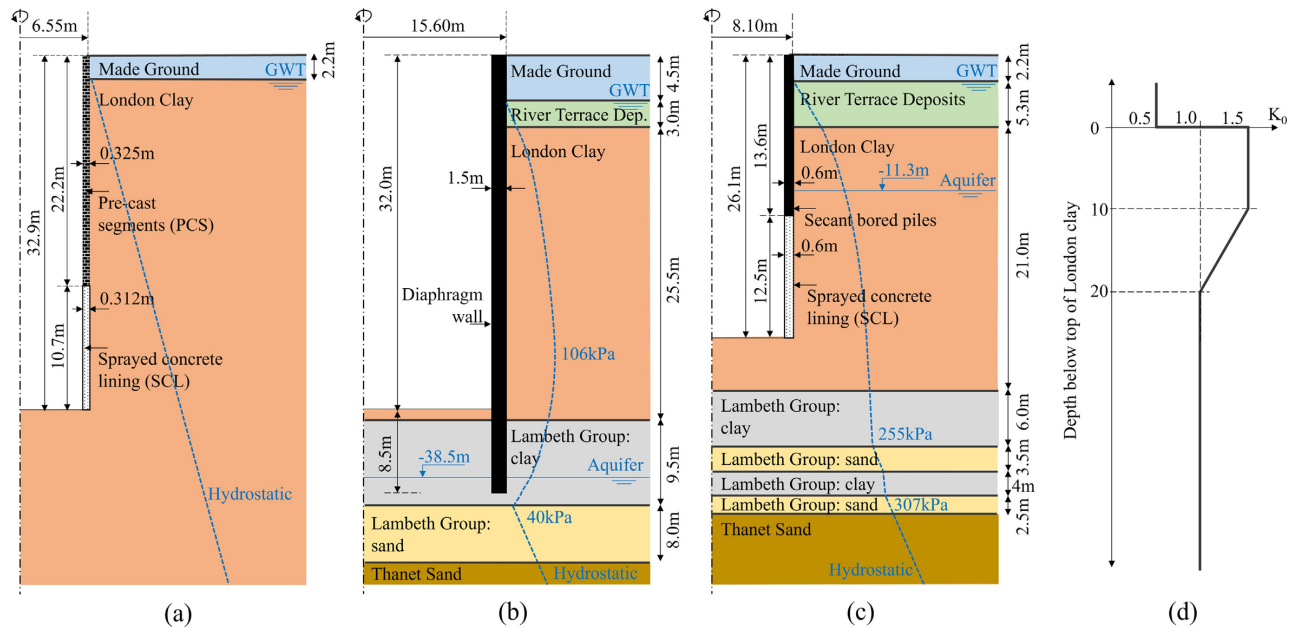


Figure 1. Geometry and ground conditions at the site locations: (a) CGR; (b) WCH; (c) KG; (d) K_0 profile.

that could impact the results. As a result, jacked shafts were excluded from this study.

At all three sites, variations of the typical ground conditions in London are found (King, 1981). At CGR, beneath a thin layer of Made Ground, there is a thick deposit of London Clay extending to more than 75 m. At WCH, the superficial deposits are thicker, with Made Ground and River Terrace Deposits having a combined thickness of 7.5 m. Below, a 25.5 m thick layer of London Clay is followed by Lambeth Group (17.5 m thick), with its typical interbedded clay and sand layers; at greater depths lies the Thanet Sand deposit (Crossrail Ltd., 2011). The stratigraphy at KG is similar to that of WCH, with slight differences in the thickness of layers. According to Schroeder et al. (2004) who considered a site in close proximity to the KG site, the Lambeth Group presents more frequent intercalations of sand and clay.

As discussed in Simpson et al. (1989), the groundwater profile in the London basin is typically underdrained, with various factors, including the thickness of the soil formations, the variation of permeability with depth and the location of the lower aquifer, affecting the pore water pressure distribution. At all sites the ground water table (GWT) was found at the bottom of the Made Ground. However, while at CGR a hydrostatic profile can be assumed, due to the significant thickness of London Clay, at the other two sites an underdrained profile is encountered. According to Crossrail Ltd. (2011), at WCH, the ground water pressure in the top of the sandy layer of the Lambeth Group is approximately 40 kPa, indicating that the aquifer level is located approximately 4 m above, at a depth of 38.5 m, while at KG, the aquifer is estimated at approximately 11.3 m depth (Schroeder et al. (2004)). By imposing these groundwater conditions, the

pore water pressure distribution can be estimated through a steady-state analysis, assuming the variations with depth of permeability detailed in the subsequent section. The obtained distributions are represented in Figure 1, with WCH exhibiting a much more pronounced underdrainage, due to a significantly deeper lower aquifer. It should be noted that, prior to excavation dewatering was carried out through sumps or depressurization wells located within the shafts. According to Faustin et al. (2018), there was little drainage of groundwater associated with the shaft construction due to the low permeability of the formations, making settlements likely to be very small. Consequently, in this study, and in the absence of monitoring data, the settlements induced by prior dewatering were disregarded.

2.2 Details of the numerical analysis

The numerical simulations of the shafts excavation were performed using Plaxis 2D (Bentley Systems, 2024), with all analyses performed as coupled consolidated and making use of the axisymmetry of the problem. The top boundary corresponded to the ground surface and the left-hand side boundary to the axis of symmetry. To minimize the boundary effects, the right-hand side and bottom boundaries were positioned at distances of 100 m and 70 m, respectively. The water flow and the displacements in the horizontal direction were restricted in both lateral boundaries while along the bottom boundary no displacements were allowed. The domain was discretized in 15-noded triangular elements each with 12 Gaussian integration points for determining the displacements and stress states (Bentley Systems, 2024), respectively. 5-noded plate elements were also used in

some of the models to simulate the shaft lining. In the zone around the shaft wall a higher concentration of elements was employed in order to capture the higher stress and strain variations expected in that zone. Lastly, 12-noded interface elements were placed between the lining and the retained soil to prevent water inflow and to simulate the soil-structure interaction. Figure 2 shows the detail around the shaft of the mesh employed in each of the case studies analyzed.

The initial conditions were generated assuming a geostatic profile, with the vertical total stresses determined based on a unit weight of 18 kN/m³ for the Made Ground and 20 kN/m³ for all other formations. The horizontal total stress was calculated using the at-rest coefficient of earth pressure (K_0) profile presented in Figure 1d, which replicates that adopted on previous studies (Gawecka et al., 2017; Sailer et al., 2019; Taborda et al., 2025b). As previously mentioned, the initial pore water pressure profile was obtained through a steady-state analysis, ensuring equilibrium based on the specified GWT and aquifer locations.

A linear elastic-perfectly plastic behavior was adopted for the Made Ground, with a stiffness characterized by $E = 10$ MPa and $\nu = 0.2$ and its strength described by the Mohr-Coulomb failure criterion with $\phi' = 30^\circ$, $c' = 0$ kPa and $\psi' = 0^\circ$ (Taborda et al., 2025b). For the other formations the non-linear elastic-perfectly plastic IC MAGE M01 model, developed by Taborda et al. (2024a) and implemented into Plaxis 2D as a user-defined soil model, was employed, as it has been shown to effectively capture the complex behavior of these formations

in different geotechnical applications (Gawecka et al., 2017; Sailer et al., 2019; Taborda et al., 2025b). In this model, strength is characterized by the Mohr-Coulomb failure criterion and the stiffness by non-linear elastic relationships for both the tangent shear and bulk moduli (Taborda et al., 2016). The specific implementation of the model for Plaxis User-Defined Soil Model is detailed in Taborda et al. (2024b). Table 1 presents the parameters employed in the analysis, which were adopted from the previous studies by Gawecka et al. (2017) and Sailer et al. (2019).

The Made-Ground was assumed to be dry since it was above GWT, while for the sandy materials (River Terrace Deposits, Lambeth Group: sand and Thanet Sand) a drained condition with an isotropic permeability of 1×10^{-5} m/s was adopted. For the London Clay and Lambeth Group: clay formations, an undrained condition was specified, with anisotropic permeability given by Equation 1:

$$\log_{10} k_v = 0.0215 \cdot (y - y_{LC}) - 9.427 \geq 1 \times 10^{-12} [m/s] \quad (1)$$

where k_v represents the vertical permeability, y is the vertical coordinate on the model and y_{LC} is the coordinate of the top of the London Clay. As suggested by Taborda et al. (2025b) an anisotropy ratio of 2.0 was adopted between the horizontal and vertical permeabilities ($k_h = 2 k_v$). This permeability model was introduced in Plaxis as a user-defined flow model (Bui et al., 2019) as detailed in Taborda et al. (2023).

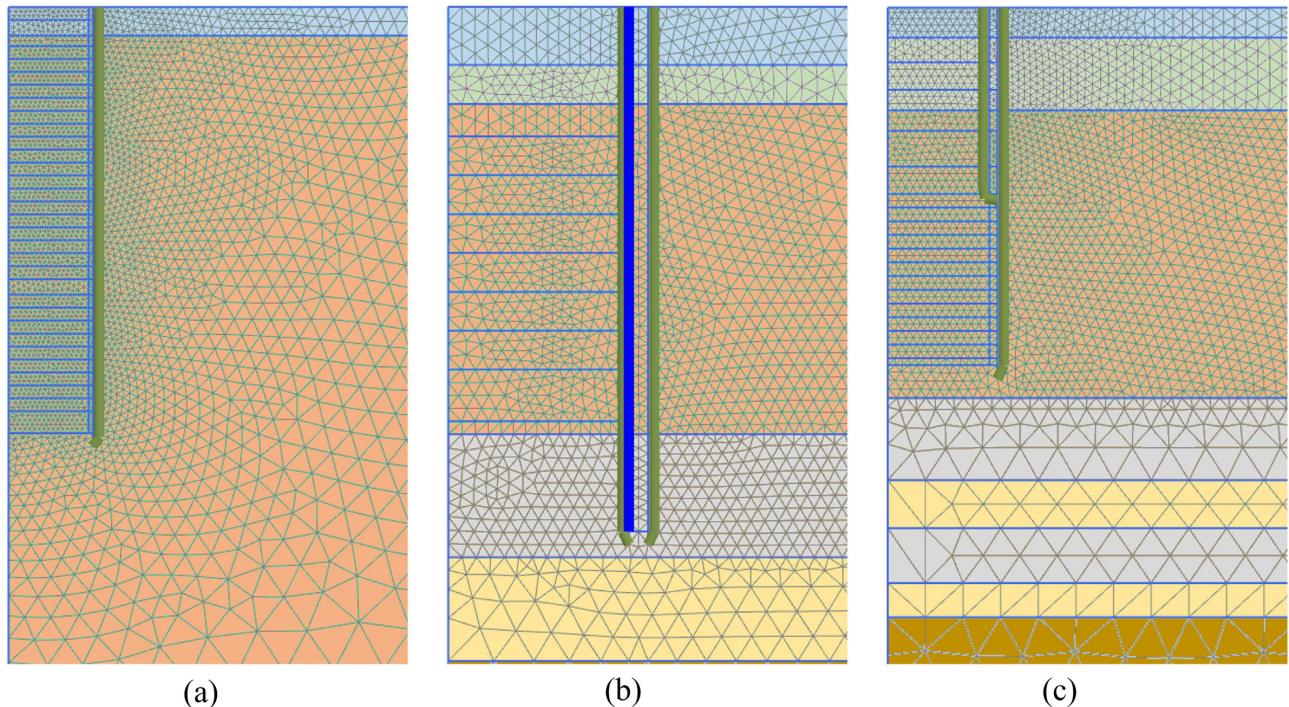


Figure 2. Detail around the shaft of the mesh employed in the numerical simulations: (a) CGR; (b) WCH; (c) KG.

Table 1. Parameters of the soils adopted in the analyses.

Parameter	River Terrace Deposits	London Clay	Lambeth Group: clay	Lambeth Group: sand	Thanet Sand
p'_{ref}	100.0	100.0	100.0	100.0	100.0
$f' (^{\circ})$	35.0	25.0	27.0	34.0	40.0
$c' \text{ (kPa)}$	0.0	5.0	25.0	0.0	0.0
$\gamma' (^{\circ})$	17.5	12.5	13.5	17.0	20.0
$G_{ref} \text{ (kPa)}$	41939.61	51743.11	51924.52	81346.31	65275.23
m_G	1.0	1.0	1.0	1.0	1.0
a	0.000145	0.000056	0.000110	0.000015	0.000046
b	1.00	0.90	0.95	1.00	0.85
$R_{G,min}$	0.03511	0.06450	0.04662	0.14557	0.02631
G_{min}	3000.0	2667.0	2667.0	1000.0	2000.0
$K_{ref} \text{ (kPa)}$	49843.08	26692.73	61331.71	49843.08	29813.53
m_K	1.0	1.0	1.0	1.0	1.0
r	0.000247	0.000127	0.000065	0.000026	0.000155
s	1.25	1.80	1.40	0.90	1.10
$R_{K,min}$	0.15440	0.13275	0.07589	0.08377	0.27947
K_{min}	3000.0	5000.0	5000.0	5000.0	5000.0

For modelling the lining of the shafts, four additional materials were required. For the PCS, diaphragm wall and bored piles a linear elastic behavior with the parameters presented in Table 2 was adopted.

To describe the behavior of SCL, a non-linear time-dependent model was employed. The model IC MAGE M07 was implemented into Plaxis as a user-defined soil model (Taborda et al., 2025a) and is based on the Model Code 90 proposed by CEB-FIP (1993), where the stiffness is defined by Equation 2:

$$E(t_{conc}) = \sqrt{\exp \left[s \cdot \left(1 - \left(\frac{28}{t_{conc} / 1 \text{ day}} \right)^{1/2} \right) \right]} \cdot E_{28} \quad (2)$$

where E_{28} is the stiffness of the material at 28 days, t_{conc} [days] is the age of the concrete and s is a parameter which depends on the type of concrete. In this study values of $E_{28} = 30$ GPa and $s = 0.25$ were used, together with $\nu = 0.2$.

Lastly, for the interface elements, a strength reduced to two-thirds of that of the adjacent soil was adopted, while the normal and tangential stiffnesses were set to 10^6 kN/m³, in agreement with the values adopted by previous studies (Pedro et al., 2024; Taborda et al., 2025b).

2.3 Channel Gate Road shaft (CGR)

The numerical simulation of the CGR shaft followed the typical EBS sequence as described in Taborda et al. (2025b). After the generation of the initial pressures, the shaft was excavated and supported in 33 stages. In each stage, 1 m of

Table 2. Parameters of the linings adopted in the analyses.

Parameter	PCS	Diaphragm wall	Bored piles
$\gamma \text{ (kN/m}^3\text{)}$	25.0	25.0	25.0
$E \text{ (GPa)}$	37.0	30.0	30.0
n	0.2	0.2	0.2

soil was excavated, and the lining of the precedent excavation step was installed. This procedure was followed in all stages, except for the first two, which corresponded to the excavation of the Made Ground, where the excavation and support were performed simultaneously to avoid soil collapse, and in the last stage, where only the final piece of lining was installed. As illustrated in Figure 1, the upper part the shaft was supported by underpinned PCSs, while SCL was used in the lower part. An excavation rate of 1 m/day was adopted in all analyses, resulting in a total simulated duration of 32.9 days. To evaluate the influence of SCL hardening, two analyses were performed in which a constant value of 15 and 37 GPa were adopted for the Young's modulus of the SCL. The lower value represents the stiffness after about 1 day, according to Model Code 90 (CEB-FIP, 1993), while the higher value corresponds to a support stiffness equivalent to PCSs. Since the simulation of the lining is often performed using plate elements instead of solid elements, two additional analyses were carried out ($B-E = 15$ GPa; $B-E = 37$ GPa), each with a constant stiffness of 15 and 37 GPa, to evaluate the influence of using this type of element, instead of the solid elements used in the original analyses.

The final settlements (S_v) normalized by CGR shaft depth (H) are plotted in Figure 3 against the normalized distance behind the wall (x/H) for the five analyses performed. The results show that including SCL hardening leads to a

better agreement with shaft monitoring data, particularly for the data located nearer to the shaft wall. At greater distances the model overestimates settlements and fails to reproduce the slight heave recorded on site. The effect of SCL hardening cannot be captured by using a constant Young's modulus, with even the analyses with the lower stiffness (15 GPa) predicting smaller settlements. The results also show that the difference between adopting the two extreme values of stiffness for solid elements is small, with the analysis where $E = 37$ GPa showing only slightly smaller settlements. Interestingly, when plate elements are used the impact of stiffness increases, leading to larger differences in settlements between the two stiffnesses considered. These results emphasize the importance of considering lining hardening in the modelling, which can only be accounted for using solid elements.

2.4 Whitechapel Cambridge Heath shaft (WCH)

Modeling the SBE methodology is apparently simpler, as it requires fewer stages compared to EBS methodology. After generating the initial stresses, the lining is installed, followed by the excavation of the enclosed soil until the shaft reaches its final depth. This approach was generally adopted in the excavation of WCH shaft, with the excavation of the soil carried out over 10 stages. One aspect that is often disregarded in SBE simulation is the installation of the lining, with most studies considering it simply as wished-in-place (WIP). However, for the WCH shaft, data regarding the installation of the diaphragm wall was available (Faustin et al., 2018), allowing for an assessment of its impact on the final settlements. As acknowledged in different studies there is some uncertainty in the simulation of the wall installation, particularly for deep walls, as the pressure of the wet concrete is not linearly distributed with depth (Lings et al., 1994; Gourvenec & Powrie, 1999). The numerical simulation of the wall installation was performed in three stages, as proposed by Schwamb & Soga (2015): (1) the panel was excavated and a pressure was applied in the contour of the excavation to simulate the presence of the bentonite (linearly increasing with depth, based on $g = 11$ kN/m³); (2) an additional pressure representing wet concrete ($g = 25$ kN/m³) was applied with its distribution with depth following that proposed in Gourvenec & Powrie (1999); (3) finally, the elements of the diaphragm wall and interface were activated and the boundary between them and the soil set to impermeable. Following the observation related to the importance of modelling concrete hardening when analysing the CGR shaft, model IC MAGE M07 was used to simulate the diaphragm wall material. It was assumed that the installation of the diaphragm wall took 1 day, followed by a rest period of 27 days to allow the concrete to cure before shaft excavation began. Since an excavation rate of 1 m/day was also adopted, a total duration of 60 days was simulated.

Several studies (Cabarkapa et al., 2003; Zdravkovic et al., 2005; Schwamb & Soga, 2015) have highlighted the importance of appropriately modelling wall stiffness in the circumferential

direction, as the presence of joints between the diaphragm panels limits the development of hoop stiffness. In the absence of studies quantifying the anisotropy ratio between the vertical and circumferential stiffnesses (E_z/E_q), those studies tested different ratios and observed a significant increase in ground movements as the stiffness anisotropy ratio increases or, equivalently, as the circumferential stiffness reduces. To evaluate the importance of this aspect several analyses were performed with different stiffness anisotropy ratios.

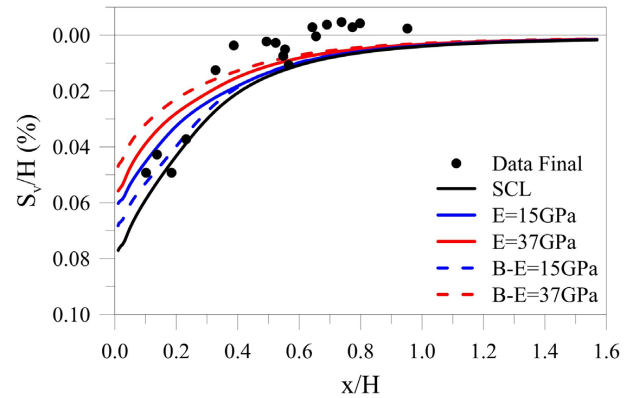


Figure 3. Settlements determined for CGR shaft.

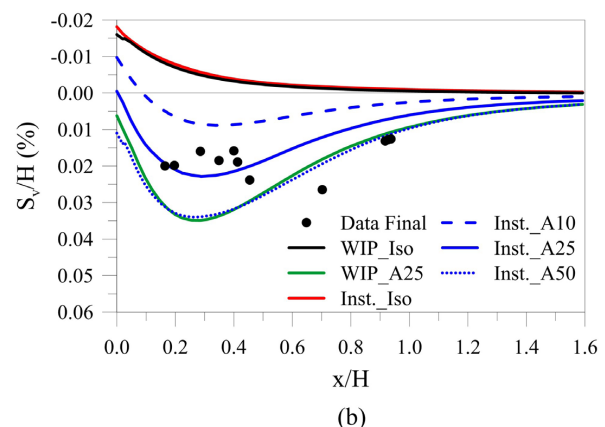
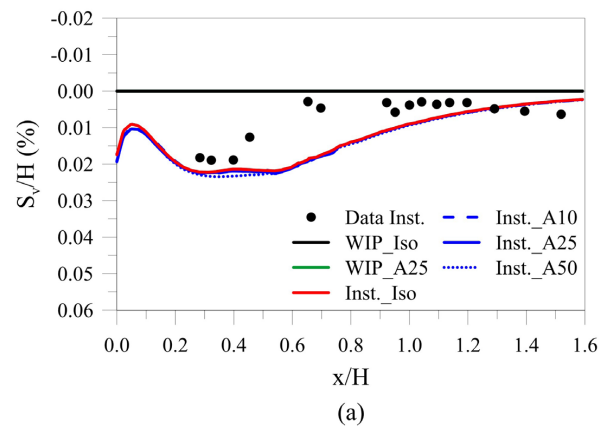


Figure 4. Settlements determined for WCH shaft: (a) Installation; (b) Final excavation.

Figure 4 illustrates the settlements induced by the diaphragm wall installation and those caused solely by shaft excavation (i.e. zeroed after the rest stage). As expected, no movements are obtained when the diaphragm wall is considered WIP. However, when the installation is simulated a reasonable agreement with the monitoring data is obtained, with the calculations slightly overestimating the maximum settlement and predicting a wider settlement trough. The differences between the analyses with different stiffness anisotropy ratios are negligible, with all analyses yielding identical results. In contrast, the predicted settlements due to the shaft excavation are strongly influenced by the stiffness anisotropy ratio. When an isotropic behavior is adopted (anisotropy ratio of 1) only heave is predicted, with its maximum value at the shaft wall and gradually decreasing with distance. As the stiffness anisotropy ratio increases to 10 (Inst._A10) or 50 (Inst._A50), settlements occur, reaching a maximum at a normalized distance of about 0.3 in all cases. Near the wall, the settlements are smaller and affected by its presence, due to the upward movements caused by the soil decompression at the base of the excavation. For the specific case of the WCH shaft, a stiffness anisotropy ratio of $E_z/E_q = 25$ (Inst._A25) provides the best agreement with monitoring data. It is interesting to note that if WIP is adopted, there is no difference in settlements if isotropic behavior is adopted, but the best fitting stiffness anisotropy ratio is lower than 25, indicating that neglecting installation effects could impact the choice of affects the parameter selection. The influence of the stiffness anisotropy ratio is highlighted in Figure 5, where the maximum normalized settlement is plotted for several ratios tested, ranging from 1 (isotropic) to a 100. The results show that heave only occurs under isotropic behavior, while the maximum settlements increase almost linearly with the increase of the stiffness anisotropy ratio.

2.5 Kennington Green shaft (KG)

The DLS methodology adopted in the KG shaft combines both SBE and EBS techniques and, consequently, the simulation

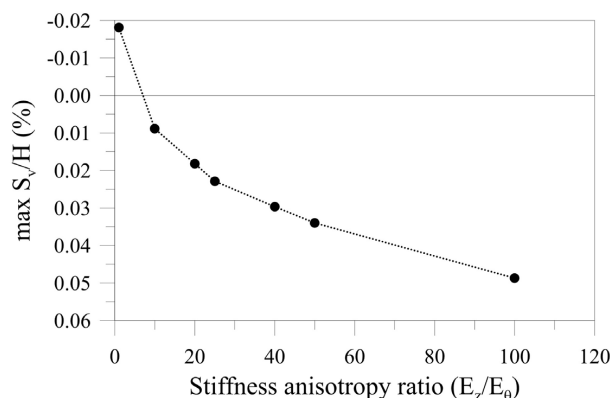


Figure 5. Influence of the stiffness anisotropy ratio in the maximum settlements determined for WCH shaft.

of its excavation followed the principles described in the previous sections. As a result, the bored piles were installed first, followed by soil excavation over 7 stages until the tip of the piles was reached. Subsequently, excavation proceeded similarly to the CGR shaft where, in each stage, 1 m of soil was excavated, and the lining of the preceding excavation step was installed until the shaft reached its final depth. Similar to the previous case studies, additional analyses, following the

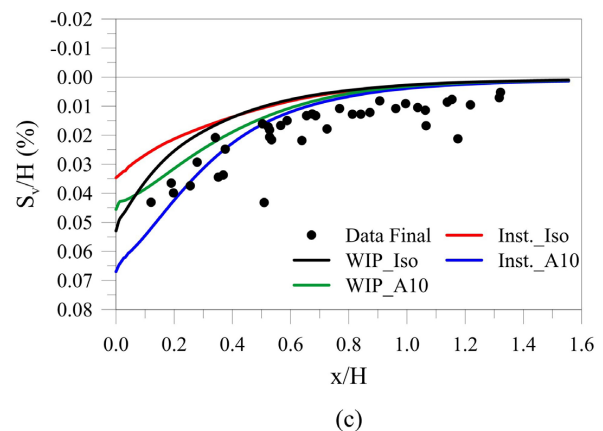
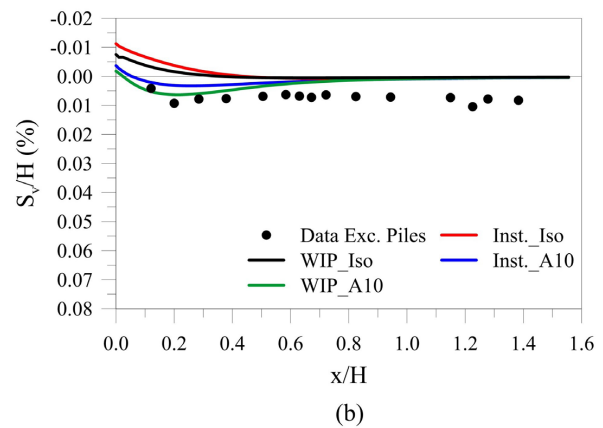
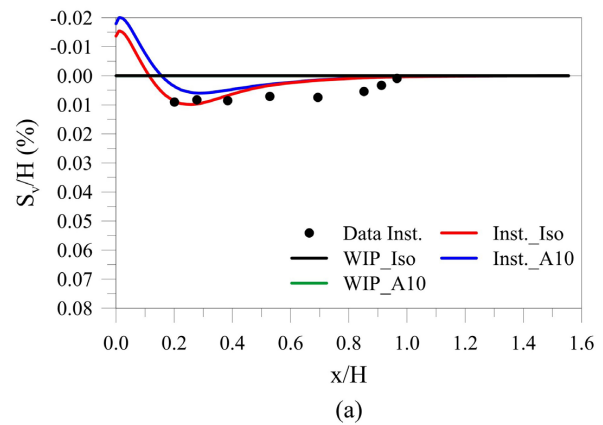


Figure 6. Settlements determined for KG shaft: (a) Installation; (b) Excavation of piles; (c) Final excavation.

same procedures applied for the WCH shaft, were performed to assess the effect of pile installation and stiffness anisotropy.

Figure 6 presents the settlements induced by the bored piles installation and those caused solely by shaft excavation (i.e. zeroed after the rest stage) for the stages where the tip of the piles is reached (end of SBE technique) and when full excavation is completed. A stiffness anisotropy ratio of 10 for the secant pile wall portion of the lining provided the best fit to the monitoring data across all stages. The installation of the bored piles induced very small settlements (less than 0.01%), which are slightly better predicted when an isotropic behavior of the lining was adopted. When the excavation reached the tip of the piles minimal settlements were also observed. Similar to the WCH shaft, an isotropic behavior resulted in heave at this stage, and only by introducing stiffness anisotropy was it possible to predict reasonably the measured settlements. At the final excavation stage, the settlements increased considerably, with the best prediction achieved when the installation of piles is considered and a stiffness anisotropy ratio of 10 is adopted, although the model predicts a slightly narrower settlement trough than that observed at the site.

3. Factors affecting the construction methodologies

Using the calibrated models as reference, a parametric study was carried out to evaluate the influence of key parameters for each methodology. The analysis focused on assessing the impact on ground movements, pressures and forces in the linings by varying parameters such as lining thickness, excavation step and length of the embedded wall.

3.1 EBS – Excavation Before Support

One of the main drawbacks of the EBS methodology is the high number of excavation-support cycles, which increase both cost and construction time. One option to minimize this issue is to increase the excavation step height. To evaluate the impact of such approach, additional analyses were performed using a 2 m height excavation step (double the height used in the reference CGR analysis). Another key aspect is the thickness of the lining (t), which varies significantly from case to case (Faustin et al., 2018), even under similar ground conditions and shaft geometries. This aspect was analyzed by increasing the lining thickness (PCS+SCL) to 0.600 m. A final analysis was performed in which the entire lining, except for the initial 2.2 m excavated on Made Ground, consisted of SCL, replacing the PCS employed in the upper part of the shaft, to assess if this aspect has a significant influence on the shaft behavior.

The settlements and the primary results for the lining are presented in Figures 7 and 8, respectively, for all additional analyses performed. As expected, increasing the excavation

step to 2 m height induces higher settlements due to the greater soil decompression, with the maximum settlement near the wall increasing by about 25%. The influence of this aspect is also observed when the lining thickness is increased from 0.325 to 0.600 m, though in this case, the maximum settlement increase is only of approximately 19%. Interestingly, increasing the lining thickness and, consequently its stiffness, results in higher settlements rather than a decrease. This outcome is inherent to the EBS methodology and is justified by the greater weight of the lining. When a new excavation cycle begins, the lining is left hanging, with its weight acting as a destabilizing force, and naturally, a heavier lining leads to greater settlements. This effect is minimized on-site by adopting a construction sequence which involves several excavation steps in the circumferential direction rather than completing a full excavation cycle at once, though this approach increases considerably the number of operations required for shaft construction and cannot be reproduced numerically if axisymmetric conditions are assumed.

In all analyses, the horizontal displacements in the lining follow a similar pattern, increasing almost linearly with depth. The oscillations observed at each excavation step are a consequence of the EBS methodology, with the lower part of the lining installed at the previous step exhibiting higher displacements due to soil decompression and arching effect. Naturally, these oscillations and overall displacements increase with a larger excavation step height but are almost unaffected by the lining thickness. The total pressure acting on the lining is similar across all analyses, increasing almost linearly with depth until reaching the depth of the last excavation cycle, where it approaches zero due to the exposed soil surface. It should be noted that an averaging smoothing filter was applied to the total pressures results to remove the extremely high numerical stress concentrations that occur at excavation step transitions. These stress concentrations cause the fluctuations observed in the hoop forces within each excavation step height. On average, the hoop force tends to increase with depth, with smaller values

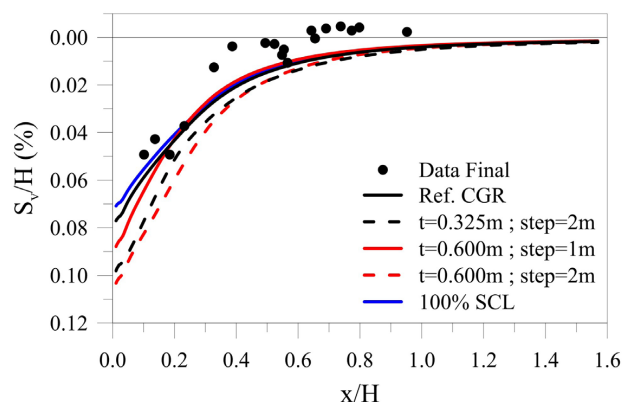


Figure 7. Influence of lining thickness and excavation step height on settlements in the EBS methodology.

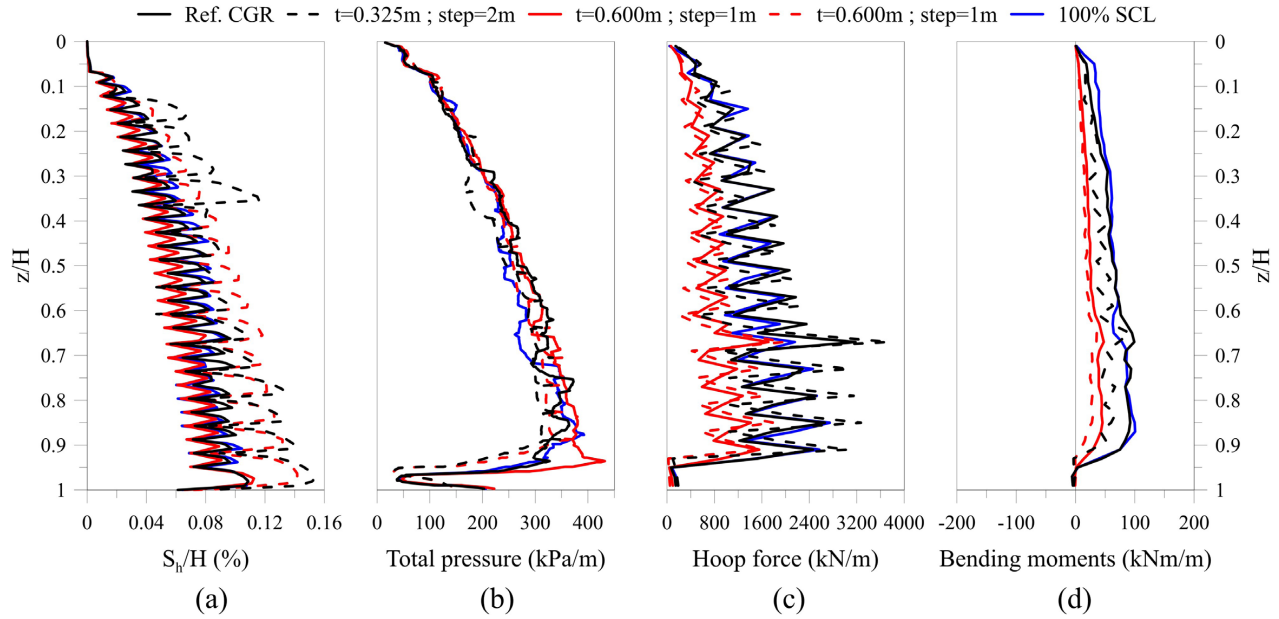


Figure 8. Influence of lining thickness and excavation step height on the lining in the EBS methodology: (a) horizontal displacement; (b) pressure; (c) hoop force; (d) bending moments.

observed for thicker linings. Increasing the excavation step height also slightly increases the hoop force values. Due to the construction process, the bending moments in the lining are small, not surpassing 100 kNm/m in the reference analysis, being even lower when thicker linings or greater excavation step heights are adopted. The analysis where PCS was replaced by SCL (100% SCL) provided very similar results to those of the reference analysis, indicating that the combination of the two materials has no significant impact on the overall shaft behavior.

3.2 SBE – Support Before Excavation

The thickness of the retaining structure and the length of the embedded wall are critical parameters in controlling ground movements and lining forces in deep excavations (Fernandes, 2015). To investigate the influence of these parameters on shaft construction, additional analyses were performed using the WCH shaft case as reference. In these analyses, the diaphragm wall thickness (t) was reduced by half to 0.75 m, and the embedded length (L) was reduced to 50 and 0%, with the latter simulating the absence of a wall beneath the excavation base.

Figure 9 shows that reducing the wall thickness has a considerable impact on settlements, with the maximum settlement increasing by about 75%, reaching 0.035% H . Reducing the embedded length to half has almost no impact on the settlements, while its complete removal originates a considerable increase in settlements between the wall and a distance of approximately 0.35 H from its edge.

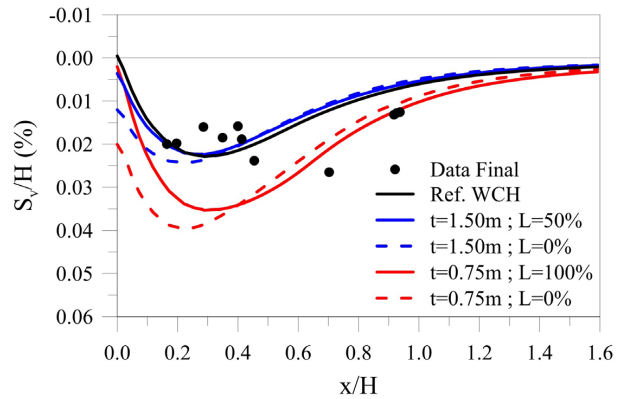


Figure 9. Influence of lining thickness and embedded length of the wall on settlements in the SBE methodology.

The results for the lining, presented in Figure 10, confirm the influence of the wall thickness on shaft behavior. With a thinner wall, larger lateral movements are observed, reaching a maximum in the middle part of the shaft of 0.12% H (a 50% increase). As a result, the total pressure decreases by about 25%, to approximately 150 kPa over the London Clay thickness. The reduction in lining forces is even greater, with the maximum hoop force observed in the middle part of the shaft decreasing from 3300 to 2250 kN/m (-32%), while the maximum absolute bending moment reduces from 1400 to just 430 kNm/m (-69%). In contrast, the embedded length appears to have a reduced impact, with slight reductions of about 5% in horizontal

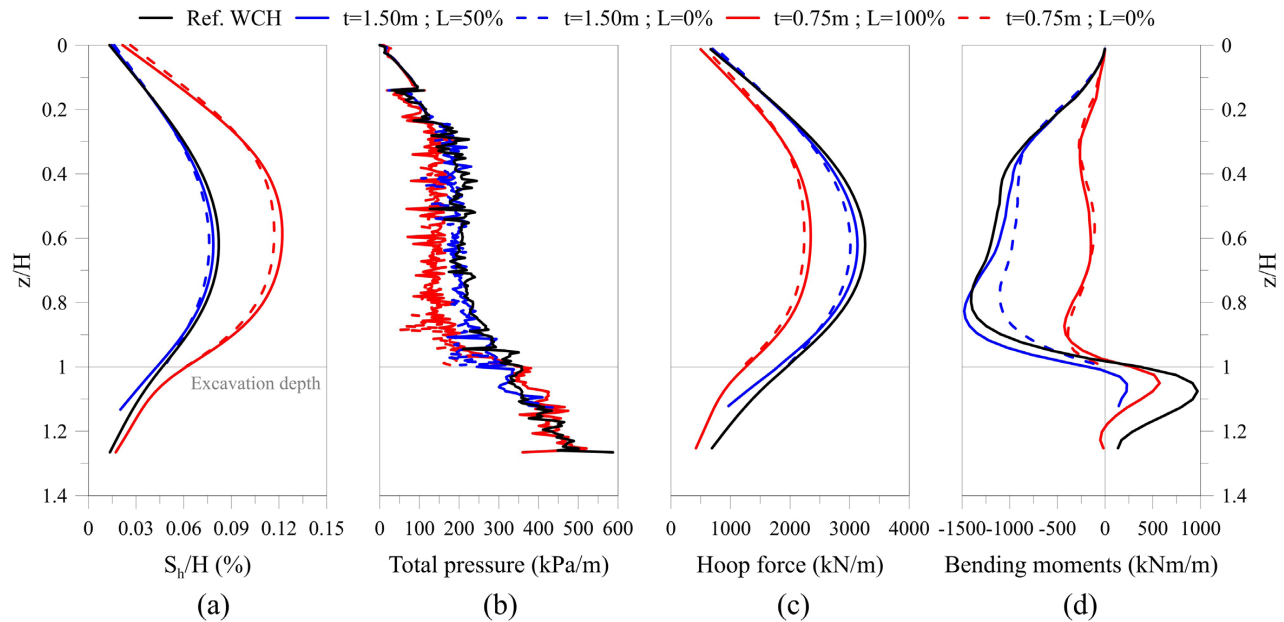


Figure 10. Influence of lining thickness and embedded length of the wall on settlements in the SBE methodology: (a) horizontal displacement; (b) pressure; (c) hoop force; (d) bending moments.

movements, 9% in the hoop force and 20% in the bending moment being observed. The reduced impact of the embedded length of the diaphragm wall is related to the auto-balanced circular shape of the shaft and the favorable mechanical properties of London Clay and Lambeth Group. However, as demonstrated by Fernandes (2015) the embedded length of the wall is fundamental not only for controlling ground movements but also for ensuring stability against base heave mechanisms. Therefore, its omission should not be considered without proper analysis.

3.3 DLS – Dual-Lined Shafts

In the case of the DLS methodology, one of the main aspects is related to the extent of SBE technique within the overall shaft height. A smaller SBE percentage implies that most of the shaft excavation follows the EBS approach, whereas a higher percentage resembles a behavior more similar to the SBE technique. Naturally, the extent of SBE is usually determined by ground conditions, as the pre-installed support system is essential for stabilizing less competent soils during the excavation. To assess the influence of the balance between these two different support systems, two additional analyses were carried out based on the KG case. Given that SBE accounts for 52% of the KG shaft, the two analyses considered SBE extents of 30% and 75%, with the former being conditioned by the thickness of the superficial deposits, which require prior support before excavation.

The settlements obtained from the analyses are presented in Figure 11 for the stages where the excavation reaches the tip of the piles (i.e. marking the end of the SBE technique

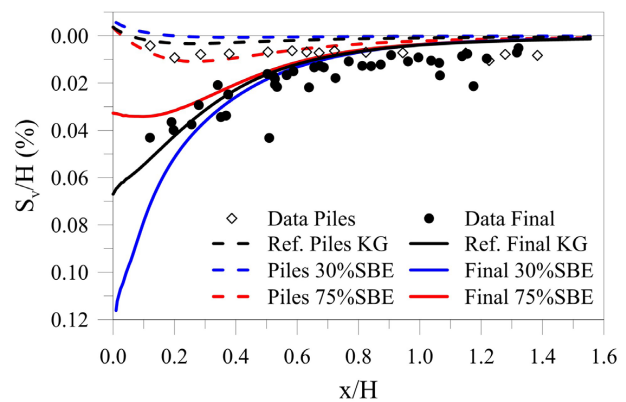


Figure 11. Influence of the percentage of SBE technique on settlements in the DLS methodology.

and thus corresponding to a different depth for each of the three analyses shown) and for the final excavation. When the excavation reaches the tip of the piles, a slight heave is observed near the wall in all cases. At greater distances, minor settlements occur, which increase with the SBE percentage. This result is justified by the much deeper excavation level associated with a higher SBE extent. In contrast, the settlements observed for the final excavation stage increased considerably with the decrease in the SBE percentage. For 75% SBE, a maximum settlement of $0.033\%H$ is determined, while for 30% SBE the settlement increased to approximately $0.115\%H$. Moreover, the shape of the settlement trough also varied considerably with the increase of the SBE percentage from a concave to a spandrel curve (Hsieh & Ou, 1998).

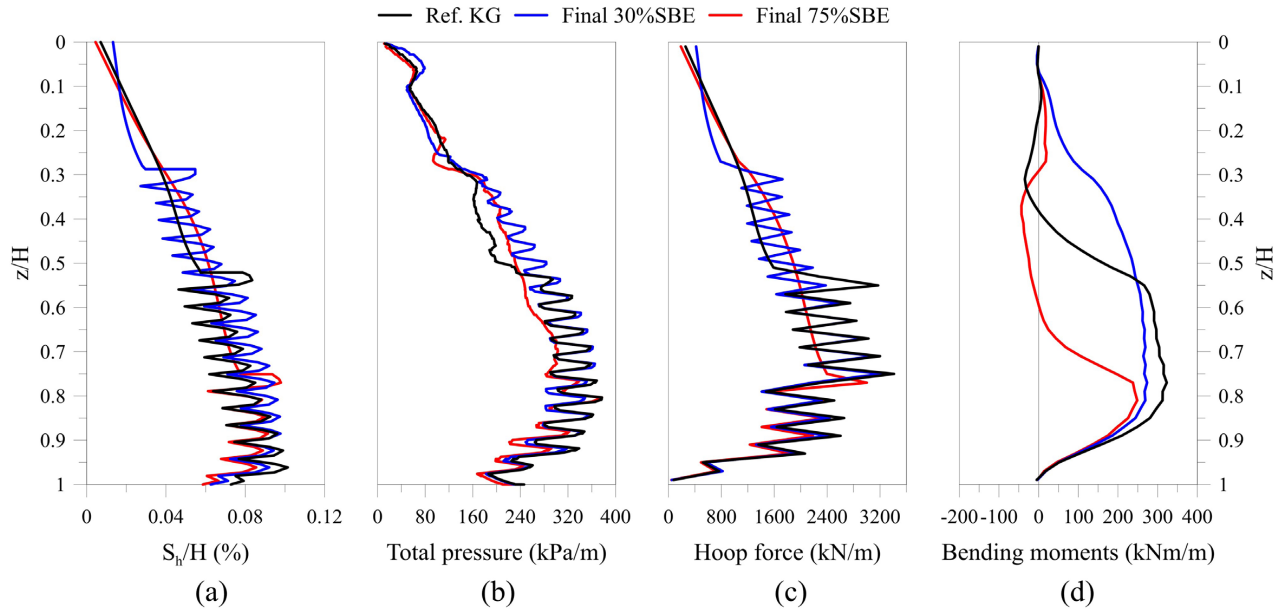


Figure 12. Influence of the percentage of SBE technique on settlements in the DLS methodology: (a) horizontal displacement; (b) pressure; (c) hoop force; (d) bending moments.

The lining results for the final excavation stage are presented in Figure 12 and show that increasing the percentage of SBE has a limited influence on the overall behavior of the lining. After transitioning from SBE to EBS, the lateral movements begin to fluctuate between the top and the base of the excavation steps, as occurred in the EBS study, but following a general trend comparable to that observed in the SBE extent. A similar trend is observed in total pressures and hoop forces, with a slight reduction as the SBE percentage increases. As for the bending moments, small values – within the same order of magnitude of those in EBS analyses – are predicted. The pressures and lining forces tend to increase with depth, reaching their maximum at about 75% of the total shaft height.

4. Comparison of construction methodologies

This section presents a comparison of the different shaft construction methodologies. Due to its simplicity, the stratigraphy and ground conditions observed at CGR site were selected for this study. Since the analyzed shafts had different geometries, two idealized shafts were considered: both with an excavation depth of 30 m – consistent with the three case studies – but with diameters of 15 m (similar to CGR and KG) and 30 m (similar to WCH). To ensure a realistic comparison the lining characteristics from each case study were adopted, with minor adjustments to accommodate the idealized geometries. In total, six analyses were performed, two for each of the three construction methodologies, corresponding to the two shaft diameters.

The settlements predicted by the six analyses are displayed in Figure 13, with the results showing that a larger diameter corresponds to higher settlements, regardless of the construction

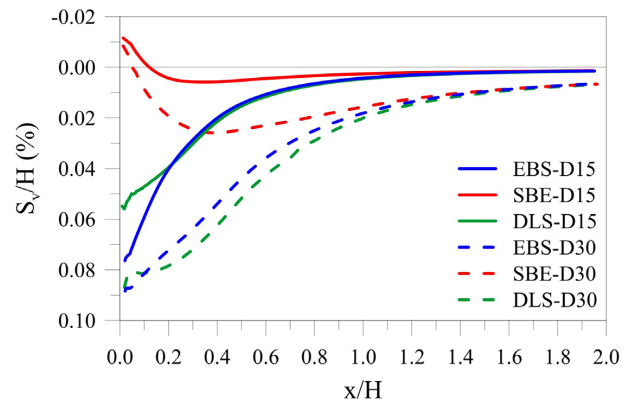


Figure 13. Settlements induced by different shaft methodologies.

method adopted. As expected, the smaller settlements – including slight heave near the wall – are predicted by the SBE methodology. The EBS and DLS techniques yield similar results, although EBS predicts higher settlements in the vicinity of the wall.

The lateral displacements of the wall are presented in Figure 14a and confirm that an increase in shaft diameter corresponds to greater displacements across all construction methodologies. In agreement with the settlement results, the smallest horizontal displacements occur with the SBE method, while the EBS and DLS techniques predict similar values for the smaller shaft. However, for the larger shaft, a different behavior between EBS and DLS is observed. In the upper part of the shaft, where the DLS method employs SBE, higher movements are observed, whereas in the lower part the opposite occurs, with EBS predicting greater horizontal

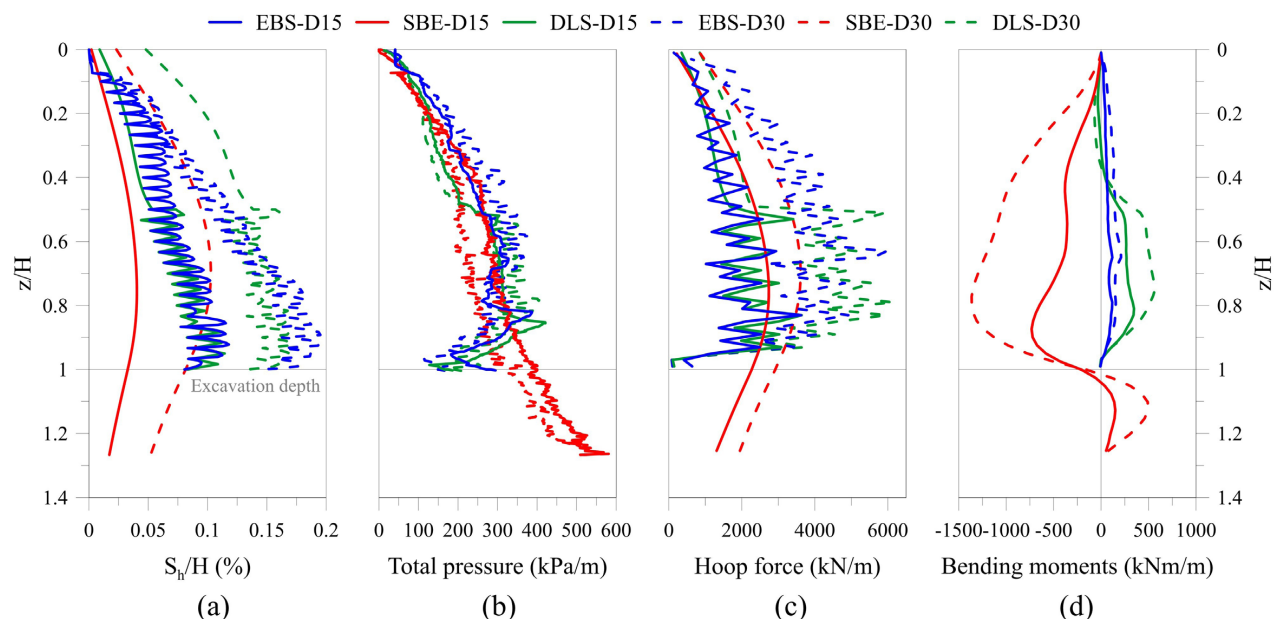


Figure 14. Lining results for different shaft methodologies: (a) horizontal displacement; (b) pressure; (c) hoop force; (d) bending moments.

displacements. The total pressure results in Figure 14b indicate that larger shafts experience lower pressures, with the differences being less pronounced, particularly in the SBE and DLS cases. All construction methods predict similar pressure distributions that increase with depth, though EBS presents slightly higher values throughout most of the shaft height, except near the base, where lower total pressures are observed due to soil decompression. The hoop forces exhibit a similar distribution with depth across all cases, with its maximum located at about 75% of the shaft height. The magnitude of the hoop forces increases considerably for the larger shaft, regardless of the construction methodology adopted (Figure 14c). Interestingly, for the smaller shaft, the hoop force predicted in the SBE method is greater than that observed in EBS and DLS, which yield identical results. In contrast, for the larger shaft, the opposite trend is observed, with SBE predicting smaller forces than EBS. The most significant differences in behavior are observed in the bending moments shown in Figure 14d, with the SBE analyses predicting considerably higher values. Moreover, the SBE cases indicate a curvature of the wall towards the interior of the excavation, while the opposite trend is observed in both the EBS and DLS methodologies. In agreement with the previous results, the larger shaft exhibits greater bending moments across all construction methods. However, it should be noted that in large shafts, the construction sequence of each excavation step is often divided into sections. As a result, arching develops not only in the vertical plane but also in the horizontal plane, contributing to the overall stability of the excavation.

The use of 2D axisymmetric analysis cannot capture this construction sequence, leading to more conservative predictions as observed in the analyses.

5. Conclusions

This study evaluates the performance of three distinct construction methodologies, EBS, SBE and DLS, through extensive numerical analysis, which is validated against real case studies. The validation performed provided the following insights regarding the numerical modelling of each methodology:

- Incorporating hardening behavior in the lining is particularly important for the ESB methodology, improving the reproduction of available monitoring data.
- The method proposed by Schwamb & Soga (2015) simulates adequately the settlements induced by the wall installation.
- Lining stiffness anisotropy, caused by discontinuities between diaphragm wall panels or bored piles, has a considerable impact on shaft settlements, with the selection of the adequate stiffness anisotropy ratio still requiring further research.

The parametric studies provided the following insights regarding how some of the main variables inherent to each methodology influence shaft behavior:

- EBS – increasing the excavation step height and lining thickness leads to higher settlements due

to the greater unsupported soil length and to the additional weight of the lining, respectively, which are not compensated by the increased stiffness.

- SBE – the thickness of the wall is crucial for controlling ground movements, whereas the embedded length of the wall had a minor impact on the shaft behavior in the analyzed case, although it can be important in scenarios where less competent materials or higher water pressures are found.
- DLS – increasing the SBE percentage modifies the shaft behavior, making it closer to an EBS shaft excavation, with the settlement trough shifting from a concave to a spandrel shape.

The comparative study between all techniques led to the following conclusions:

- SBE induces the smallest ground movements but produces the highest lining forces.
- EBS exhibits the opposite, with higher ground movements due to soil decompression but lower lining forces.
- DLS methodology presents an intermediate behavior, although closer to EBS.
- The increase of shaft diameter translates into an increase of ground movements and lining forces, regardless of the shaft construction methodology.

The findings of this study provide valuable guidance into optimizing shaft design and selecting the appropriate construction methods, contributing to safer, more efficient and sustainable underground infrastructure development.

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Declaration of interest

The authors have no conflicts of interest to declare. All co-authors have observed and affirmed the contents of the paper and there is no financial interest to report.

Authors' contributions

António M. G. Pedro: conceptualization, methodology, writing – original draft. David M. G. Taborda: conceptualization, data curation, methodology, software, writing – original draft. Lucas M. Repsold: investigation, validation, writing – review & editing. Jorge Almeida e Sousa: supervision, validation, writing – review & editing.

Data availability

The datasets generated and analyzed in the course of the current study are available from the corresponding author upon request.

Declaration of use of generative artificial intelligence

This work was prepared without the assistance of any generative artificial intelligence (GenAI) tools or services. All aspects of the manuscript were developed solely by the authors, who take full responsibility for the content of this publication.

List of symbols and abbreviations

c'	Effective cohesion
k_v	Vertical permeability
k_h	Horizontal permeability
p'_{ref}	Effective pressure of reference
s	Parameter controlling the effect of time on shotcrete stiffness
t	Lining thickness
t_{conc}	Age of sprayed concrete
y	Vertical coordinate on the model
y_{LC}	Coordinate of the top of the London Clay
x	Radial distance behind shaft wall
z	Depth
CGR	Channel Gate Road shaft
D	Shaft diameter
DLS	Dual-Lined Shafts
E	Young's modulus
E_{28}	Young's modulus of sprayed concrete at 28 days
E_z	Vertical stiffness
E_θ	Circumferential stiffness
EBS	Excavation Before Support
GWT	Ground Water Table
$G_{ref}, m_G, a, b, R_{g,min}, G_{min}$	Parameters for defining shear stiffness
H	Shaft depth
K_o	At-rest coefficient of earth pressure
$K_{ref}, m_K, r, s, R_{K,min}, K_{min}$	Parameters for defining bulk stiffness
KG	Kennington Green shaft
L	Embedded length of the lining
PCS	Pre-Cast Segments
S_v	Settlements
S_h	Horizontal displacements
SBE	Support Before Excavation
SCL	Sprayed Concrete Lining

WCH	Whitechapel Cambridge Heath shaft
WIP	Wished-in-place
γ	Unit weight
ν	Poisson's ratio
ϕ'	Angle of shearing resistance
ψ'	Angle of dilatancy

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Structural health monitoring and inspection of dams based on unmanned aerial vehicles photogrammetry with 3D model reconstruction

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Article

Keywords

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Dam
Monitoring
Safety
WebODM
Ground Control Points (GCP)
Finder

Abstract

Ensuring dam safety requires continuous monitoring to detect structural anomalies such as cracks, displacements, and seepage. This study explores aerial surveys using Unmanned Aerial Vehicles combined with open-source software to enhance dam monitoring capabilities. The methodology enables efficient inspections in difficult-access areas and supports early identification of potential failure indicators. High-resolution aerial imagery was processed to generate detailed three-dimensional representations including point clouds, orthophotos, and textured models. A novel Ground Control Points Finder tool was developed to automate identification and georeferencing, significantly improving spatial accuracy and survey efficiency. The study systematically presents fourteen failure modes detectable through aerial mapping, including piping, hydraulic fracturing, foundation instability, and freeboard loss, with their characteristic surface manifestations. The approach was validated using Lapa Dam as a case study through two aerial surveys conducted ten months apart. Analysis using open-source software successfully identified surface deformations, settlements, and vegetation changes, demonstrating the methodology's effectiveness in detecting structural anomalies associated with potential failure modes. The results confirm that integrating aerial surveys with open-source processing tools offers a low-cost solution to complement dam safety assessments. The approach improves early detection of various failure modes, reduces inspector exposure to hazardous conditions, and supports informed decisions in monitoring embankment dams through accessible technology.

1. Introduction

Dam safety is a multidisciplinary field involving various areas of study. According to Portuguese legislation, dam safety encompasses structural, hydraulic, operational, and environmental aspects (Portugal, 2018). Ensuring the effectiveness of water tightness, filtering, and drainage systems is crucial for preventing percolation issues in the body of embankment dams and foundation and surrounding areas. Proper management, commitment to safety regulations, and implementation of operation and maintenance programs are essential for operational safety. Environmental considerations focus on minimizing social, economic, and environmental impacts, prioritizing protection (Quintela et al., 2001).

From a performance perspective, dam safety depends on three fundamental criteria: controlled flow, stability, and deformation compatibility (Cruz, 1996). A dam is considered hydraulically and structurally safe when percolation occurs in a controlled manner, preserving the mechanical stability of the structure under operational stresses. Figure 1 shows an embankment dam with a central core, where the core, cut-off trench, and injection curtain act as flow barriers, while filters and drainage systems regulate water movement downstream in a controlled manner. Nonetheless, inadequate flow control, instability, or excessive deformations can lead to anomalies in the body or foundation. If not detected and addressed in time, these issues can evolve into failure modes, ultimately compromising the dam's integrity. In this context,

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visual inspections play a crucial role in identifying early-stage anomalies, such as cracks, settlements, and seepage, which might otherwise go unnoticed.

Recent technological advances have enhanced dam monitoring, particularly through Unmanned Aerial Vehicles (UAV) and laser scanning, which offer cost-effective solutions for structural assessment (Leite & Marcelino, 2020). The accuracy of UAV-collected data, however, depends on many factors, such as the quality and distribution of ground control points (GCP). These points are essential for correcting positional errors in three-dimensional models during post-processing. This paper presents an algorithm designed to improve the efficiency of GCP identification, optimizing data processing on the WebODM platform (Freire, 2022).

2. Benefits of using UAVs for visual inspection and monitoring of dams

The importance of visual inspection in the context of dam security is fundamental, and UAV can play a crucial role in developing and implementing innovative visual observation methodologies. By collecting data from UAVs, it is possible to generate a dam model from a point cloud and subsequently reconstruct its geometric representation. Point clouds result from processing georeferenced images captured during UAV flights. These three-dimensional models exhibit high geometric

precision and enable comprehensive modeling of the entire structure, including typically inaccessible perspectives, such as views from above and upstream.

Beyond capturing the entire geometry during flight surveys, these models promote more thorough and phased inspections over time. The data derived from the geometric model closely resembles that obtained through instrumental monitoring, offering a numerical representation stored in a database, devoid of subjectivity. This approach is not limited to the visible spectrum; alternative systems capturing data outside this range (thermal, infrared, or ultraviolet) can also be employed. While the current study focuses on visible spectrum data, the developed technique can be applied to any sensor, if the maps obtained refer to the same positions in the image.

Although traditional visual inspections remain a fundamental tool in dam safety monitoring, they can, in some cases, involve risks and limitations due to access difficulties and potential hazards - particularly during emergency situations. An example is the Pampulha dam collapse (Brazil, 20 April 1954), where an inadequate drainage system resulted in high gradients that led to piping (Guidicini et al., 2021). As demonstrated in Figure 2, the Brazilian authorities conducted a high-risk inspection a few hours prior to the rupture, which occurred approximately 48 hours after the anomaly was first detected.

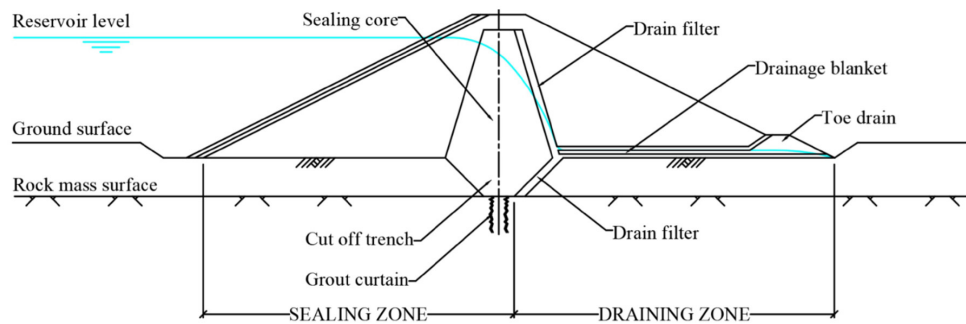


Figure 1. Representation of adequate internal flow in an embankment dam with a central core.



Figure 2. High-risk inspection conducted at the Pampulha Dam, Belo Horizonte (CMBH, 1954): (a) expert group; (b) rupture of the dam.

In this context, UAVs offer significant benefits. The ability of UAVs to reach hard-to-access areas efficiently and safely is well-documented, leading to a significant reduction in the exposure of workers to potential hazards. UAVs can capture detailed and precise images of the structures, allowing for the early identification of potential issues.

Figure 3 shows an example of a slope about 40 m high, where cracks and slips have been identified. This image was taken from a 3D model obtained from a point cloud generated by processing images taken by a drone. It was possible to identify and map the anomalies in approximately 40 minutes, without putting the inspectors at risk, with the advantage of obtaining an accurate description of the geometry and characteristics of the anomalies. UAVs can be equipped with high-resolution cameras and advanced sensors, including multispectral cameras, radar, LiDAR, infrared, and magnetic sensors. This enables comprehensive data collection across various spectrums, enhancing the detection of anomalies, such as deformations, settlements, cracks, seepages, and wet zones.

When aerial photogrammetry processes are conducted, UAVs produce orthophotos and spatial models, providing valuable information on the dam's current condition and allowing precise comparisons of anomaly evolution over time. Aerial photogrammetry is a well-established technique traditionally carried out using manned aircraft, where a sequence of overlapping images is captured along a predefined flight path. This methodology has been successfully adapted for use with UAVs, enabling greater flexibility and reducing operational costs, while maintaining the core photogrammetric principles. Images captured by digital cameras or even mobile phones can also be employed in the photogrammetric process to enhance the resolution of orthophotos in areas affected by anomalies, especially in localized or hard-to-access zones. However, the accuracy and resolution of the final products depend strongly on several parameters, including the camera sensor specifications, flight altitude, and image overlap. Lower altitudes allow finer Ground Sampling Distance (GSD), meaning that each pixel represents a smaller area on the ground. This approach enhances the efficiency of traditional visual inspections, allowing for a more comprehensive and precise analysis of the structural condition.

To establish a potential failure mode, it is essential to conduct a comprehensive analysis of the anomalies, emphasizing their correlation with monitoring, whenever such a link can be established. Integrating UAV-based techniques with traditional visual inspections is crucial for identifying the failure mode most consistent with the observed anomalies. Creating accurate three-dimensional models of the dam, and associated structures, provides numerous visualization and analysis possibilities, that would otherwise not be achievable.

3. Main failure modes mappable with UAVs

Most anomalies in embankment dams typically appear on the surface, abutments, and adjacent areas, and they may

be linked to potential failure modes. Accurately identifying the developing failure mode in an embankment dam is crucial, as early detection allows for effective mitigation measures to address the issue and prevent severe consequences. To categorize anomalies into a potential failure mode, they should be analyzed collectively and, whenever possible, associated with monitoring devices. Combining these findings with visual inspections is important to identify the failure mode most consistent with the observed anomalies.

Figures 4 to 17 schematically present anomalies related to fourteen failure modes manifesting on the surface, in the foundations, and/or in the dam abutments, which can be mapped using UAVs. These anomalies include cracks, slumping, seepage, settlements, uplifts, deformations, sinkholes, heaving, muddy water, wet zones, and others. It is essential to examine these together, to correctly assess the failure mode developing in the embankment dam.



Figure 3. Visual inspection of a high-risk area using a drone (image extracted from a textured 3D model generated from drone-captured images).

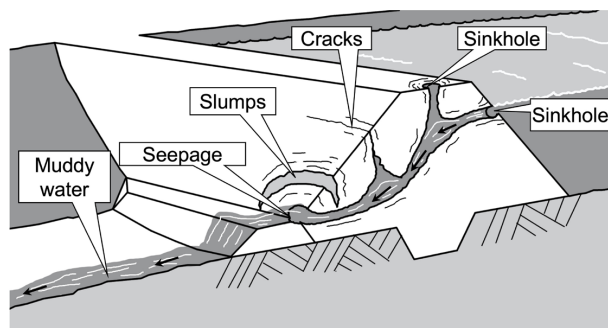


Figure 4. Piping in embankments. This can be identified through seepages with muddy water, slumps, and arch cracks, as well as the presence of sinkholes (Guidicini et al., 2021; USBR, 2019; GBC, 2020).

As shown in the previous figures, different failure modes can present signs of anomalies both on the surface of the structures, as well as in adjacent areas. These anomalies can be detected through conventional visual inspections and/or using UAVs for aerial dam mapping (Leite & Marcelino, 2020). The use of UAVs allows for monitoring the entire dam and adjacent areas

from perspectives not possible with traditional inspections and recording images with dimensional accuracy. By combining the expertise of dam inspectors with sophisticated monitoring technologies, it is possible to identify anomalies associated with the failure modes at an early stage, enabling the implementation of preventive measures to guarantee dam security.

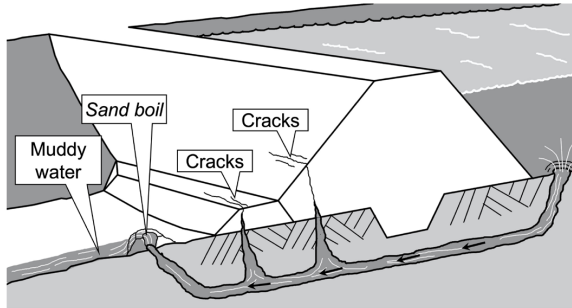


Figure 5. Piping in foundations. In foundations, the piping can manifest itself through downstream seepages with the presence of turbid water or the dragging of fines from the foundation, sand boils, and cracks on the embankment (FEMA, 2023).

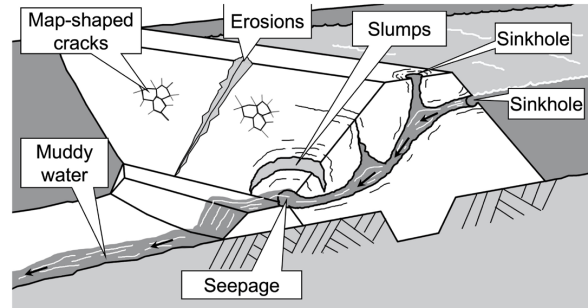


Figure 8. Piping in dispersive soils. Dispersive soils are susceptible to piping, which can be manifested by map-shaped cracks, erosions, as well as seepages, slides, sinkholes, and the presence of muddy water (USDA, 2006; Tosun & Kılıbıyık, 2006; Hardie et al., 2009; Cruz, 2008).

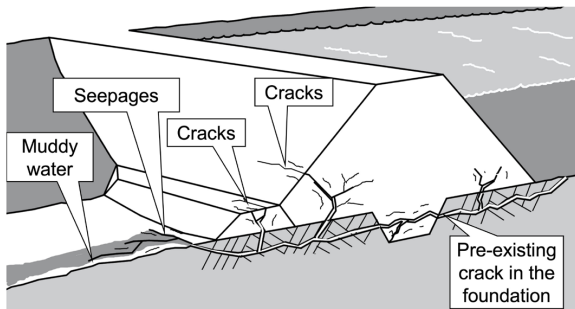


Figure 6. Piping into pre-existing fractures. Pre-existing fractures in the foundations can facilitate the occurrence of piping, which is initially manifested by cracks, seepages with muddy water, and/or fines from the foundation (Mejia, 2013).

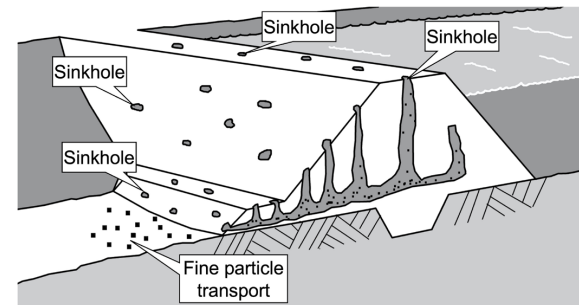


Figure 9. Piping by suffusion. Poorly graded soils, which allow for the migration of fines, can result in the suffusion process, which is typically evidenced by the formation of numerous sinkholes and the presence of fines in the downstream slope (Richards et al., 2015; USACE, 2022).

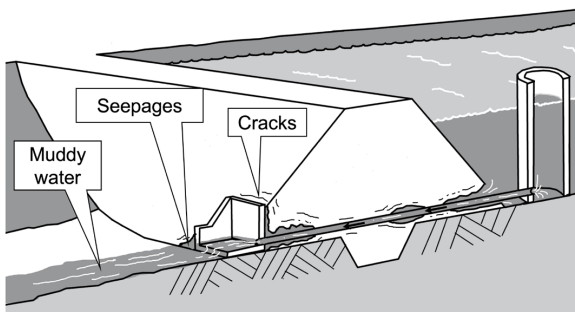


Figure 7. Piping on contact between soil and structure. Piping can occur at the interface between the soil and the structure, initially manifesting itself through cracks, seepages of muddy water, and with the presence of fines from the foundation in the zone downstream of the dam (Silveira, 2019).

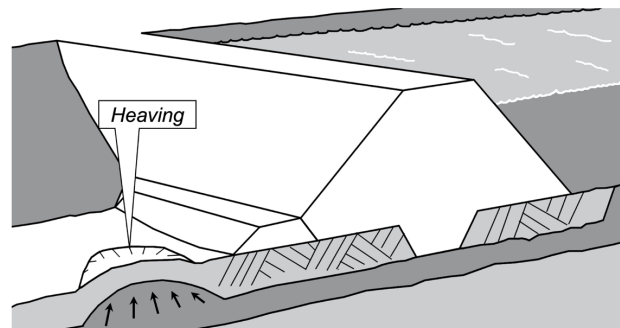


Figure 10. Heaving and Hydraulic lifting. The aerophotogrammetry process allows the comparative analysis of point clouds, facilitating the identification of anomalies, such as heaving, caused by excessive underpressure (Robbins et al., 2015; Bandeira et al., 2018).

4. Open-source software for processing and analyzing 3D models

Aerial surveys have become increasingly efficient and cost-effective with the use of UAVs. By applying aerophotogrammetry techniques, which utilize images

captured by UAVs, various spatial models can be generated, including Digital Surface Models (DSM), Digital Terrain Models (DTM), orthophotos, elevation maps, and contour lines. These models are valuable in the context of dam safety, as they enhance structural monitoring and anomaly detection.

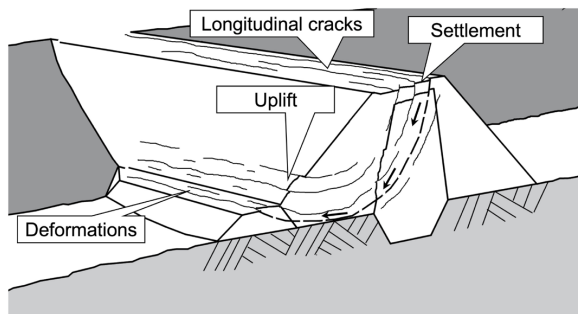


Figure 11. Excess pore pressure. When the loads applied to soils increase rapidly and the pore water pressures are not allowed to dissipate, a loss of shear strength may occur, thereby facilitating the occurrence of longitudinal cracking and settlement (Souza, 2017; Mello, 1982).

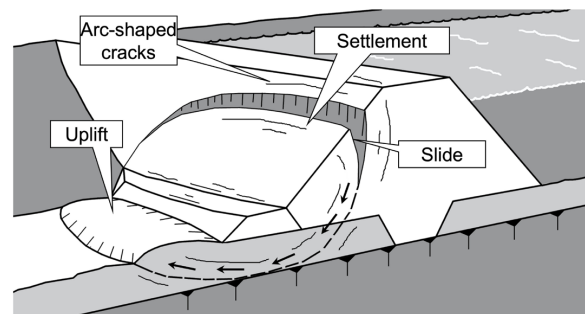


Figure 14. Foundation shear instability. In the presence of soft soils in the foundation with insufficient load-bearing capacity, there is a greater propensity for sliding, settlement, uplift, and arch cracks (Vu et al., 2023).

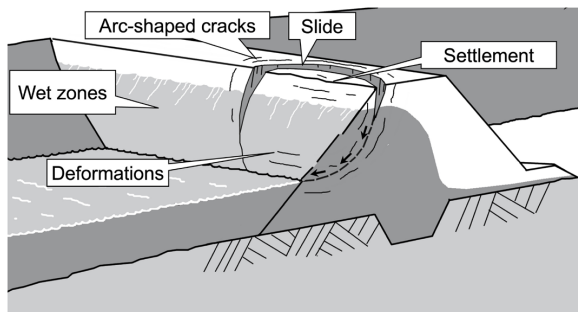


Figure 12. Rapid drawdown failure. The rapid drawdown of the reservoir can cause instability in the upstream slopes, increasing the probability of slides, settlements, and arch cracks (Narita, 2000; FERC, 1978).

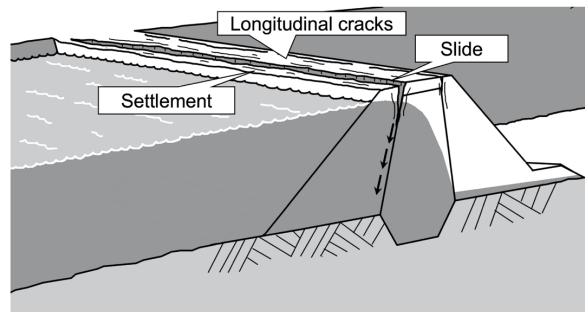


Figure 15. Freeboard loss failure. Rockfill dams or dams with collapsible material can experience excessive settlement due to wetting, especially during the first filling of the reservoir, resulting in longitudinal cracks, sliding, and deformations (Marcelino, 1996).

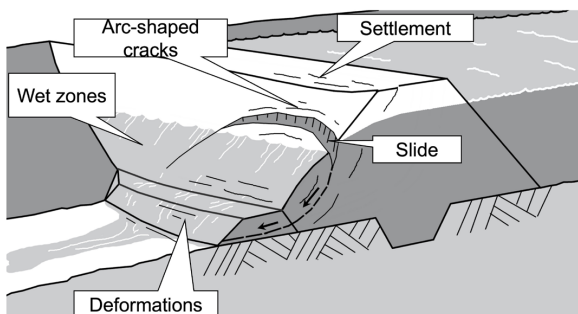


Figure 13. Faulty drainage system. Deficiencies in the internal drainage system have the potential to elevate the water level within the embankment, favouring the formation of wet zones, settlements, cracks in the arches, deformations, and slides (Mello, 1997).

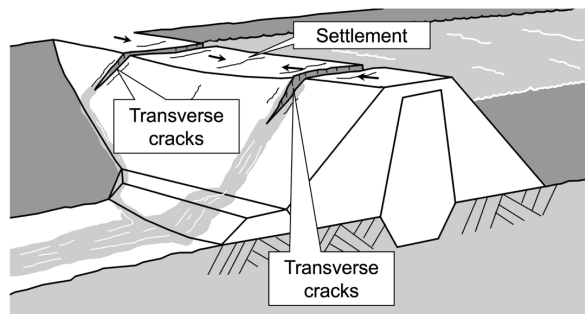


Figure 16. Freeboard loss due to cracks. Differential settlements in the foundation and embankment can cause transverse cracks, resulting in a loss of clearance and favouring the opening of a breach in the dam (He et al., 2021; Fell, 2015).

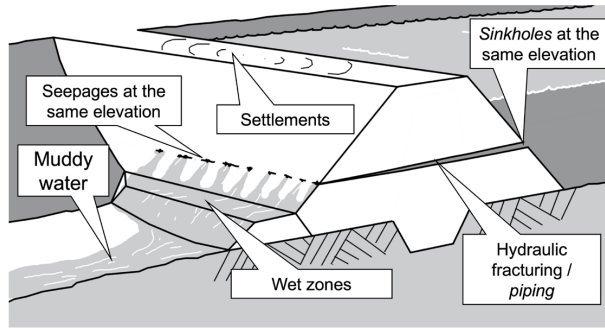


Figure 17. Hydraulic fracturing. Construction problems between joints or layers in embankments, as well as differential settlements, can lead to hydraulic fracturing and seepage at the same level, as well as sinkholes, and settlements (Sherard, 1986).

While UAV's acquisition costs are relatively low, commercial software for image processing can require significant investment. Open-source software (OSS) provides a viable alternative, offering high-quality results at a lower cost. Additionally, OSS fosters flexibility, independence from specific suppliers, and the potential for further application development.

Ensuring high geometric accuracy is essential to improve model quality and efficiency. Ground Control Points (GCPs) with known coordinates may be used during data collection to enhance spatial accuracy. These points, visible in multiple images, provide pixel-based coordinate references that align photographs with real-world coordinates. This process significantly improves the precision of object reconstruction algorithms by correcting image geometry.

In this study, UAV technologies were applied in dam observation using Open Drone Map (ODM) in its web interface version, WebODM (OpenDroneMap, 2025). This tool serves as a graphical user interface (GUI) for ODM, a collection of open-source tools that convert aerial images into georeferenced outputs, such as maps, point clouds, digital elevation models, and 3D textured models. The image processing workflow in ODM consists of six key operations: 1. Structure from motion (SfM); 2. Multi-view stereo (point cloud densification); 3. Meshing; 4. Texturing; 5. Georeferencing; and 6. Orthophoto generation.

By leveraging WebODM, this approach optimizes financial resources while maintaining full control over data analysis processes. Although OSS may lack some of the user-friendly interfaces of commercial software, it offers greater flexibility and facilitates result dissemination (Leite et al., 2024, 2025; Marcelino et al., 2024). The prioritization of OSS in aerial data processing provides an accessible and adaptable solution for UAV-based dam monitoring. Furthermore, it allows the sharing of data and developments without the constraints of licensing systems. It also helps to maintain data in the future due to open standards used in data files.

In addition, within the domain of point cloud comparison, the CloudCompare program emerges as an essential tool, assisting in the identification of surface changes across different surveys. Developed as an open-source project, CloudCompare is designed for visualizing, editing, and processing 3D data, with a focus on point cloud and mesh analysis. Its latest version, 2.14. alpha, Kharkiv, released on September 29, 2024, is governed by the GNU General Public License (GPL), allowing its use for various purposes, including commercial and educational applications (CloudCompare, 2024). CloudCompare supports several features, such as change detection, and can compare two different point clouds, proving useful in detecting settlements, heaving, and other deformations related to mass movements in dams and adjacent areas.

5. GCP finder tool

To enhance the precision and accuracy of aerial mapping conducted with UAVs, the use of Ground Control Points (GCPs) is essential. These points serve to correct errors and distortions that may arise during image processing and play a crucial role in verifying topographical accuracy. Additionally, they enable the assessment of the quality of the generated models.

Despite their importance, manually identifying and assigning reference coordinates to GCPs in hundreds of UAV-captured images is a labour-intensive and error-prone process. To address this challenge, LNEC, in collaboration with Nova University Lisbon, developed the GCP Finder tool (Freire, 2022). This software automates the detection and georeferencing of GCPs, significantly improving efficiency in aerial surveys.

Even when images are georeferenced, various factors – such as image resolution, pixel size, optical quality, and minor UAV flight variations – can introduce substantial errors in determining camera positions. GCPs help mitigate these errors by providing high-precision reference points.

An effective GCP should be easily identifiable from high altitudes, feature distinct shapes or colours that contrast with the surrounding environment, and include a well-defined reference point for GPS placement. Regardless of their format, GCPs should have a matte finish to prevent reflections that could interfere with automatic detection in the images (Kuze, 2015). Figure 18 shows an example of GCPs locatable with the GCP Finder software.

The process of manually compiling a list of GCP coordinates for each photograph is feasible for a small dataset, though it becomes highly inefficient for large-scale surveys. The GCP Finder tool automates this process, enabling the accurate and efficient extraction of GCP coordinates across extensive datasets. The software is freely available on GitHub (2025a) (console version) and GitHub (2025b) <https://github.com/octaviojardim/Web-Interface-GCP-Finder> (web interface version for server installations). These repositories include the source code, installation instructions, and practical application examples.

6. Case study

To illustrate the benefits of using UAVs in dam inspection, a case study is presented, based on surveys conducted as part of safety inspections by LNEC at the Lapa Dam. Lapa is an earthfill dam with a maximum height of 24 m and a crest elevation of 174.0 m (Figure 19). The normal water level (NWL) is 170.0 m, the maximum water level (MWL) is 171.92 m, and the minimum operating level (MOL) is 162.2 m. The reservoir at MWL has a relatively small volume (0.64 hm^3), with a flooded area of 9.5 ha. The upstream and downstream slopes have inclinations of 1:3.25 (V:H) and 1:2.5 (V:H), respectively. The downstream face features a berm, 2.5 m wide, located at 164.0 m elevation. To ensure high-resolution imaging in this study, aerial surveys were carried out using

a DJI Phantom 4 Pro drone equipped with a 20 MP camera. The precision of UAV-based photogrammetry is influenced by several factors, including the sensor specifications, flight altitude, and the distribution of ground control points (GCPs). Specifically, lower flight altitudes enable the generation of finer GSD, which represents the real-world size of one pixel in the image. For this case, flights were conducted at a height of approximately 15 meters, resulting in a GSD of about 0.4 cm/pixel at the dam crest. While this resolution ensures detailed surface modelling, the incorporation of GCPs remains essential to improve georeferencing accuracy and minimize distortions during the reconstruction of 3D models. These surveys were conducted on two separate occasions, on 07/20/2021 and 05/19/2022, aiming to identify possible settlements and other anomalies.



Figure 18. Example of ground control points identifiable with the GCP Finder.

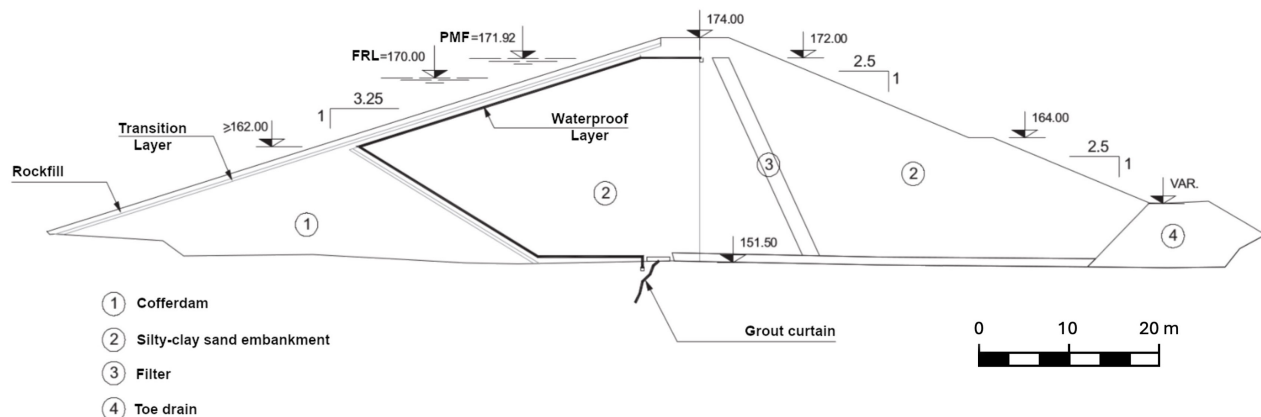


Figure 19. Cross-sectional profile of the Lapa Dam.



Figure 20. Textured model of the Lapa Dam (WebODM).

As shown in Figure 20, processing the images using WebODM software allows the generation of textured 3D models. It also enables the creation of DTM and DSM, such as the one illustrated in Figure 21, and contour lines associated with orthophotos (Figure 22). This results in a set of accurate data on the geometry and condition of the structure, highly useful for dam safety. These models faithfully represent the condition of all areas of the dam and surrounding regions.

As illustrated in Figure 22, this survey enabled the detection of an area of old settlement in the central region of the dam, indicating a loss of freeboard, mass movement, landslides, and cracks in the embankment and/or foundation. In detail, Figure 23 shows the cracks in the area where the settlements occurred.

Figures 24 and 25 show a comparison between two orthophotos generated in WebODM, from surveys performed over a period of approximately 10 months, whose differences in vegetation can be easily observed.

For a comparative analysis of deformations over a specific period, the use of CloudCompare software is recommended. This program allows for examining discrepancies between point clouds generated from images captured by UAVs, as shown in Figure 26. The areas highlighted in red represent differences greater than 1 m, indicating the removal of shrub vegetation during dam maintenance. The green and yellow areas denote changes of about 0.5 m, associated with the removal of ground vegetation. By analysing the colour patches and comparing them with the orthophotos (Figures 24 and 25), it is possible to distinguish between the deformations in vegetation and minor deformations in areas of interest, such as the crest and upstream rock protection. However, no significant deformations were observed in the

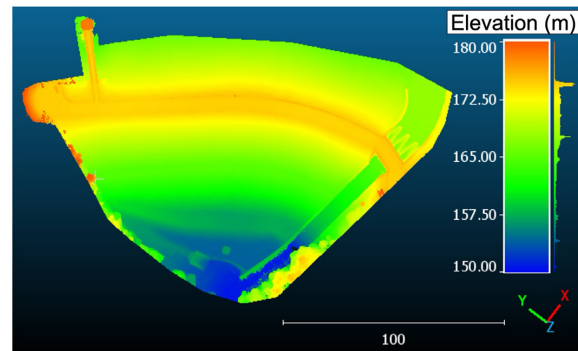


Figure 21. Digital Surface Model.

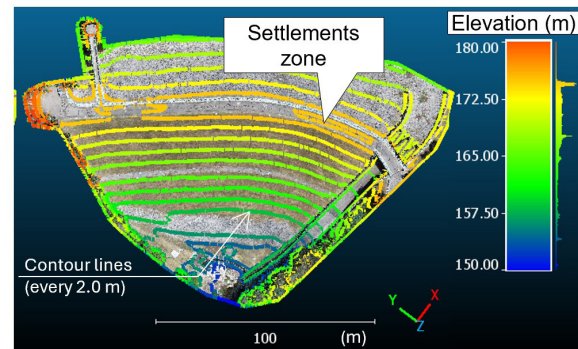


Figure 22. Contour lines (every 2.0 m).

body of the dam (blue patch), indicating that no substantial deformations occurred between the two surveys, as expected.

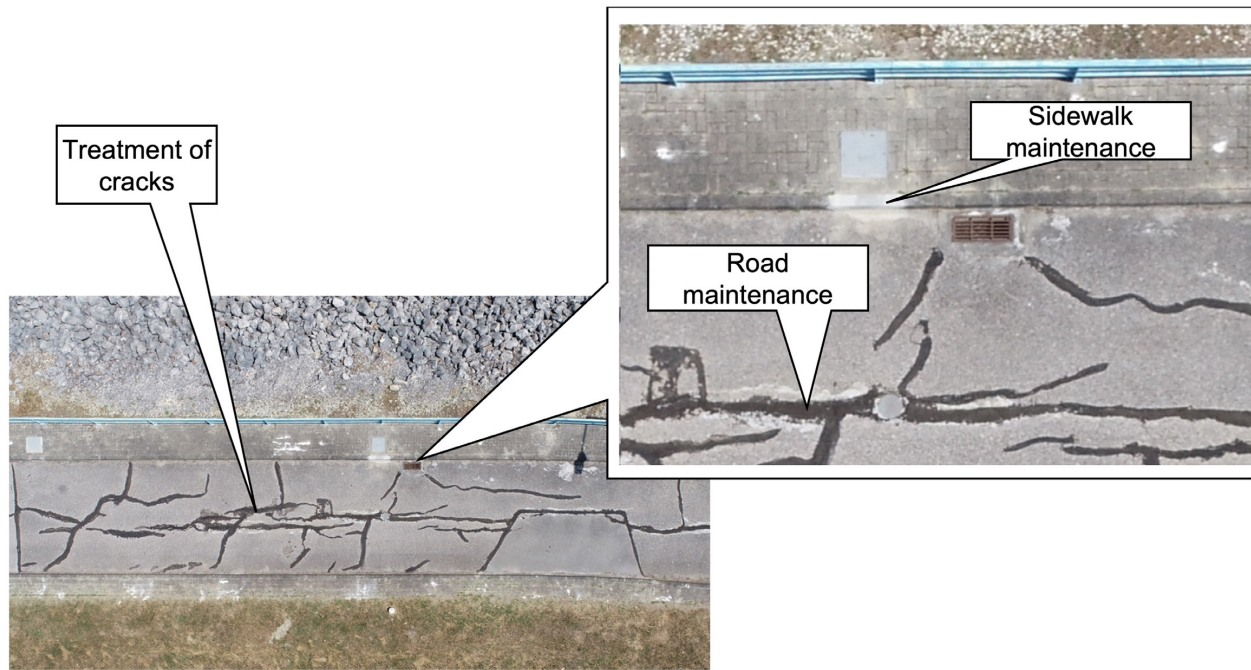


Figure 23. Treatment of cracks in the area where settlement has occurred.



Figure 24. Survey performed 07/20/21.



Figure 25. Survey performed on 05/19/22.

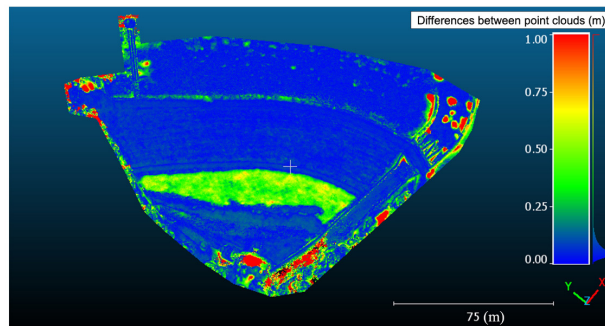


Figure 26. Point cloud comparison (CloudCompare).

7. Conclusions

The integration of UAV technology and open-source software has significantly enhanced dam monitoring, offering a cost-effective, precise, and comprehensive approach to anomaly detection. This study demonstrates how UAVs equipped with high-resolution sensors, coupled with advanced photogrammetric processing techniques, can improve structural assessments by generating detailed 3D models, orthophotos, and point clouds. The proposed GCP Finder tool further optimizes this process by automating the identification and georeferencing of Ground Control Points, improving accuracy and efficiency in data processing.

However, despite the clear advantages of using UAV photogrammetry for dam monitoring and safety, several limitations must be recognised. Firstly, weather conditions such as strong winds, rain and poor visibility can make

drone operation difficult and reduce image quality. Secondly, dense vegetation or steep, inaccessible terrain can obstruct the line of sight of images and jeopardise the generation of complete point clouds. Thirdly, although low-altitude flights and multiple GCPs allow for high-resolution surveys, these factors increase operational complexity and require a good command of operating this equipment. In addition, reliance on GCPs implies additional fieldwork, time and logistical planning.

A case study of the Lapa Dam highlights the effectiveness of UAV-based monitoring in identifying settlements, cracks, and vegetation changes over time. By utilizing Open Drone Map (ODM) and CloudCompare software, structural deformations and surface alterations were systematically analysed, reinforcing the value of UAV applications in dam safety assessments. This approach not only enhances the ability to detect potential failure modes at an early stage, though it also reduces human exposure to hazardous environments, thereby improving overall safety. Moreover, UAVs facilitate periodic inspections from multiple perspectives, ensuring a more thorough and reliable assessment compared to traditional methods. The integration of UAV-based techniques with conventional visual inspections enables a more data-driven and proactive approach to dam safety management. By leveraging open-source tools like WebODM, this methodology is accessible, scalable, and adaptable to various dam monitoring applications worldwide.

Ultimately, the findings of this study underscore the transformative potential of UAV technology in dam monitoring. The combination of high-resolution imaging, automated georeferencing, and robust data analysis tools provides engineers and decision-makers with valuable insights,

promoting early anomaly detection and ensuring the structural integrity of embankment dams. Future advancements in UAV capabilities and artificial intelligence-driven data processing are expected to further refine and expand the applications of this technology, making dam safety assessments more efficient, reliable, and sustainable.

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Declaration of interest

The authors have no conflicts of interest to declare. All co-authors have reviewed and approved the contents of the paper, and there are no financial interests that could have appeared to influence the work reported in this study.

Authors' contributions

Daniel Teixeira Leite: conceptualization, data curation, visualization, writing – original draft. João Marcelino: supervision, validation – original draft. Nuno Manuel da Costa Guerra: supervision, validation – review & editing. João Miguel Gomes Pires Manso: supervision, writing – review & editing. Nuno Miguel Cavalheiro Marques: supervision, validation – review & editing.

Data availability

The datasets generated analysed in the course of the current study are available from the corresponding author upon request.

Declaration of use of generative artificial intelligence

This work was prepared with the assistance of generative artificial intelligence (GenAI) tools, specifically OpenAI ChatGPT, only with the aim of improving the clarity and flow of the English language in the manuscript, assisting in the refinement of figure captions, and suggesting concise phrasing for technical sections while ensuring consistency with journal style. The entire process of using this tool was supervised, reviewed and when necessary edited by the authors. The authors assume full responsibility for the content of the publication that involved the aid of GenAI.

List of symbols and abbreviations

DSM	Digital Surface Models
DTM	Digital Terrain Models
GCP	Ground Control Point
GNU	GNU's Not Unix
GPL	General Public License
GUI	Graphical User Interface
LiDAR	Light Detection and Ranging
MOL	Minimum Operating Level
MWL	Maximum Water Level
NWL	Normal Water Level
ODM	Open Drone Map
OSS	Open-source Software
SfM	Structure from Motion
UAV	Unmanned Aerial Vehicle

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Large-scale testing of free-ended piles in unsaturated expansive clays under cyclic and monotonic lateral loading

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Article

Keywords

Pile foundations
Large-scale physical modelling
Field testing
Unsaturated expansive clays
Cyclic lateral loading

Abstract

This article presents an investigation into the influence of swell on the behaviour of large-scale free-ended piles installed in an unsaturated expansive clay deposit when subjected to lateral loading. A ‘dry’ pile was founded in a profile kept at natural water content conditions, and a ‘wet’ pile was founded in a profile which was flooded for six months to allow swelling of the clay prior to testing. Load cycles under two different load magnitudes were applied to the piles, followed by monotonic loading to failure. The applied load, pile head displacement and bending strain distributions with depth (determined using fibre Bragg gratings) were recorded during these tests. The behaviour under 113 kN load cycles was similar for the two piles, with the ‘wet’ pile exhibiting marginally stiffer response. This was attributed to a swell-induced increase in lateral stress acting against the pile shaft. Under load cycles of 145 kN and during the monotonic test, the ‘wet’ pile exhibited a substantially softer response and significantly lower ultimate capacity than that of the ‘dry’ pile. This was attributed to the effects of swell-induced softening and the reduced yield stresses in the swelled clay. These phenomena were interpreted with reference to a site investigation programme including seismic continuous surface wave tests and standard penetration tests, as well as some laboratory testing. Effects of local yielding of the bonded material at fissure interfaces upon pile ratcheting during cyclic loading have also been discussed.

1. Introduction

Expansive clays pose significant challenges for engineers worldwide. These clays are typically characterised by their high plasticity, causing them to significantly expand during wetting and shrink and crack during drying. This differential movement can have substantial economic consequences for overlying or adjacent structures (Jones & Holtz, 1973). The volumetric behaviour of such clays becomes a particular challenge in the context of wind turbines, where strict tolerances for differential foundation movements are typically prescribed.

Additionally, in the case of piled foundations, prior research has indicated that the aforementioned volume changes are accompanied by variations in shaft capacity (Blight, 1984; Elsharief et al., 2007; Gaspar et al., 2024). To address these challenges, the WindAfrica research project was initiated with the goal of developing practical guidelines for designing onshore wind turbine (piled) foundations in expansive soil conditions.

Previous publications on the WindAfrica project have presented (i) the hydromechanical behaviour of expansive clays (Gaspar et al., 2022; Murison et al., 2023), (ii) centrifuge modelling of piled foundations in expansive clays (Gaspar et al., 2024), and (iii) the response to soil swell of a large-scale instrumented pile socketed into bedrock (da Silva Burke et al., 2022). In the current study, the behaviour of large-scale free-ended piles under lateral loading is presented.

2. Background

Expansive soils are known to be a severe geohazard due to the distress imposed upon infrastructure by seasonal swelling and shrinkage. A number of construction techniques may be employed to mitigate these damages. Treatment or replacement of the expansive strata may be a feasible option, but is not recommended for depths beyond 2 m (Byrne et al., 2019). Another technique that may be employed to avoid excessive heaving of a foundation is pre-wetting

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the expansive profile and allowing swell to occur prior to construction, which is feasible up to depths of about 10 m (Weston, 1980; Byrne et al., 2019). If this method is implemented successfully and moisture loss of the layers is limited to avoid the effects of shrinkage, it is crucial to consider the influence of wetting the soil on the strength and resistance to deformation of the foundation system. Finally, the system may be isolated from the expansive layer using underreamed/anchored piles, as first suggested by Jennings & Henkel (1949). By providing anchorage at the base of the pile and a gap between the ground surface and the underside of a pile cap, this approach aims to provide a ‘suspended’ foundation. Although this is the most costly technique, the superstructure can be almost completely isolated from ground movements if the piles are sleeved (Fleming et al., 2009). If this technique is employed, thorough characterisation of the expansive soil is still required. Blight (1984) and Meintjes (1991) reported case studies where surface heave substantially exceeded what was predicted, causing the gap to swell shut and inducing heave-related damage to the superstructure. In cases where the bedrock or stable stratum is deep, this method might not be feasible. Expansive profiles extending to depths in the order of 30 m have been reported near Kimberley in South Africa (Byrne et al., 2019), and profiles in Sudan are commonly found to be deeper than 10 m (Elsharief, 2012). Under such conditions, founding the structure within the expansive strata may be the best option. In such cases, the use of piled foundations may be necessary to mitigate the effects of swell/shrink behaviour and may be particularly useful in cases where lateral loading is anticipated.

The design of pile foundations in expansive clays is a complex task due to the changes in lateral stress acting on the pile brought about by seasonal swelling and shrinkage of the clay, as well as the changes in resistance to lateral loading brought about by fluctuations in soil water content. The influence of these changes in moisture regime on the capacity of piles has been investigated in a handful of studies. Blight (1984) investigated the influence of wetting on shaft pull-out capacity for full-scale short piles. The shaft capacity increased by approximately 66% due to flooding of the profile for a 3 to 4-week period. Elsharief et al. (2007) reported a decrease in capacity of approximately 60% due to wetting for full-scale vertically loaded piles. In this case, the flooding had been maintained for two months. A study by Gaspar et al. (2024) shed light on these seemingly contradictory findings. In their study, centrifuge modelling was used to investigate the pull-out capacity of short piles, as well as the variation of lateral stresses acting on a long, wished-in-place pile instrumented with lateral load cells along its length. It was found that the lateral stresses acting on the pile increase initially during the swell process, and then reduce past initial ‘pre-swell’ values as the swelling process continues. It was thus suggested that an increase in capacity is exhibited early on into the swell process (or at low degrees of swell) due to the increase in lateral stresses on the pile, which was the mechanism observed by

Blight (1984). At later stages of swell, a reduction in capacity is caused due to the effects of swell-induced softening, which was likely the mechanism observed by Elsharief et al. (2007). An effect of overburden stress upon the post-swelling capacity was also noted – the reduction in capacity at high degrees of swell was not prominent at greater depths where the clay swell (and thus swell-induced softening) was suppressed. Gaspar et al. (2024) thus concluded that the influence of swell on shaft capacity depends on the degree of swell, i.e. whether the increased lateral stress or the swell-induced softening is the dominant mechanism at the given stage of swell, as well as the given overburden stress.

The current study investigates the performance during lateral loading, under full-scale conditions.

3. Site investigation and in-situ testing

The large-scale testing described in this study took place adjacent to an existing bentonite mine near Vrededorf, in the Free State Province of South Africa. A simplified borehole log, determined during site establishment, is presented in Figure 1a. Particle size distributions and soil classifications for samples taken from each of the distinct layers within the profile (at depths of 1.0, 3.0, and 5.0 m) are presented in Figure 1b and Table 1 respectively. X-ray diffraction (XRD) analyses confirmed that the dominant clay mineral constituent of each of the clays was smectite. This, along with the large plasticity indices and the highly fissured in-situ macrofabric, gives an indication of the high expansive potential of the profile.

Tests were conducted at two separate areas on the site. The first set of pile tests was conducted on a ‘dry’ area, where the soil was kept at natural water content conditions. A second set of tests (referred to as the ‘wet’ tests) was conducted on a section of the site which was flooded after pile installation for a period of six months. The flooding was facilitated using infiltration wells installed to a depth of 6 m. This allowed the intact masses within the fissured profile to swell prior to the pile testing. The two sections were sufficiently far apart (in excess of 50 m) to ensure that flooding of the ‘wet’ site did not influence the ‘dry’ site.

In-situ site characterisation tests were carried out at various stages throughout the project. The timeline of this testing, relative to the flooding period for the ‘wet’ site and the large-scale pile tests, has been summarised in Figure 2. Total rainfall over each complete wet and dry season within the monitoring period, which was recorded using a rain gauge installed at the site, has been included. Initial standard penetration tests (SPTs) were carried out at the eventual ‘dry’ and ‘wet’ site locations during site establishment, and no water table was observed at either location during this time. Approximately two months after site establishment, flooding of the ‘wet’ site commenced and continued for six months. Five weeks after the flooding period had been completed, the lateral pile loading tests commenced.

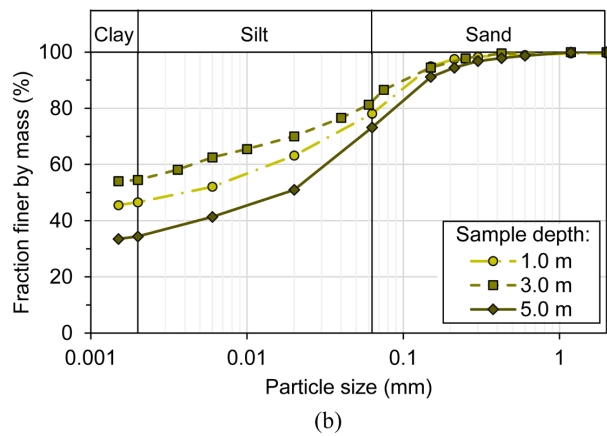
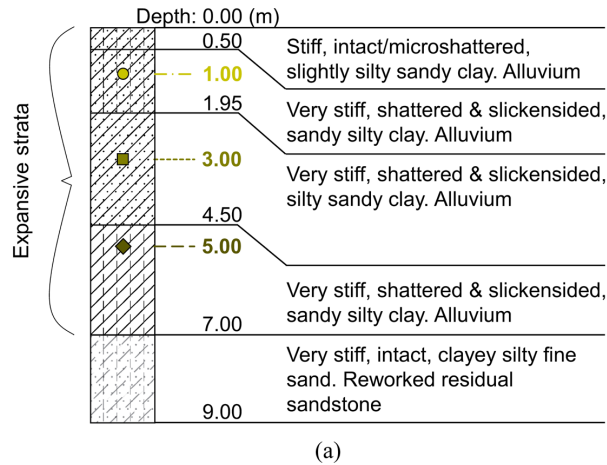


Figure 1. (a) Simplified soil profile from borehole logs; (b) particle size distributions at various sampling depths determined according to BS 1377-2 (BS, 1990).

This testing took place over a month-long period, during which the first sets of seismic continuous surface wave (CSW) tests were conducted as well. As a result of the flooding, the water table at the ‘wet’ site was observed to be very near the ground surface throughout the pile testing period. Throughout the subsequent 8-9 months, the upper strata of the ‘wet’ site profile partially dried out and the water table naturally subsided. At this stage, the second sets of SPT and CSW tests were conducted within two weeks of one another. The water table was observed at a depth of approximately 4 m during profiling of the ‘wet’ site, whilst the water table at the ‘dry’ site remained deep.

Table 1. Soil properties and classifications to BS 1377-2 (BS, 1990).

Property	Sample depth		
	1.0 m	3.0 m	5.0 m
In-situ water content, w (%) ^a	20.4	21.2	20.2
In-situ void ratio, e^a	0.90	0.86	0.83
Specific gravity, G_s	2.662	2.692	2.669
Liquid limit, w_L (%)	66	109	72
Plasticity index, I_P (%)	47	82	50
Clay fraction by mass (%)	46	54	34
Classification (BS 1377-2)	CH	CE	CV
Smectite/illite-smectite content from XRD (%)	49	52	46
Activity	1.02	1.52	1.39
Potential expansiveness class	Very high	Very high	Very high

(after Skempton, 1953)

(after van der Merwe, 1975)

^aAt end of dry season.

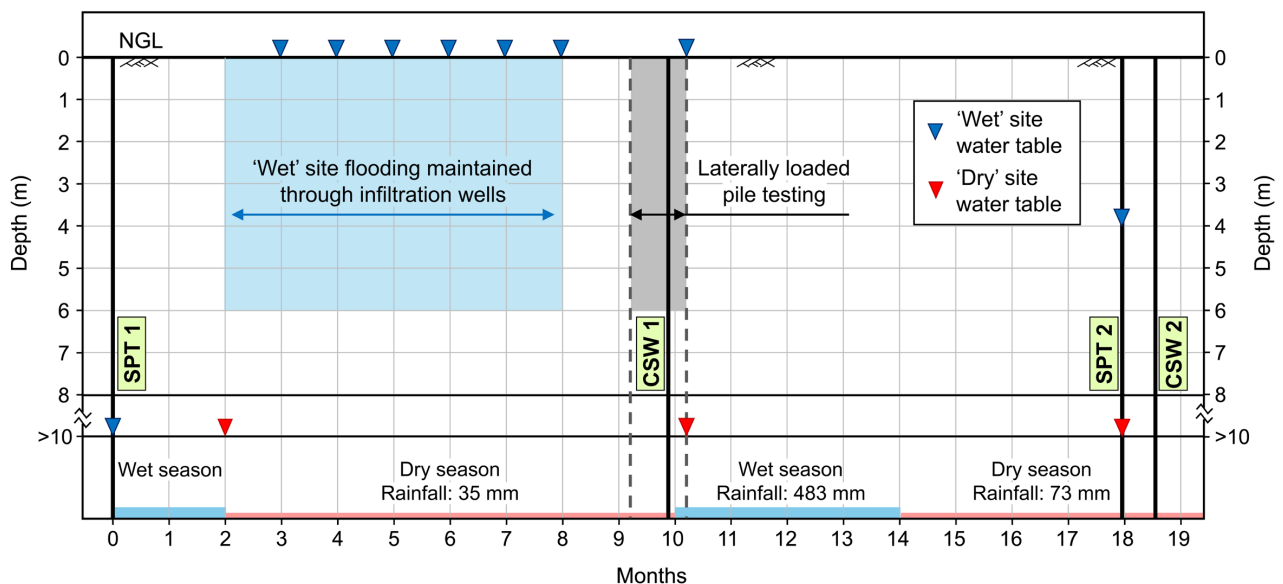


Figure 2. Timeline for flooding of the ‘wet’ site, in-situ testing and pile testing, along with respective water table positions and seasonal rainfall (after Murison et al., 2024).

The SPT blow counts were converted to equivalent N_{60} values (i.e. corresponding to the standard energy ratio of 60%), and are presented in Figures 3a and 3b. The profiles obtained during site establishment (i.e. prior to alteration of the water content of the ‘wet’ area) gave an indication of the uniformity of the site, and are given in Figure 3a. The profiles were satisfactorily similar for what would become the ‘wet’ and ‘dry’ sites. The second set of tests – conducted eight months after pile testing and after subsidence of the ‘wet’ site water table, given in Figure 3b – showed a significantly stiffer/stronger response at the ‘dry’ site, and a significantly softer/weaker response (at greater depth) at the ‘wet’ site. The initial profiles for each site are depicted using the broken lines in Figure 3b for reference. The changes in strength can be attributed to changes in suction occurring within the intact clay masses. As an example, gravimetric water contents for samples taken from a depth of 3 m at the

site have varied naturally between approximately 27% after the wet season and 21% after the dry season. Soil-water retention curves for material from this depth showed that this variation would cause an increase in total suction from approximately 1.5 MPa to 6 MPa (Murison et al., 2023), which would induce a significant increase in shear strength and thus blow count. The first SPT tests were conducted in the middle of a wet season, and the second towards the end of a dry season. Such variations in water content would thus be expected, which explains the increase in blow count at the ‘dry’ site. The converse is also true: samples taken from the ‘wet’ site during the second set of SPT tests were approximately fully saturated at depths below 3.0 m, and a water table was established at 4.0 m. Suctions would thus be significantly lower at these depths, which would explain the reduction in blow count in the ‘wet’ site for depths below the water table.

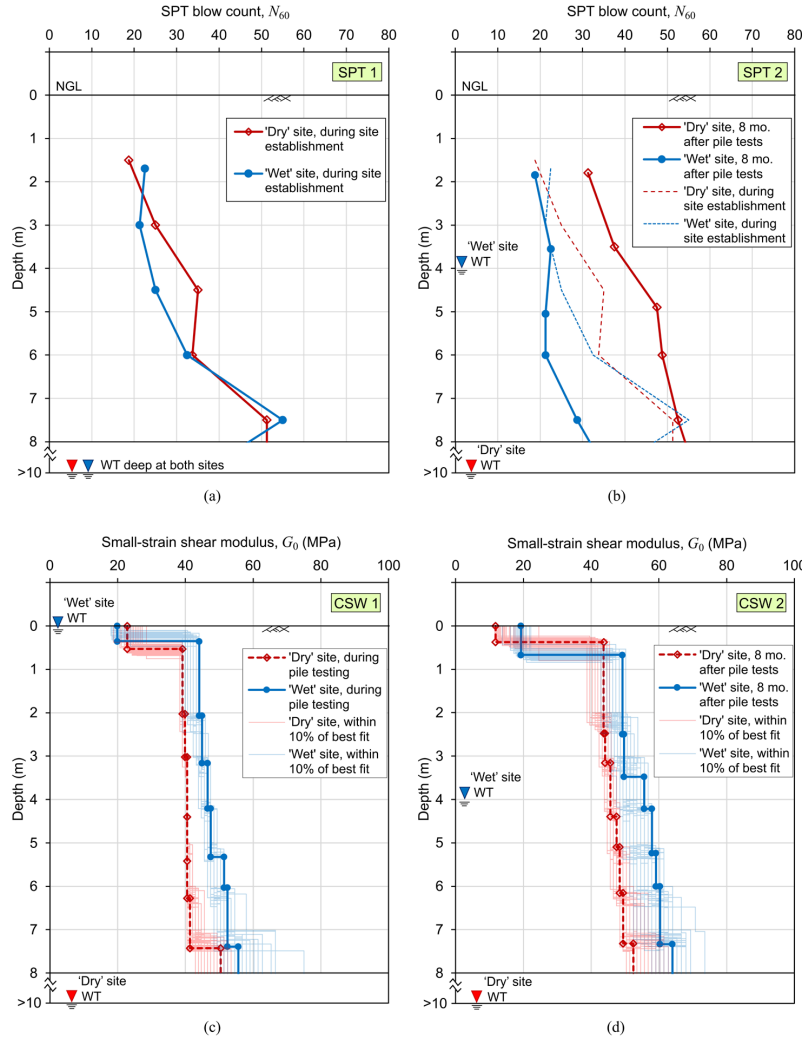


Figure 3. In-situ test results (after Murison et al., 2024): standard penetration test blow counts with depth for (a) SPT 1, conducted during site establishment, and (b) SPT 2, conducted 8 months after pile testing; small-strain shear modulus profiles with depth from seismic continuous surface wave tests for (c) CSW 1, conducted during pile testing, and (d) CSW 2, conducted 8 months after pile testing.

Detailed descriptions of the test setup and inversion algorithm used for the CSW testing and analyses were provided by Murison et al. (2024). The small-strain shear moduli (G_0) with depth resulting from the CSW testing and inversion are given in Figures 3c and 3d. All theoretical profiles trialled in the inversion search algorithm which contained a theoretical dispersion curve within 10% of the best-fit have also been reported in Figure 3. This gives an indication of the confidence level of the final profiles. At both instances in time and for both water table positions, the G_0 profiles for the two sites were similar. As a result, any engineering decisions for either site made on the basis of these measurements would not differ (this is supported by the overlap of the 10% misfits).

However, the ‘wet’ site profile was marginally stiffer in both cases. This observation is seemingly counterintuitive, as one would expect the stiffness of the ‘wet’ site profile to reduce during wetting due to the dissipation of suctions in the order of several megapascals, as discussed previously for the SPT results. The similarities may be explained by closer consideration of the fissured macrofabric of the clay profile and the strain range over which the measurement of shear moduli occurs during seismic tests. Matthews et al. (2000) reported significantly lower in-situ shear moduli for fissured hard soil and weak rocks using seismic techniques when compared to moduli measured in triaxial tests with local strain instrumentation. The very small strains imposed during CSW testing do not sufficiently deform the soil elements (within the intact masses) to close the fissures, and the measured shear wave velocity is thus dominated by the joint infill material rather than the intact material, which is tested in laboratory element tests. Regardless of whether or not wetting may have influenced the behaviour of intact material, if the fissures had not swelled closed

and the fissure infill characteristics were not significantly changed, the measured small-strain stiffness would be similar. Additionally, if some of the fissures were to swell closed, the intact clay may start dominating the measured stiffness behaviour, and increases in the measured shear wave velocity (and thus G_0) would be expected.

4. Pile testing layout

The test layout was identical for both the ‘dry’ and ‘wet’ sites. The layout for the free-ended piles considered in this study is illustrated in Figure 4a. The setup consisted of two 600 mm diameter reinforced-concrete bored piles, at a 2 m centre-to-centre (c/c) spacing, embedded 6 m into the expansive clay profile.

Cyclic lateral loading was applied to the pile heads under load control using a hydraulic jack. The point of load application was 500 mm above the natural ground level (NGL). Lateral displacements of the piles during loading were measured using linear variable differential transformers (LVDTs) with a range of 50 mm, and the load applied through the hydraulic jack was recorded using a load cell with a 500 kN full-scale capacity. Fibre Bragg gratings (FBGs) were attached to the steel reinforcing to measure bending strain distributions along the length of the piles throughout testing. Detailed descriptions of the performance of these measurements relative to vibrating wire strain gauges – as well as the calculation procedure to obtain strains from the wavelengths recorded by the FBGs – were provided by Murison et al. (2022), who found that the tensile strain measurements using the different sensors were satisfactorily similar. Due to unforeseen circumstances, it was not possible to log the FBGs on the compression side of the piles for some of the tests considered in this study. Only tensile strain distributions (rather than bending moment distributions) have thus been

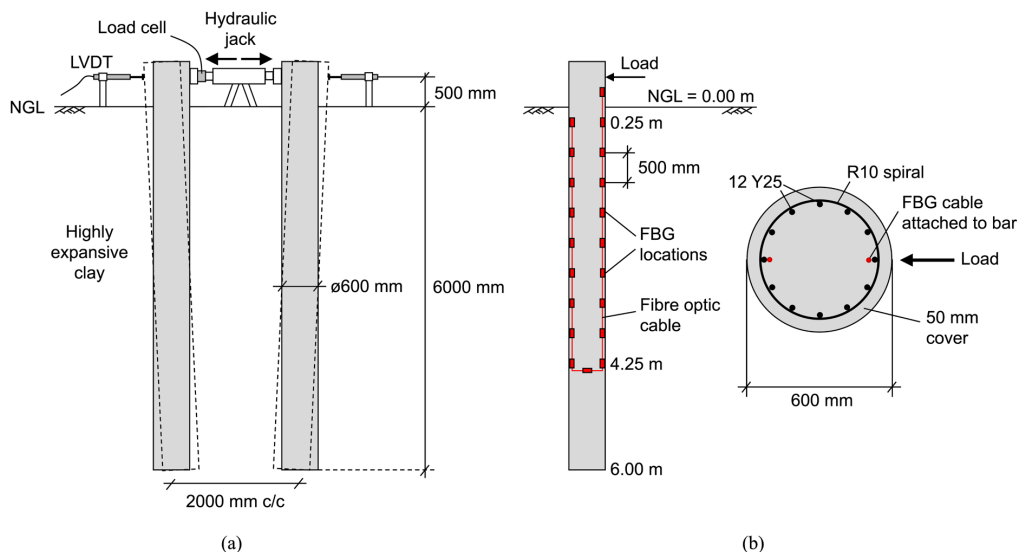


Figure 4. Experimental setup and instrumentation for lateral pile loading tests: (a) loading setup; (b) strain gauge layout and section details.

considered regarding bending behaviour. The positions of FBG sensors within the piles, as well as the reinforcement layout in the concrete section, are illustrated in Figure 4b. Note that the depiction of the fibre optic sensors is not to scale. The outer casing of the fibre optic cable was 1 mm in diameter, and each individual Bragg grating was 8 mm in length. Concrete testing was conducted on specimens cast on site and water-cured for 28 days. The properties of the concrete and steel reinforcement are summarised in Table 2.

The pile relative stiffness (K_R) was evaluated on the basis of Equation 1 (Poulos, 1982):

$$K_R = \frac{(EI)_{\text{pile}}}{E_{\text{soil}} L^4} \quad \begin{array}{l} K_R > 0.2080 - \text{rigid pile} \\ K_R < 0.0025 - \text{slender pile} \end{array} \quad (1)$$

The Young's modulus of the soil (E_{soil}) at the pile base ($L = 6$ m) was estimated from the small-strain shear modulus determined in the CSW testing ($E_{\text{soil}} = 2(1 + \nu)G_0$). Poisson's ratios of $\nu = 0.25$ and $\nu = 0.5$ were assumed for hypothetical drained and undrained loading respectively. As illustrated in Figure 3, the small-strain shear modulus at a depth of 6 m varied between approximately $G_0 = 40$ MPa and $G_0 = 60$ MPa. For drained loading, E_{soil} would thus vary between 100 and 150 MPa, resulting in a K_R range between 0.0011 and 0.0016. For undrained loading, E_{soil} would vary between 120 and 180 MPa, and K_R between 0.0009 and 0.0013. It is clear that regardless of the rate of loading imposed, the piles would be classified as slender according to the criterion set out in Equation 1.

5. Pile test results

The laterally loaded pile behaviour was evaluated based on the stiffness of the foundation system (i.e. lateral resistance), progressive displacement or "ratcheting" with increasing cycle number, and the bending strains experienced along the length of the pile at the peak of each cycle. The secant stiffness for each load–displacement cycle was calculated as illustrated in Figure 5, i.e. cycle stiffness was simply defined as the slope of a line connecting the trough and peak of a single load–displacement cycle. An initial set of 500 cycles under load control with a trough-to-peak load difference of approximately 100 kN and a maximum load of 113 kN was applied to the piles in the 'wet' and 'dry' sites. Thereafter, the maximum load magnitude was increased to approximately 145 kN for the 'wet' test for roughly 400 cycles. The pile considered in this study for the 'dry' test was not subjected to increased loading, but the first 400 cycles with a maximum load of 175 kN for an identical pile with identical loading history at the 'dry' site have been considered for comparison.

Thereafter, the 'wet' and 'dry' piles considered initially were loaded monotonically to failure.

5.1 Initial cyclic loading

The maximum load for the initial load cycles was approximately 113 kN for both tests. The applied lateral load versus displacement for piles founded in clays under natural water content and swelled states are shown in Figures 6a and 6b respectively. The cycle stiffness versus cycle number is plotted for both piles in Figure 6c. Note that some portions of noisy data were caused by strong gusts of wind, which interfered with the LVDT measurements.

The 'dry' pile exhibited an accumulation of progressive displacement of approximately 0.5 mm over the 500 cycles. In addition, the cycle stiffness gradually increased with increasing cycle number, with an increase of approximately 14% from the start to end of loading. Given that the 'dry' test took place in a desiccated, highly fissured profile, this behaviour was attributed to local yielding of material along the fissure contacts. It is hypothesised that as the material at these contacts yielded, the displacement of the pile head

Table 2. Reinforced concrete material and section properties.

Property	Value	Test and/or reference
Steel Young's modulus (GPa)	200	Assumed for high-yield steel (BS, 2004)
Steel yield strength (MPa)	450	Assumed for high-yield steel (SABS, 2011)
Concrete Young's modulus (GPa)	27.8	E-value tests on 167 mm × ø150 mm cylinders (BS, 2021)
Concrete compressive cube strength (MPa)	34.9	Compression tests on 100 mm cubes (BS, 2019)
Concrete tensile strength (MPa)	2.0	Indirect tensile strength tests on 150 mm × ø150 mm split cylinders (BS, 2009)
Pile bending stiffness, $(EI)_{\text{pile}}$ (kNm)	$207.6 \cdot 10^3$	Section analysis, assuming uncracked pile

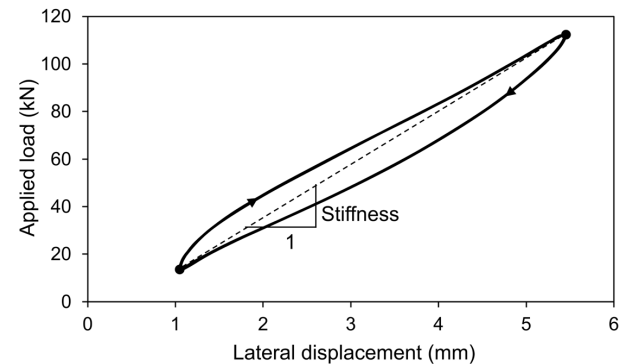


Figure 5. Definition of cycle stiffness for analyses.

progressively increased. However, as the process continued, the stiffer intact masses of clay would be pushed against each other at the fissure interfaces. As the behaviour of the system became increasingly dominated by the stiffness of the intact masses, the system cycle stiffness increased, and eventually tended to plateau around 500 cycles.

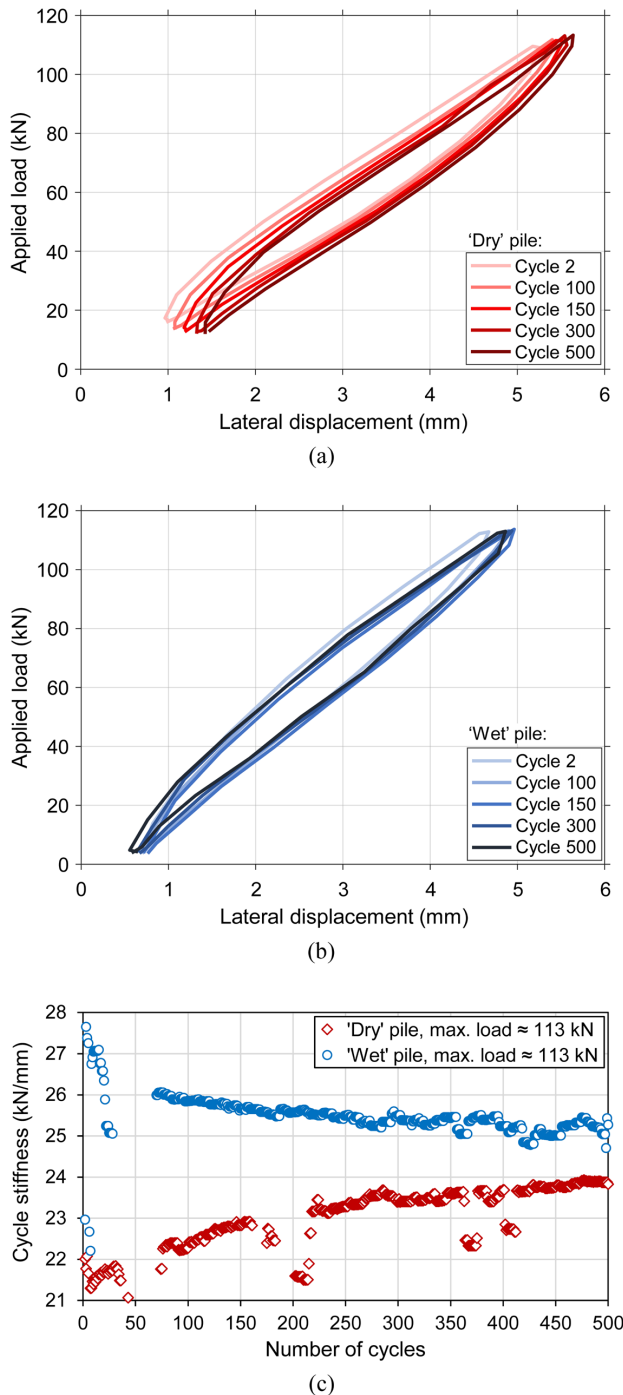


Figure 6. Cyclic test results for the initial 113 kN load cycles: (a) load vs displacement for 'dry' pile; (b) load vs displacement for 'wet' pile; (c) cycle stiffness for both piles.

Conversely, the 'wet' pile did not exhibit any significant ratcheting. Given that fissures would largely have swelled closed during the flooding process, local yielding within the infill material would not have occurred to the same extent during the subsequent application of lateral loading. Additionally, Gaspar & Jacobsz (2021) highlighted that as an initially unsaturated soil is wetted up, the strain required to cause failure (i.e. ductility) increases, which further justifies the absence of local yielding during the 'wet' test. It is noteworthy that the initial cycle stiffness for the 'wet' test was greater than that of the 'dry' test in Figure 6c. This is likely due to an increase in lateral net stress brought about due to the swelling of the clay, as was illustrated by the results reported by Gaspar et al. (2024). The increased confining stress would result in a decrease in volume compressibility of the clay, as long as the yield stress had not been exceeded. The stiffness of the 'wet' test system decreased marginally over the 500 cycles, with a total reduction of approximately 7%. The slight reductions can be described by considering the principles of unsaturated soil mechanics. Throughout the application of consecutive load cycles, local compaction would have occurred behind the pile. Assuming no changes in water content, this slight reduction in void ratio would have increased the degree of saturation, thus reducing the magnitude of soil suction and furthermore the stiffness of the clay. The same phenomenon was likely not observed for the 'dry' pile due to the high suction range under natural water content conditions, and the physical effects which dominate suction within this range. Monroy et al. (2015) performed oedometer tests with suction measurements, and found that expansive clay samples that were loaded or unloaded under constant water content conditions exhibited negligible changes in suction and vice versa, despite the volume changes invoked due to changes in net stress. These authors proposed that the relationship between water content and suction was unique (i.e. not influenced by volume change) beyond a certain suction threshold, due to the fact that suction is dominated by adsorption effects in this range (Baker & Frydman, 2009). This threshold was suggested to be between 100 and 400 kPa for various clay soils. As discussed previously, the suction at the 'dry' site would be at least an order of magnitude greater than this threshold based on measurements by Murison et al. (2023). Further evidence of a unique suction vs water content relationship at high suctions for the clay in this study was presented by Murison et al. (2023). Full primary drying and wetting curves were measured in the megapascal range for samples prepared at various different initial void ratios. Observable differences due to initial density could be seen in the suction vs void ratio and suction vs degree of saturation relationships, but there were no significant differences due to initial density in the suction vs water content relationships.

The bending strains measured along the length of the tension side of the piles at various cycle peaks have

been plotted in Figure 7, where $1 \mu\epsilon = 10^{-6} \text{ m/m}$. For both of the piles, there was no significant change in shape of the strain distribution with increasing cycle number. As shown in Figures 7a and 7b, any increase in the strain magnitude was largely confined to the first 100 cycles, after which the distributions remained constant for all practical purposes. For the gauges at depths between 0.75 m and 1.75 m (i.e. the three gauges that registered the largest strains for both piles), the increase in peak strain over the 500 cycles ranged between 6.3% and 9.9% for the ‘dry’ pile, and 11.2% and 14.6% for the ‘wet’ pile. Also illustrated in Figure 7 is the theoretical strain at which the concrete would crack ($72 \mu\epsilon$), given the properties presented in Table 2 and assuming linear elastic behaviour prior to cracking. Evidently, cracking took place at the outermost fibres of the concrete section in the upper portions of both piles at the onset of load application. Figure 7c shows that whilst the strains in the ‘dry’ pile were of marginally greater magnitude, the bending behaviour of the ‘wet’ and ‘dry’ piles was notably similar. Maximum strains registered for the ‘dry’ and ‘wet’ piles were $280 \mu\epsilon$ and $220 \mu\epsilon$ respectively.

5.2 Cyclic loading under increased load magnitude

The lateral load–displacement responses for a ‘dry’ pile under cycles with a maximum load of 175 kN, and for the ‘wet’ pile under cycles with a maximum load of 145 kN, are given in Figures 8a and 8b respectively. The cycle stiffness behaviour for both cases is given in Figure 8c.

A contrast in behaviour is evident through the significant increase in magnitude of the displacements of the ‘wet’ pile in Figure 8b. The pile exhibited a significant reduction in stiffness immediately upon the application of the first load cycle, suggesting that the capacity of the system had been exceeded. Seeing as the same phenomenon did not occur for an identical reinforced-concrete pile at the ‘dry’ site, this reduced capacity has been attributed to the applied stresses under increased loading exceeding the yield stresses of the intact clay masses adjacent to the ‘wet’ pile. These yield stresses would have reduced due to swell-induced softening brought about during flooding of the ‘wet’ site (Gens & Alonso, 1992).

The ‘dry’ pile exhibited progressive displacement of approximately 1.8 mm, largely taking place within the first 100 cycles. After approximately 200 cycles, a stable load–displacement loop had formed with no further ratcheting. The ‘wet’ pile (which had not exhibited ratcheting under the 113 kN cycles) continued to accumulate progressive displacements throughout the 400 cycles, with a translation of more than 12 mm from the first to final cycle. In similar fashion to the previous load magnitude, as the cycle number increased the cycle stiffnesses marginally increased and decreased for the ‘dry’ and ‘wet’ piles respectively. These phenomena are likely to be attributable to the same respective mechanisms that were discussed for the 113 kN stages. One key difference is that the ‘dry’ pile cycle stiffness was roughly equal for the final cycle under both load magnitudes, whereas the ‘wet’ pile exhibited a reduction in cycle stiffness in excess of 80% due to an increase in load magnitude of approximately 28%.

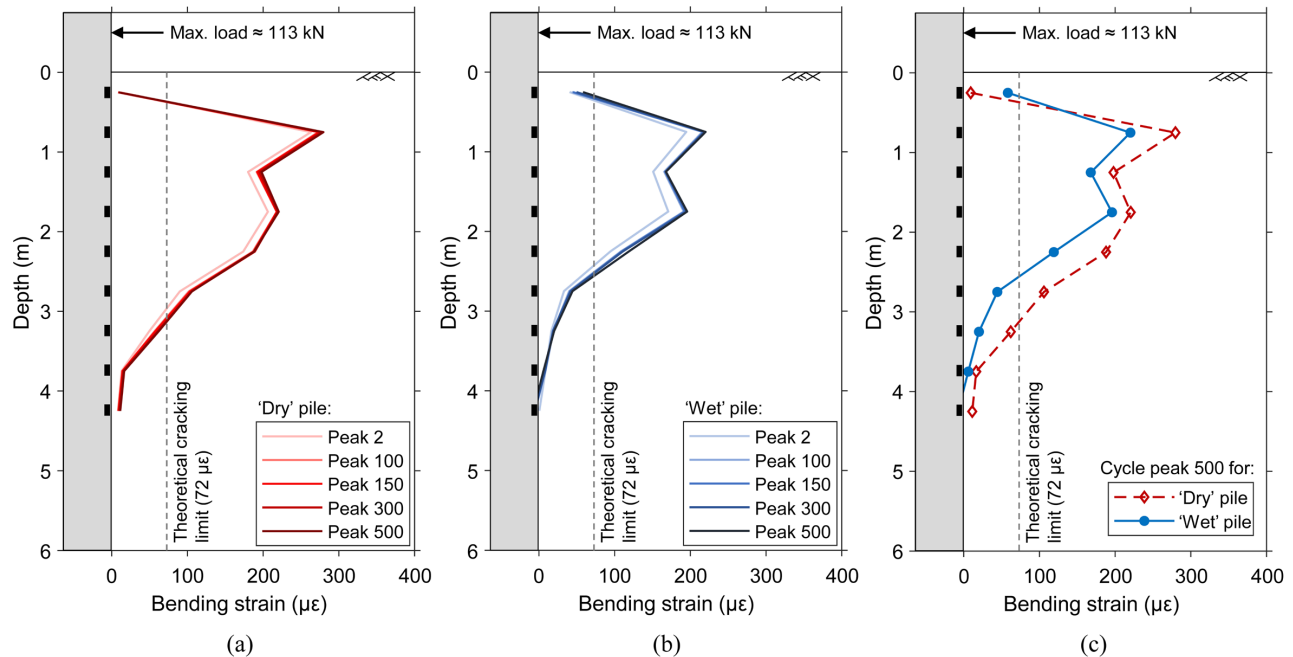


Figure 7. Bending strain distributions measured on the tension side of the piles at cycle peaks under the initial 113 kN load cycles: (a) ‘dry’ pile; (b) ‘wet’ pile; (c) comparison of final cycles.

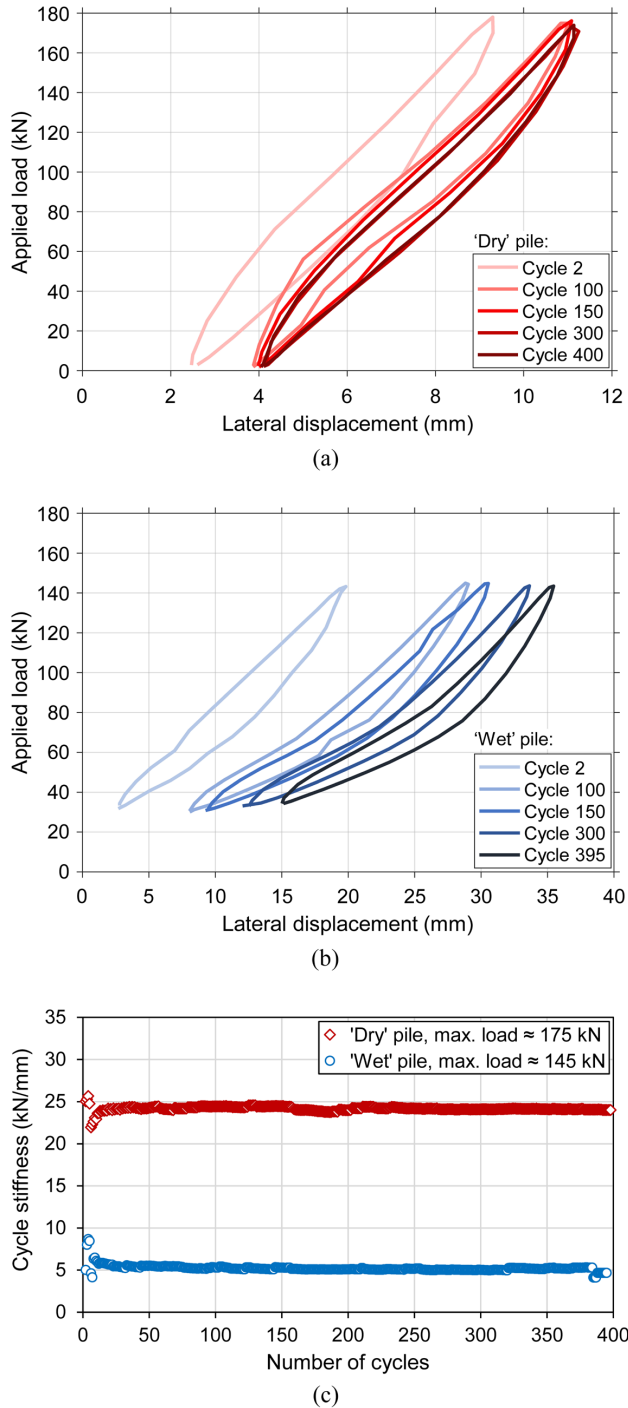


Figure 8. Cyclic test results under increased load magnitudes: (a) load vs displacement for 'dry' pile; (b) load vs displacement for 'wet' pile; (c) cycle stiffness for both piles. Note the different displacement axis scales.

The distributions of tensile strain due to bending for various cycle peaks are given in Figure 9. Once more, it should be noted that the 'dry' pile considered here is not the same pile considered in the previous and subsequent sections, but rather an identical pile founded in the 'dry' site with identical loading

history prior to this test. This is the reason for the slightly different shape in bending strain distribution when comparing Figures 7a and 9a. A significant crack had developed in the 'dry' pile at the onset of loading, with a maximum strain of $795 \mu\epsilon$ at 1.25 m. The greatest change in strain registered over the application of 400 load cycles was at 2.25 m, where the progression of cracking caused an increase in strain of 39% (from 184 – $256 \mu\epsilon$), the majority of which took place during the first 100 cycles. However, the shape and magnitude of the strain distribution remained largely unchanged under repeated loading. The 'wet' pile exhibited strains exceeding $400 \mu\epsilon$ over the entire upper 4 m of the pile from the onset of loading. This indicates significant cracking throughout the pile due to the reduced post-yielding soil resistance. The cracks continued to propagate as further load cycles were applied, with a tensile strain increase between 36 and 43% in the first 100 cycles and an increase between 52 and 67% over the entire 395 cycles (considering depths between 0.75 m and 3.75 m). The maximum strain experienced at these higher applied loads ($1347 \mu\epsilon$ at 1.75 m) was 512% greater than that of the previous load stage, despite an increase in load of only 28%, highlighting the severe contrast in behaviour after the intact clay masses had yielded. Considering the properties outlined in Table 2 and assuming linear elastic conditions for steel, the steel reinforcement to which the FBGs were attached would yield at $2250 \mu\epsilon$, which would constitute ultimate failure of the pile. Since the strains did not exceed this limit, the pile at the 'wet' site had not structurally failed during this test, but displacement of over 35 mm would constitute a serviceability failure for many applications.

Although the increase in lateral stress due to swelling of the clay increased the system stiffness at lower imposed loads, the effects of swell-induced softening were evident under the increased load. This was marked by a reduction in both the resistance (cycle stiffness) and the serviceable capacity of the 'wet' pile when compared to the 'dry' pile. It is thus likely, with reference to the discussion by Gaspar et al. (2024), that the degree of swell at the time of testing was such that the dominant mechanism influencing pile capacity was swell-induced softening.

5.3 Monotonic loading to failure

After the application of load cycles, the piles were loaded monotonically to 'failure' under load control. During these tests, the monotonic load was incrementally ramped up and held constant for a period of 10 minutes at each target load. Due to the large deformations anticipated for the 'wet' pile, an LVDT with a range of 100 mm was used. After the loading phase was complete, the load was incrementally reduced back to zero whilst the unloading displacements were recorded. Figure 10a shows the applied load versus lateral displacement for the 'dry' and 'wet' piles. Figure 10b shows the secant stiffness relative to the origin (i.e. the ratio between load and displacement at any point) for the monotonic tests.

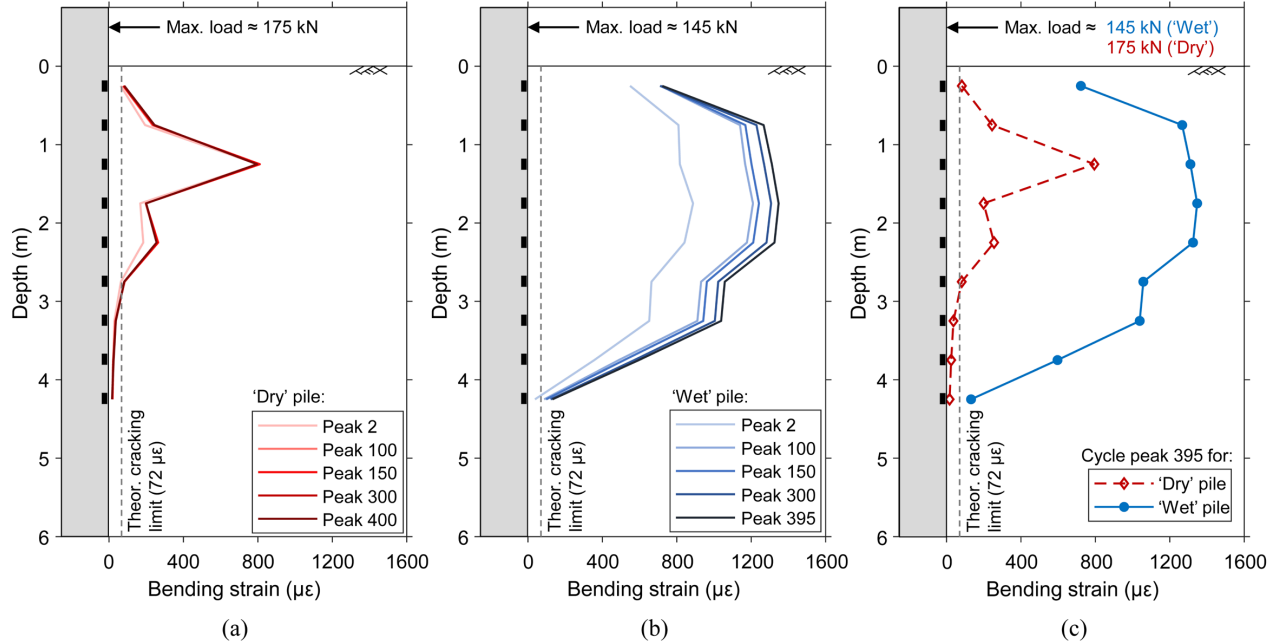


Figure 9. Bending strain distributions measured on the tension side of the piles at cycle peaks under increased load magnitudes: (a) 'dry' pile; (b) 'wet' pile; (c) comparison of final cycles.

To reference the cyclic test results to the monotonic tests, the cycle stiffness ranges for each peak load magnitude have been included in Figure 10b as well.

In contrast to the cyclic tests, the 'dry' pile exhibited a stiffer response than the 'wet' pile from the onset of loading. This is most likely due to the significant cracking and associated reduction in structural stiffness of the 'wet' pile that occurred during the second cyclic stage. Once the loads applied during the cyclic test stages had been exceeded, the 'dry' pile continued to exhibit a stiffer response than that of the 'wet' pile by at least a factor of two for all load magnitudes. At the maximum applied load of 413 kN for the 'dry' test, the maximum pile head displacement was 27.0 mm (4.5% of the pile diameter). For the 'wet' test, upon reaching a maximum load of 366 kN, the actuator went out of range, causing a reduction in load from 366 to 316 kN over the period where the load was meant to be held constant. However, despite the decrease in applied force, the lateral displacement continued to increase. This indicated ultimate structural failure of the pile (a fact that is supported by the later discussion of measured strain distributions). A maximum displacement of 68.4 mm (11.4% of the pile diameter) was recorded for the 'wet' test. After unloading back to a lateral load of 0 kN, 4.8 mm and 12.9 mm of imposed permanent deformation were recorded for the 'dry' and 'wet' piles respectively.

The distributions of tensile strain due to bending at the start and end of various target loads are given in Figure 11. Upon inspection of the 'wet' test distributions in Figure 11b, ultimate structural failure of the pile can be confirmed seeing as strains which significantly exceeded the limit at which steel

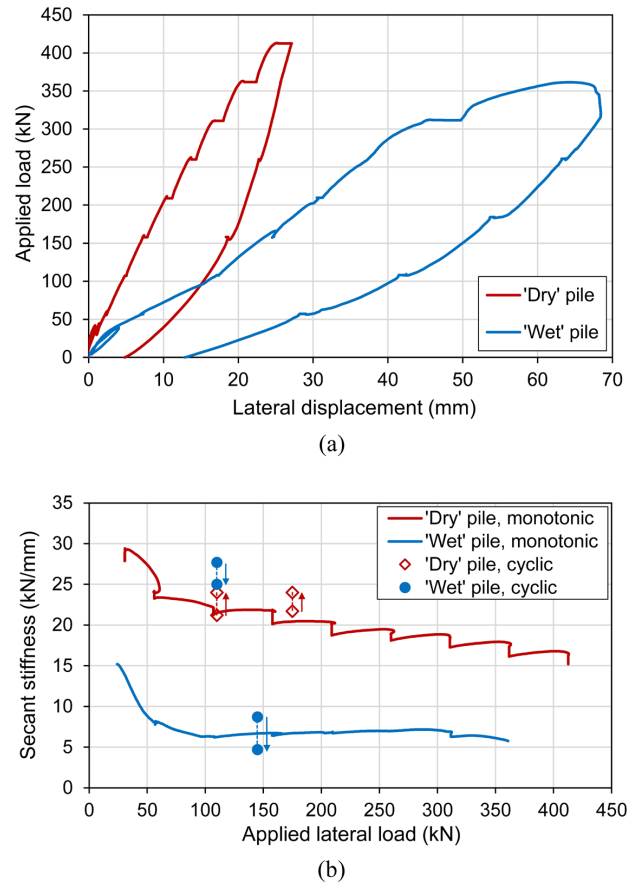


Figure 10. Monotonic test results: (a) load–displacement behaviour under monotonic loading and unloading; (b) secant stiffness as a function of peak lateral load for the monotonic and cyclic tests.

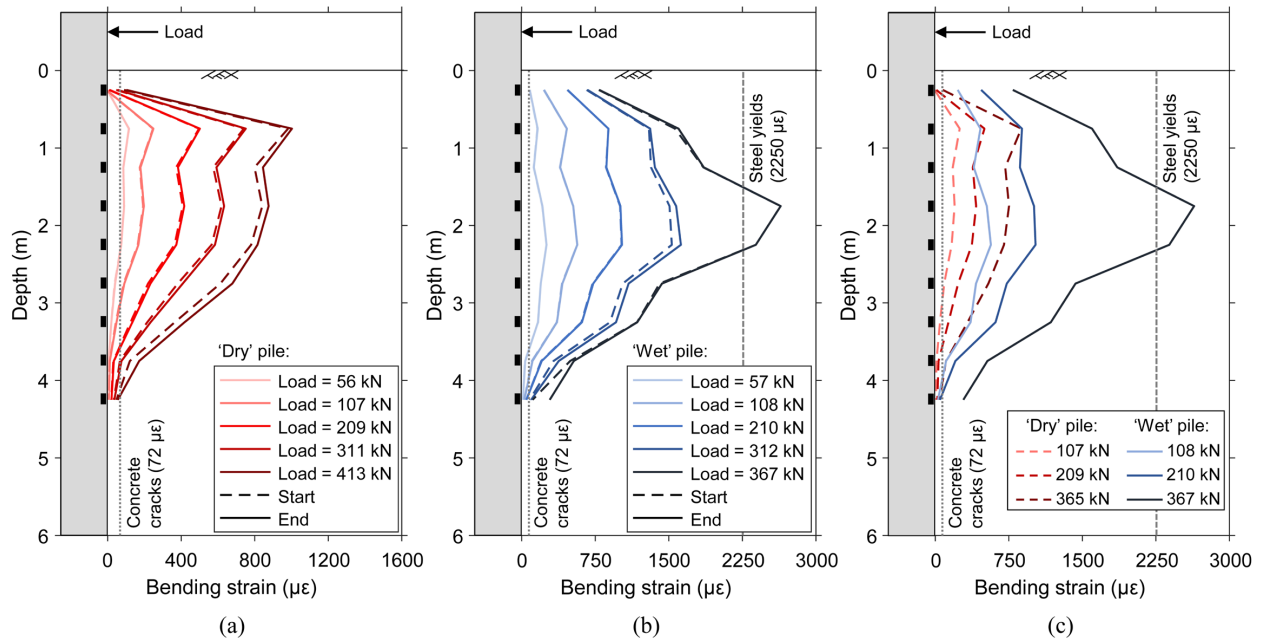


Figure 11. Bending strain distributions measured on the tension side of the piles at various load magnitudes during monotonic loading: (a) 'dry' pile; (b) 'wet' pile; (c) comparison of final cycles. Note the different strain axis scale for (a).

yields were recorded at depths of 1.75 m and 2.25 m. The shape of the strain distribution for the 'dry' pile in Figure 11a closely resembled the shape exhibited in the 113 kN cyclic stage (Figure 7a). A maximum strain of 1002 $\mu\epsilon$ was recorded at the maximum load of 413 kN. Despite the maximum load being nearly 3 times greater, the largest bending strain and pile head displacement recorded during the monotonic 'dry' test were both less than that of the 145 kN cyclic stage for the 'wet' pile. Considering the comparison presented in Figure 11c, it is also evident that the bending strains of the 'wet' pile were substantially greater than those of the 'dry' pile at all depths during monotonic loading. These observations all highlight the considerable contrast in behaviour of piles in the large-strain (i.e. high stress) range that is brought about by swelling of the expansive clay.

The effects of the structural damage incurred by the 'wet' pile during the 145 kN cyclic stage can be further illustrated by considering the strain distributions registered during the 113 kN cyclic stages, compared with the distributions at the same load magnitude during the monotonic tests. These have been illustrated for both piles in Figure 12. The distributions after 500 cycles at 113 kN are given by the broken lines. Due to the permanent deformations from the 145 kN cyclic stage, when reloaded to 108 kN monotonically, the 'wet' pile exhibited strains more than double that of the 113 kN cyclic stage for depths up to 2.25 m, and an order of magnitude greater from 2.75 m onwards. The presence of these permanent strains gives an indication of the permanent bending moments built into the 'wet' pile after cyclic loading. In contrast, the 'dry' pile strains when reloaded to 107 kN closely resembled the strains at the end of the 113 kN cyclic

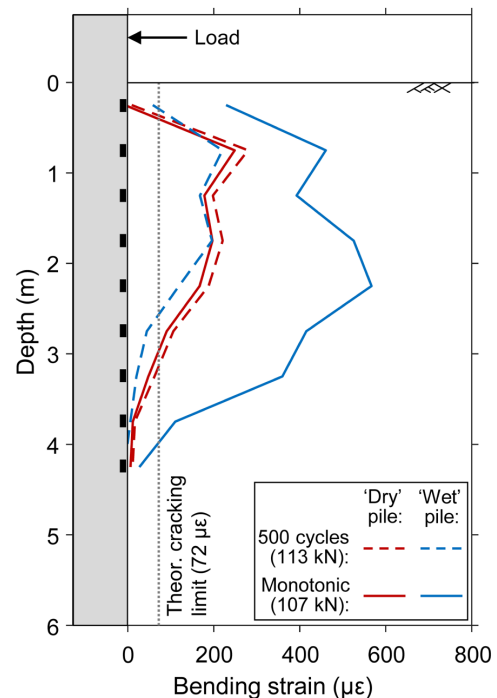


Figure 12. Bending strain distributions measured on the tension side of the piles at approximately 110 kN, for the 500th cycle during initial cyclic loading and for the monotonic loading after the cyclic tests.

loading stage. In fact, the monotonic test strains were slightly lower than those recorded during cyclic loading, likely due to the 6 kN difference in load magnitude.

The differences in behaviour depending on moisture conditions and stress levels, as well as the associated implications, have been summarised with reference to the site characterisation data and element tests in the following section.

6. Insights from in-situ and laboratory tests

The results presented in this study have highlighted that the behaviour of piles under different moisture conditions highly depends on the range of strains or stresses to which they are subjected. To extend the conclusions reached by Gaspar et al. (2024): whether swelling of an expansive clay causes an increase or decrease in resistance to lateral pile deformation is not only a function of the degree of swell and overburden stress, but also of the magnitude of lateral loading being imposed on the soil. At the initial stress range tested, piles founded in expansive clay that had been artificially flooded for six months to induce swell exhibited marginally stiffer mechanical responses to piles founded in the same expansive clay profile at natural water content conditions after the dry season. This was attributed to an increase in lateral stress acting on the pile due to the clay swell. However, at higher applied loads (and thus strains), the mechanical response of the ‘wet’ pile in the artificially swelled clay was substantially softer than that of the ‘dry’ pile in clay under natural moisture conditions, as indicated by far larger displacements and bending strains. Under increased loading, it became evident that although the lateral stress had increased due to swell, the effects of swell-induced softening were the dominant mechanism. These trends were reflected in the in-situ test results as well. The shear moduli determined at very small strains from CSW testing were similar at the ‘wet’ and ‘dry’ sites (with slightly higher stiffnesses for the ‘wet’ site), but the large-strain penetration resistance (from SPTs) of the ‘wet’ site was substantially lower than that of the ‘dry’ site.

It was suggested that the differences in observed pile behaviour were due to the fact that the yield stress of the intact masses of soil in which the ‘wet’ pile was founded had been exceeded. The soil in which the ‘dry’ pile was founded had not yielded despite being tested at the same load magnitude. This can be explained by considering the phenomenon of swell-induced softening. Gens & Alonso (1992) illustrated the concept through a critical state soil mechanics-based constitutive model for highly expansive unsaturated clays. The model allows for a reduction in macrostructural yield stress to be induced by the microstructural swelling that occurs due to the reduction in suction along a wetting path. This reduction in yield stress was evident in a double oedometer test conducted on samples obtained at the test site in the current study from a depth of 3 m. The test was conducted according to guidelines by Jennings & Knight (1957), and is depicted in Figure 13. Note that the

stress state variable used is net vertical stress ($\bar{\sigma}_v$), which is simply vertical total stress minus pore air pressure (which is assumed to be equal to zero). Figure 13a shows the incremental loading path of a sealed sample under constant water content (emulating ‘dry’ site conditions), as well as a parallel sample where the cell was flooded under a seating stress of 50 kPa (the approximate in-situ net vertical stress at 3 m) and allowed to swell (thus emulating ‘wet’ site conditions). Casagrande (1936) constructions were used to determine the yield stress ($\bar{\sigma}_{v,y}$) for each sample. The yield stress for the sample that was soaked and brought to zero suction was approximately an order of magnitude lower than that of the sample loaded at natural water content. The stiffness of the samples at various stress ranges has been expressed through constrained modulus ($E_{\text{oed}} = \Delta\bar{\sigma}_v / \Delta\varepsilon_v$, where ε_v is volumetric strain). The constrained modulus for each test has been normalised by net stress in Figure 13b. This aims to separate the reduction in stiffness due to yielding from the increases in stiffness due to increasing confining stress. This figure shows the drastic reduction in stiffness after yielding of the ‘wet’ sample, and shows the significant difference in stiffness behaviour between

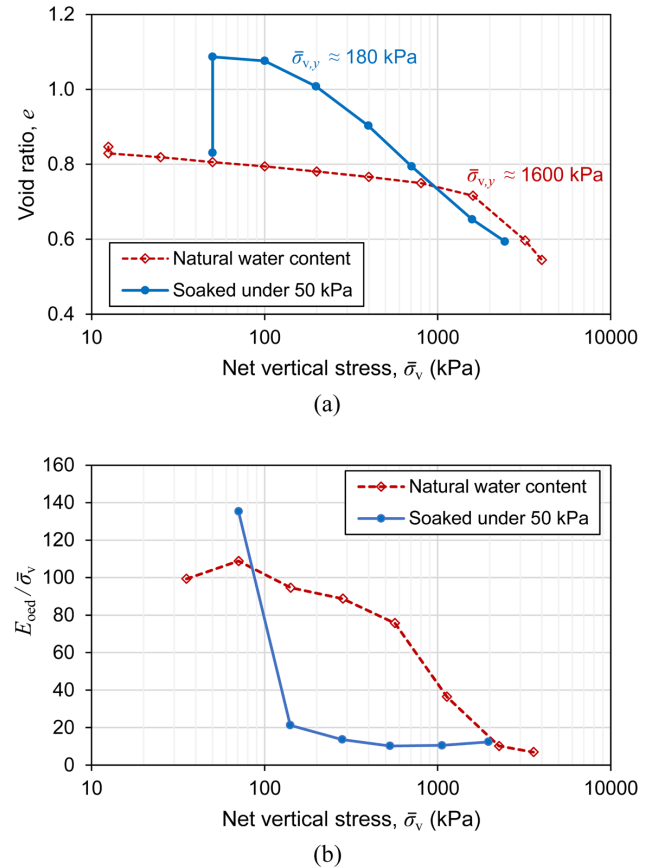


Figure 13. Double oedometer test results showing (a) reduction in yield stress due to swell, and (b) differences in soil stiffness due to swell.

the ‘wet’ and ‘dry’ samples at stresses greater than the ‘wet’ yield stress and less than the ‘dry’ yield stress. This region is an analogy for the loading range within which the ‘wet’ and ‘dry’ piles exhibited substantially different behaviour. Interestingly, the constrained modulus for the ‘wet’ sample prior to yielding was similar in magnitude (and marginally greater) than that of the ‘dry’ sample. This mimics the trend observed in the CSW G_0 profiles and in the initial pile response under the 113 kN cycles.

Although it may be straightforward to understand and categorise the pile response into small-strain and large-strain (or pre-yielding and post-yielding) behaviour, the primary challenge in implementing these findings would be determining the load at which this distinction occurs for a pile founded in expansive clay that has been subjected to large changes in water content. An increase in lateral load of 28% from 113 kN to 145 kN resulted in a five-fold reduction in secant stiffness response and a five-fold increase in maximum tensile strain from bending for the ‘wet’ pile.

It is thus imperative when designing piles in expansive clays for lateral loading to carefully consider the interrelationship of three factors and the influence that these may have on the functional capacity: (i) the duration of swelling expected, along with the ground conditions, drainage boundaries and permissible temporal water content fluctuations that may occur in situ, (ii) the changes in yield stress that these fluctuations might bring about through swell-induced softening, and (iii) the expected magnitude of loading and whether this might cause exceedance of these yield stresses of the intact swelled soil, leading to a transition from small-strain to large-strain pile behaviour.

7. Conclusions

This study detailed an investigation into the effects of clay swell on the lateral resistance of large-scale free-ended piles by comparing behaviour of a ‘dry’ pile founded in expansive clay under natural water content conditions to that of a ‘wet’ pile, which was founded in the same clay but subjected to a six-month wetting period. Under the application of 500 cycles with a maximum load of 113 kN, the two piles exhibited similar behaviour in terms of load–displacement response, cycle secant stiffness and bending strains. The ‘wet’ pile exhibited a marginally greater resistance to deformation in this test, which was attributed to the increase in lateral stress against the pile shaft due to swelling of the clay. When the magnitude of the maximum load was increased to 145 kN for the ‘wet’ pile and 175 kN for a ‘dry’ pile for the subsequent 400 cycles, no substantial change in behaviour was evident for the ‘dry’ pile, but a severe reduction in stiffness and the onset of large displacements and bending strains were noted for the ‘wet’ pile. This was attributed to the effects of swell-induced softening, which had significantly reduced

the yield stress of the intact clay masses within which the pile was founded. A five-fold increase in bending strains and five-fold reduction in cycle stiffness were brought about by a 28% increase in lateral load, which highlighted the clear contrast in pre-yielding and post-yielding behaviour of the swelled clay. Monotonic load tests aimed at causing failure of the piles were conducted after cyclic loading. A maximum load of 413 kN was applied to the ‘dry’ pile, at which point the secant stiffness remained twice that of the ‘wet’ pile after the 145 kN cyclic stage. Additionally, the maximum bending strains were significantly lower in magnitude than those of the 145 kN ‘wet’ pile cyclic stage. No signs of structural failure were observable for the ‘dry’ pile. The ‘wet’ pile exhibited a softer response than that of the ‘dry’ pile throughout monotonic loading and reached ultimate structural failure at a load of 366 kN. At ultimate failure, the tensile strains had exceeded the steel reinforcement yielding limit and a maximum displacement of 68.4 mm (0.114 times the pile diameter) was registered.

Gaspar et al. (2024) concluded that two mechanisms may influence pile capacity during swelling of an expansive clay. An increase in capacity may be brought about by a swell-induced increase in lateral stress, and a decrease in capacity by the effects of swell-induced softening. The dominant mechanism was found to be strongly dependent on the degree of swell, which can vary both temporally and spatially throughout a clay profile. In the current study, it was observed that the magnitude of strain imposed by applied loading may also influence which mechanism dominates pile behaviour. At lower load levels, the pile in swelled clay exhibited greater stiffness, consistent with an increase in lateral stress. However, at higher load magnitudes, where larger strains developed, swell-induced softening became the dominant mechanism. This resulted in a significantly softer response and reduced serviceable and ultimate capacities against lateral loading.

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Declaration of interest

The authors have no conflicts of interest to declare. All co-authors have observed and affirmed the contents of the paper and there is no financial interest to report.

Authors' contributions

Ruan A. Murison: conceptualization, formal analysis, investigation, methodology, visualisation, writing – original draft. Tiago A. V. Gaspar: conceptualization, methodology, visualisation, supervision, writing – review and editing. Gerhard Heymann: conceptualization, formal analysis, supervision, writing – review and editing. Schalk W. Jacobsz: conceptualization, funding acquisition, project administration, supervision, writing – review and editing. Ashraf S. Osman: conceptualization, funding acquisition, project administration, writing – review and editing.

Data availability

The data generated and analysed in this study are available from the corresponding author by request.

Declaration of use of generative artificial intelligence

This work was prepared without the assistance of any generative artificial intelligence (GenAI) tools or services. All aspects of the manuscript were developed solely by the authors, who take full responsibility for the content of this publication.

List of symbols and abbreviations

e	void ratio
$(EI)_{\text{pile}}$	bending stiffness of the pile
E_{oed}	constrained modulus
E_{soil}	Young's modulus of the soil at the pile base
G_0	small-strain shear modulus
G_s	specific gravity
I_p	plasticity index
K_R	relative stiffness of the pile
L	pile embedment depth
N_{60}	SPT blow count at standard energy ratio of 60%
w	gravimetric water content
w_L	liquid limit
$\Delta\varepsilon_v$	change in volumetric strain
$\Delta\bar{\sigma}_v$	change in net vertical stress
ε_v	volumetric strain
ν	Poisson's ratio
$\bar{\sigma}_v$	net vertical stress
$\bar{\sigma}_{v,y}$	net vertical yield stress
c/c	centre-to-centre
CSW	continuous surface wave
FBG	fibre Bragg grating
LVDT	linear variable differential transformer
NGL	natural ground level
SPT	standard penetration test
WT	water table
XRD	X-ray diffraction

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A strategic and novel field trial to investigate the dynamic performance of cement-stabilised chalk as embankment fill for high-speed rail earthworks

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Article

Keywords

High-speed rail
Field trial embankment
Stabilised chalk
Static stiffness modulus
Dynamic performance
Compaction control

Abstract

Significant quantities of site won fill material are generated from the High Speed Two (HS2) C23 main works civil engineering contract, during the excavation for the formation of cuttings and construction of tunnels. This study investigates the suitability of weathered structureless Chalk, stabilised with cement, for use as engineered embankment fill for high-speed rail earthworks. Given the dynamic loading conditions imposed by high-speed high frequency rail operations, the performance of stabilised chalk requires thorough evaluation. A strategic field trial embankment was constructed to assess the suitability of this material. The results demonstrate that the static stiffness modulus (E_{v2}) of the stabilised chalk meets HS2's minimum performance requirements, though it is found to be sensitive to cement curing time. Dynamic performance, evaluated through shear wave velocity (V_s) derived from Rayleigh wave characteristics, exceeded the minimum V_s assumed for engineered fill during detailed design, confirming compliance with HS2's dynamic performance criteria. Construction control measures, including optimal chalk grade, binder mix, moisture content, and compaction control are recommended to ensure consistent performance. These findings validate the use of cement-stabilised chalk fill for high-speed rail earthworks, enabling the development of sustainable earthwork solutions through material optimisation, waste reduction, and enhanced environmental efficiency.

1. Introduction

1.1 Background

The High Speed Two (HS2) project is a new High-Speed Railway Line currently under construction in the UK. It is a 230 km long new high-speed rail (HSR) that runs between London and Birmingham across the English Midlands.

Distinct system and performance characteristics of the HS2 project are described in the HS2 Technical Standard - Earthworks (TS-E) (HS2, 2019). The most notable are:

- The selected track slab system
- The high operating speed of 360 km/h
- The 120 years design life
- The stringent static and dynamic performance criteria to be achieved

- The strict construction verification and validation testing requirements of the end product

These performance requirements have specifically been set to ensure the safety, resilience and stability of the high-speed rail track.

1.2 The C23 Contract

HS2 Ltd (HS2) has awarded the C23 Contract for the design and construction of the civil infrastructure works to EKFB, a Joint Venture consisting of renowned international civil engineering construction contractors Eiffage, Kier, Ferrovial Construction and Bam Nuttal. In turn, EKFB has appointed Arcadis, Setec, and COWI Design Joint Venture (ASC) to provide design services and construction support for the civil infrastructure works.

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The C23 Contract encompasses an 80 km long open route section that connects the Chiltern Tunnel to Long Itchington Wood, located within the Central region between Wendover in Buckinghamshire and Southam in Warwickshire. The earthworks scope includes the design and construction of 30 mainline cuttings, 45 mainline embankments, and 7 km of green tunnels. Given the extensive scope of the project, a significant volume of earth moving and associated mass haul logistics are expected. The route and extent of the C23 Contract is presented as Figure 1.

The C23 contract is considered to be the largest single geotechnical contract currently being delivered in the UK.

1.3 Technical challenges

Major technical challenges were identified at the early stage of the design development process, requiring an effective strategy to be planned and meticulously executed to overcome these challenges and deliver sustainable

solutions that would ensure the safety, stability, resilience and longevity of the railway track. One such challenge was the selection of a slab-track form to limit long-term maintenance requirements. However, this track system cannot accommodate large movements, meaning that any displacement of the supporting platform must be kept within specific limits to maintain safe operations of the railway. To achieve this, key static and dynamic performance requirement together with very stringent long-term heave and settlement criteria were specified in the HS2 TS-E (HS2, 2019).

Another significant challenge was the ground conditions beneath the C23 route (Figure 2), which comprise over-consolidated high plasticity Clays, weak mudrocks, and Chalk.

These geological units are known for their unfavorable response to loading and unloading conditions. These ground risks have further exacerbated the challenge already presented by the strict post track installation heave and settlement criteria.

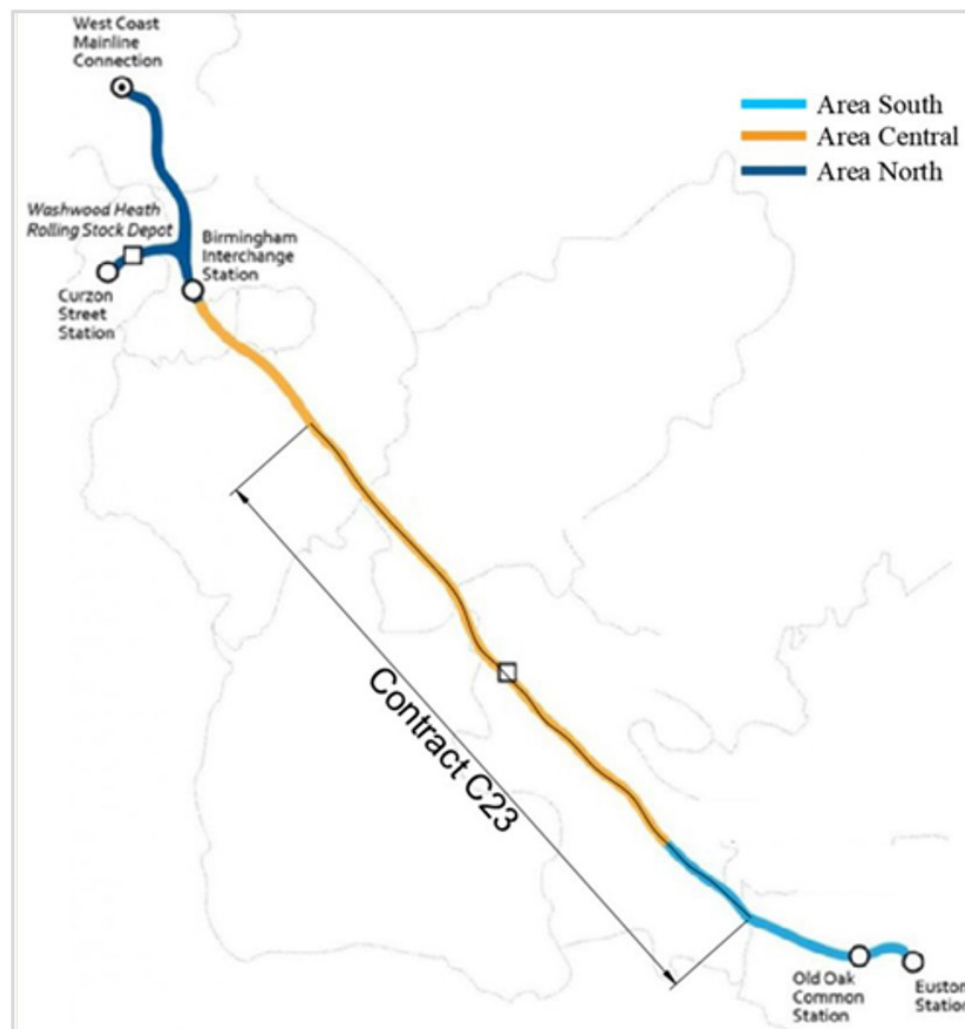


Figure 1. HS2 C23 Contract section route map.

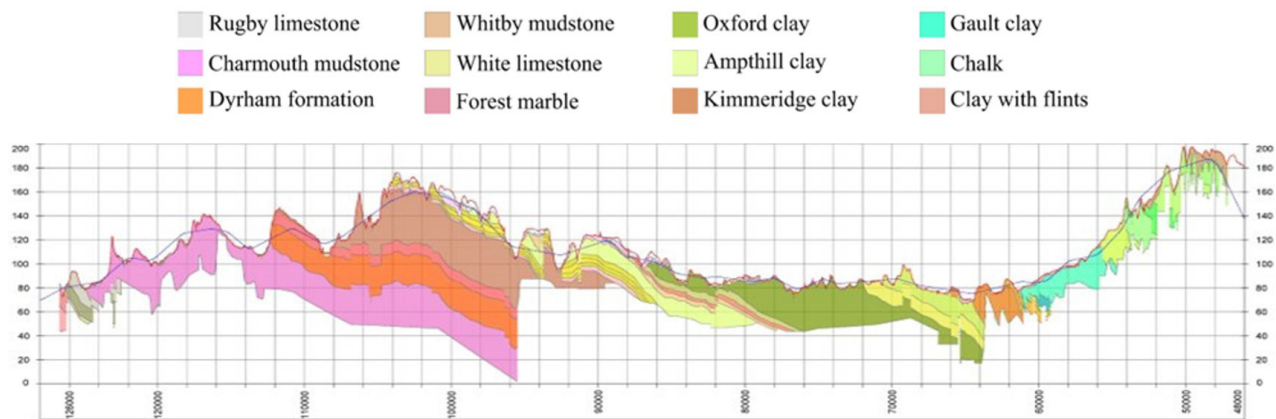


Figure 2. Ground conditions beneath the HSR C23 route.

Based on detailed evaluation of these challenges, uncertainties relating to predicting long-term earthworks heave and settlement, and dynamic amplification due to repeated transient loading, were identified as the main risks that would need to be adequately understood, assessed, and mitigated during the design and construction phases. Following collaborative discussions and risk reduction workshops, an innovative earthworks strategy was developed and executed to overcome these critical challenges. The scope comprised the construction and monitoring of three novel large scale instrumented field trials (two embankments and one cutting) to obtain better understanding of the characteristics and behaviour of the in-situ high plasticity foundation strata (Katsigiannis, 2024).

An additional significant challenge was the shortage of high-quality site-won granular fill material along the C23 route. This constraint added further complexities to mass haul logistics, including impact on costs and construction schedule. To address this challenge, the field trial programme was extended to include large scale material field trial embankments to ascertain the feasibility of treating the unsuitable site-won fills with the aim of rendering them suitable for re-use in mainline earthworks, transitions to structures, and for foundation treatment. This approach enables the development of sustainable solutions that will not only minimise the need for imported engineered granular fill but also reduce costs, shorten the construction schedule, and reduce the carbon footprint.

The suitability of re-using these site-won fill materials as earthworks fills was thoroughly evaluated by following a comprehensive selection and testing process. This included assessing their performance and demonstrating that they comply with HS2 technical standards when used as HSR embankment fill and as replacement material for foundation treatment. Both stabilised and unstabilised fills were investigated. In addition, laboratory testing trials were conducted to supplement the field trials and provide other essential material characteristics required for design and construction, using these site-won fill materials.

1.4 Scope and objectives

According to HS2 technical standards, the design of mainline earthworks, earthwork transitions and in-situ foundation strata must adhere to stringent static and dynamic performance criteria that are reflective of the high operational speed of the railway. These criteria ensure the safety and stability of the high-speed rail infrastructure.

Furthermore, the TS-E (HS2, 2019) and HS2 Specification for Civil Engineering Works (SCEW) (HS2, 2022) emphasise the importance of compliance testing during construction. This testing is crucial for verifying and validating the performance of the end product. Moreover, both the TS-E and SCEW highlight the significance of the field trials as an essential element of the design and construction control validation process. Field trials provide invaluable data for assessing material properties and behaviour under controlled conditions, enabling refinement and optimisation of the design and construction processes that will ensure the end product meets the stringent performance requirements.

This paper outlines the field trial embankment constructed at the trial site in the Great Missenden area, near section C2 of the C23 route, focusing on the use of cement stabilised chalk fill. The objectives of this trial were to:

- Assess the suitability of the cement stabilised chalk fill as HSR embankment fill in lieu of the engineered granular fill.
- Determine the minimum material acceptance criteria to be targeted during construction to demonstrate compliance with the static and dynamic performance requirements.
- Optimise the construction method for earthworks to ensure that the stabilised chalk fill meets the minimum static and dynamic performance requirements specified in the TS-E (HS2, 2019).

2. HSR earthwork embankment configuration

The TS-E (HS2, 2019) provides configuration and terminologies for the different mainline earthwork assets. Earthwork embankments are formed of two main parts, namely: the trackbed and the earthwork structure. The earthwork structure, in turn, comprises an upper embankment fill layer (UEF) and a lower embankment fill layer (LEF) below the prepared subgrade layer (PS). The configuration and terminologies for mainline earthwork embankments are presented as Figure 3.

As part of the project's environmental sustainability pledge, optimised and efficient solutions to reduce the carbon footprint were continuously sought during the detailed design process. For mainline earthworks, the re-use of the significant quantities of chalk fill to be generated on site for embankment construction was identified as an opportunity for optimising imported engineered granular fill.

3. Earthworks performance requirements

3.1 Earthworks design approach

HS2 requires that the design solution for embankments delivers a stable fill platform having consistent stiffness at formation level. The selected design solution is, therefore, expected to ascertain the following as a minimum:

- A stable foundation for the embankment structure.
- Suitable fill for use as UEF and LEF.
- Compaction targets that will achieve the prescribed minimum material stiffness.

3.2 Fill material acceptability and performance

According to HS2 TS-E (HS2, 2019), field trial earthworks shall be undertaken to demonstrate that the Designer's proposals will provide the required performance requirements.

For stabilised fills, the same standard states that: *"Treatment using lime, cement or other additives may be proposed to achieve the acceptability or performance limits required. These treatments may be by modification or stabilisation. The Designer shall confirm, with justification, that long-term performance will be maintained throughout the design life of the Earth Structure and that any proposed treatments will not be adversely affected by chemical properties of the ground or other agents."*

Accordingly, the field trial was conducted to gather construction and testing data necessary to assess the suitability of the proposed cement stabilised structureless Chalk as HSR embankment fill. The trial also aimed to determine the minimum acceptable material properties to be targeted during construction, ensuring the chalk fill is sourced, placed, pulverised, compacted, and tested in a manner that achieves the static and dynamic performance requirements specified in HS2 technical standards.

3.3 Performance requirements

HS2 requires that the embankment structures after placement and compaction achieve the minimum performance criteria defined below.

3.3.1 Dynamic performance requirements

The dynamic performance is determined from the assessment of the dynamic stiffness exhibited by the earthwork structures when subject to dynamic loading. This is the Dynamic Railway loading applied over a very short term (instantaneous) (Arup, 2017; HS2, 2019; HS2, 2022). The TS-E (HS2, 2019) defines the earthworks dynamic stiffness as the *"transient stiffness of the earthwork at any defined level in response to dynamic loading"* and specifies the performance criteria to be satisfied by the earthworks design and construction verification testing, as a limiting Rayleigh wave velocity (V_R), measured at Formation Level in the frequency range 10-50 Hz:

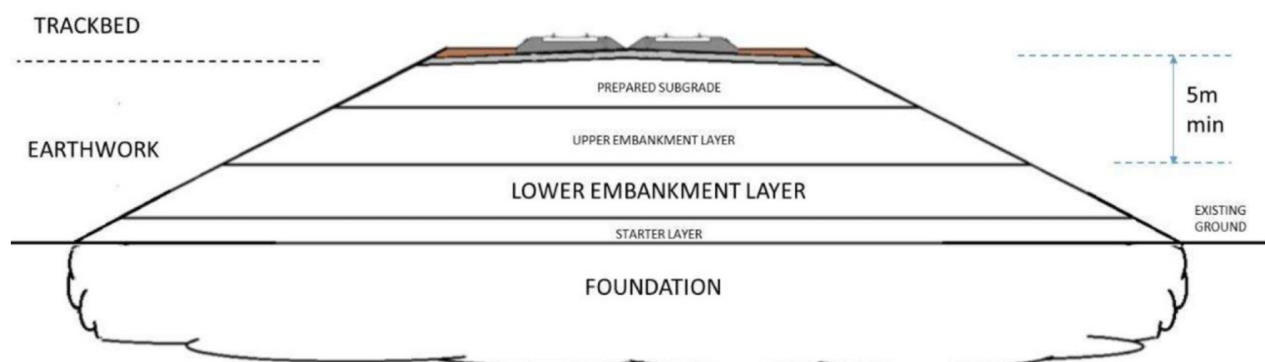


Figure 3. Typical HSR earthwork embankment configuration.

$$V_R \geq 1.6 \times DS \quad (1)$$

where DS is the design speed, which for HS2 C23 has been set at 360 km/h (100 m/s).

V_R is determined from the simplified direct relationship with the shear wave velocity (V_s) proposed by Richart Junior et al. (1970).

In the absence of trial testing information for embankment fill at the start of the design process, a minimum V_s of 250 m/s has been assumed for engineered fill forming the HSR embankments and the excavate and replace fill (E&R) for foundations treatment. This design value was revised to 300 m/s to reflect the high results recorded during field testing performed at various stages during the field trials (Hope et al., 2024; Saroglou et al., 2025).

3.3.2 Static performance requirements

The static performance is determined from assessment of the static stiffness of the embankment fill under static loading conditions. The TS-E (HS2, 2019) defines the earthworks static stiffness as the “*stiffness of the earthwork at any defined level under the application of a sustained load*”. The same TS-E specifies the performance criteria to be satisfied by the earthworks design and construction verification testing as a limiting vertical deformation modulus (E_{v2}), measured at Formation Level. The E_{v2} limits and compaction acceptability criteria for embankments are summarized in Table 1.

The HS2 TS-E requires that the static performance is demonstrated by design and validated by construction verification testing of the end product.

4. Chalk fill characteristics

Chalk is present at the C2 section of the C23 route (Figure 2). According to the BGS Lexicon (BGS, 2021b) and the ground investigation (GI) undertaken by HS2 during scheme design (HS2, 2016), the Chalk deposits in this area comprise the White Chalk Subgroup, which in turn consists of the following:

- Lewes Nodular Chalk Formation - weak to moderately strong nodular medium to high density chalk and hardgrounds with interbedded ‘soft to medium hard’ chalks (some of which are grainy), marls and

some interbedded chalks. The softer chalk is more abundant towards the top. The nodular chalks are present as regular seams.

- New Pit Chalk Formation – principally blocky, white firm to moderately hard chalk with numerous marl seams, including named marker horizons and ‘wispy marls’. Flint occurs sporadically in the upper part of the formation, described in the BGS as ‘firm to moderately hard’ (BGS, 2021a).
- Holywell Nodular Chalk Formation - generally a hard nodular chalk, weathering typically to silt rather than clay, with thin interbedded marls and significant shell debris.
- Zig Zag Chalk Formation - mostly firm, pale greenish grey to off-white blocky chalk, with the lower part characteristically more ‘marly’. Typically flint poor.
- West Melbury Marly Chalk Formation - typically, buff, grey and off-white soft, marly chalk. Typically flint poor. Noted in the BGS as hard grey limestones.

Based on the definition in CIRIA C574 (Lord et al., 2002), weathering determined from recovered cores ranged from fully weathered clast or matrix supported (grade Dc or Dm) to intact high strength (grade A). The Dm grade is defined in the CIRIA C574 as low-density Chalk that behaves as a cohesive, fine soil, where the comminuted matrix dominates. The grade Dc, on the other hand, is defined as low to medium density Chalk that behaves as a granular, coarse soil, where the clasts (intact Chalk lumps) dominate.

An extensive literature review of previous case studies was undertaken to obtain some understanding on the suitability, long-term behaviour, and performance of stabilised weathered chalk fill (Barker et al., 2018; Britpave, 2017; Judd, 1976; Lord et al., 2002; Phear et al., 2018a, b; Warren et al., 2003). Very limited data was available and was mainly related to other infrastructure projects. Key learning outcomes are as follows:

- Site specific characterisation of the weathered Chalk is needed for adequate assessment of its suitability for re-use.
- Intact dry density (IDD) is the most reliable parameter for characterisation of weathered chalk fill.
- IDD and moisture content (MC) are two characteristics that generally control the acceptability and level of treatment of weathered chalk fill.
- Setting reliable degree of pulverization (DoP) is critical to achieving uniform cement mixing distribution.

Table 1. E_{v2} limit and compaction performance requirements for embankments (HS2, 2019).

Earthwork layer	Material	E_{v2}	Min. Compaction acceptability criteria
		(MPa)	(Modified Proctor)
UEF	Stabilised	≥ 60 at surface ≥ 45 at base (1 m above LEF)	$DC \geq 98\%$ MDD $AV \leq 5\%$
LEF	Stabilised	-	To be determined from relationship testing

No E_{v2} limiting criteria are specified for LEF in HS2 TS-E.

The HS2 SCEW (HS2, 2022) classifies structured Chalk fill as Class 3 and stabilised soils as Class 9. The project specification labels the structureless Chalk grade Dm/Dc as Class 3B1 and the cement stabilised chalk fill as Class 9M fill. These terminologies are used throughout this paper. Testing found the structureless Chalk grade Dm/Dc (Class 3B1) to be low density chalk with an $IDD < 1.55 \text{ mg/m}^3$.

5. The embankment field trial

5.1 Trial location

The location plan of the field trial in relation to the HSR line is shown in Figure 4.

The trial site slopes gently from an elevation of about +140mAOD to an elevation of 138mAOD.

The structureless Chalk for the trial embankment construction has been sourced from a temporary excavation situated adjacent to the trial site. Figure 5 shows a layout plan of the trial site and the temporary excavation area.

The Chalk was excavated, transported and directly placed for the embankment trial construction without stockpiling.

5.2 Ground conditions at the temporary excavation area and the trial location

A review of the available GI information (HS2, 2016) in conjunction with the BGS Lexicon (BGS, 2021b) indicated that the trial site and the designated temporary excavation area sourcing the chalk fill were expected to be underlain by the undifferentiated Holywell Nodular Chalk and the New Pit Chalk formations. The New Pit Chalk at the top is described as consisting of principally blocky, white firm to moderate hard Chalk with numerous marls or pair marl seams. Structureless Chalk was anticipated to be locally present near surface and be predominantly of grade Dc, meaning that it behaves as granular coarse soil where the clasts (intact Chalk) dominate (Lord et al., 2002).

A supplementary GI was undertaken during the field trial to verify and validate the anticipated Chalk formation at the trial site. It comprised the excavation of 5 trial pits to a maximum depth of 4.0 m below ground level. Three of these were dug within the temporary excavation area and 2 at the embankment trial site.

For the temporary excavation, the interpretation focused on the assessment of the depth and extent of the structureless Chalk (Class 3B1), as it is the main constituent needed for preparing the cement-stabilised chalk fill Class 9M required for the embankment trial. The log and view of TP1, dug as part of the supplementary GI, are presented in Figure 6 as an example.

Interpretation of the factual data revealed the ground conditions at the temporary excavation area to comprise weathered structureless Chalk, consisting of 1 m of grade Dm Chalk becoming grade Dc with depth. Localised zones of more Structured Chalk (grades B & C) were also observed over the excavated depth. This confirmed the Chalk profile interpreted from previous investigations, using the grading proposed by Lord et al. (2002).

Similar ground conditions were encountered at the trial embankment site. These are generally consistent with the ground conditions revealed by the previous GI data. A longitudinal section depicting the interpreted ground profile at both areas is illustrated as Figure 7.



Figure 4. Great Missenden field trial location.

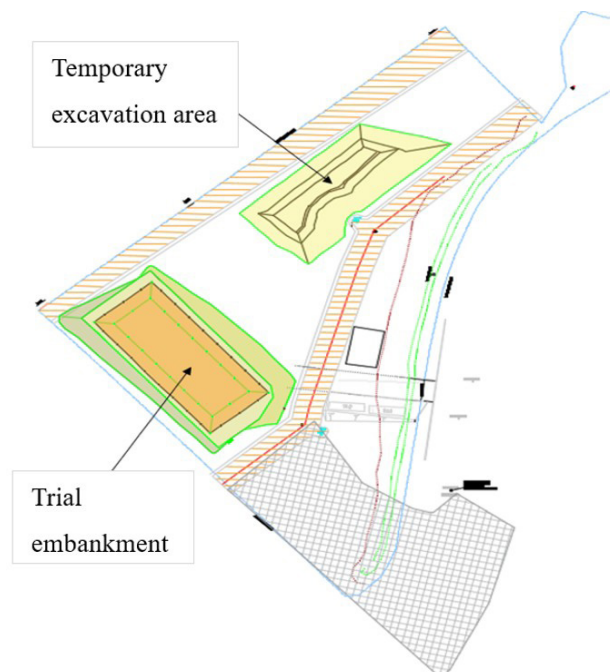


Figure 5. Layout plan of field trial site.

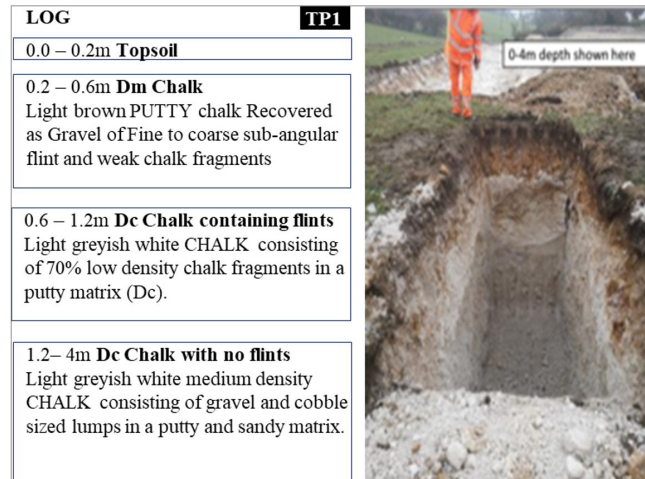


Figure 6. Ground conditions at temporary excavation area.

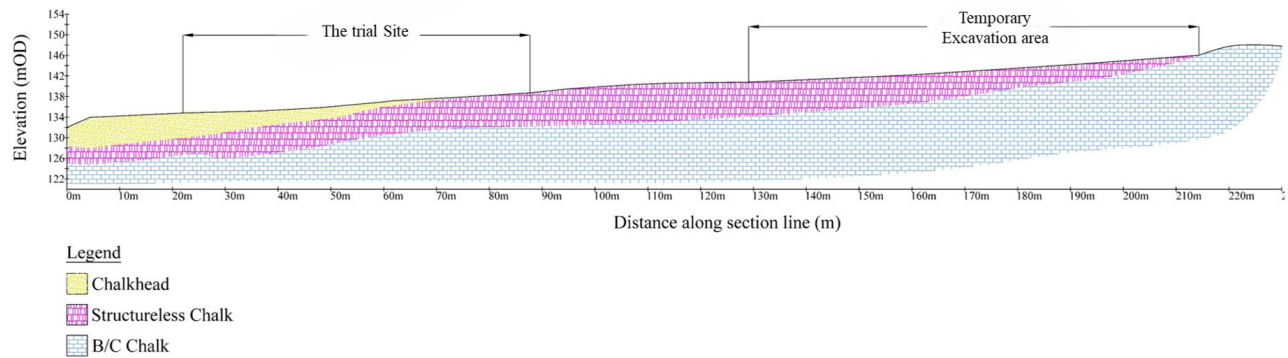


Figure 7. A longitudinal section showing the ground profile at the temporary excavation area and the trial site.

5.3 Trial embankment construction

5.3.1 Site preparation works

Due to variation in topography, the trial location was levelled, using cut and fill within the hillside to form a basal foundation platform for the trial embankment. The fill from the cutting comprised gravelly Clay with flint and Chalk. The foundation platform fill was improved using up to 5% lime stabilisation over the top 300 mm depth to provide a competent base for the trial embankment. Prior to construction of the fill platform, all topsoil at the trial location was stripped and the formation cleared in preparation for the embankment trial construction.

5.3.2 Trial embankment geometry and construction process

The trial embankment area was 61 m long and 20 m wide. The targeted maximum height was specified as 3.1 m, to be reached by placing 13 layers of Class 9M chalk fill. The side slopes were specified as 1v:2h.

The trial embankment was split into two testing areas, namely testing Area 1 and testing Area 2, each 30 m long and 20 m wide. Due to shortage of structureless Chalk Class 3B1, the height of Area 1 was revised to be 2 m and constructed, using up to 8 layers of Class 9M fill material.

The sequence adopted for the trial embankment construction is shown as a construction process in Figure 8. It includes material hauling, pulverisation, cement mixing, compaction and trimming of each fill layer. Plant and equipment used for construction are also shown on Figure 8.

The process followed for constructing the trial embankment is described below:

- For both Areas 1 and 2, fill layers 1 to 3 were constructed using grade Dm chalk mixed with 2% cement.
- For Area 1, fill layers 4 to 8 were constructed using grade Dc chalk mixed with 2% cement. Construction of testing Area 1 finished at layer 8.
- For Area 2, fill layers 4 to 8 were constructed using grade Dc chalk mixed with 3% cement, whilst layers 9 to 13 had 4% cement. Construction of testing Area 2 finished at layer 13.

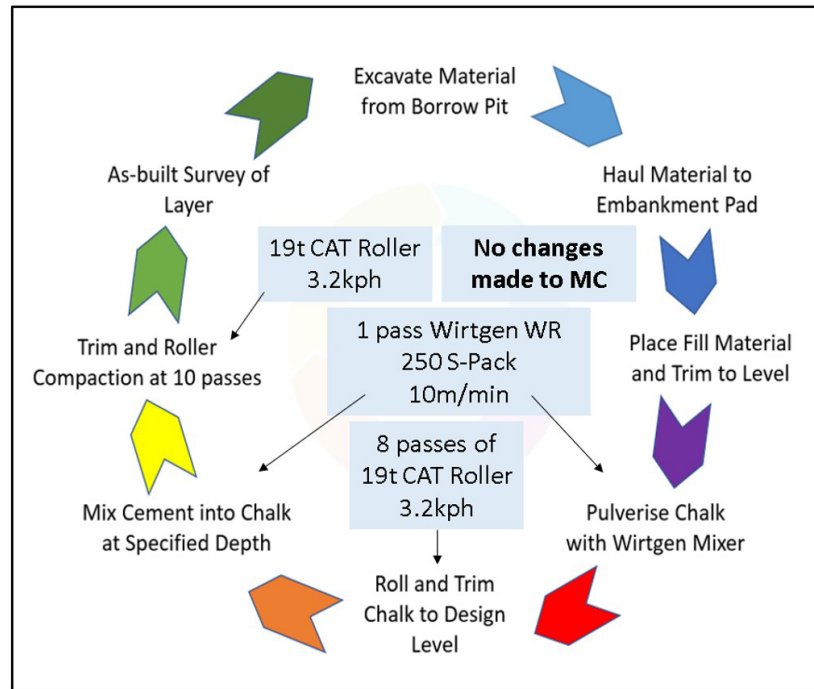


Figure 8. Construction process.

- Individual layers were specified to typically achieve a compacted thickness of 250 mm.

The completed trial embankment is presented in Figures 9a, 9b and 9c. Testing Areas 1 and 2 are apparent on Figure 9c.

Samples were obtained at targeted locations and depth during construction of each testing area, as well as at finished levels upon completion, for laboratory testing.

5.3.3 Testing schedule and locations

In situ and laboratory testing were performed during the trial. The specified in-situ tests and their locations, together with the sampling locations for laboratory testing, are shown on Figure 10. The individual fill layers in each testing area, along with the cement binder mix applied to each are also shown on Figure 10.

Key in-situ testing comprised:

- Geophysical surveys to provide V_s profiles of the engineered fill and the foundation strata, for interpretation of the dynamic performance.
- Plate load tests (PLT) at the specific locations to provide a continuous stiffness profile, E_{v2} within the embankment fill, taking in consideration the depth of influence of 1.2 m.
- Nuclear density gauge (NDG), core cutter (CC) and sand replacement density (SRD) to measure field MC , air voids (AV) and DD needed for the assessment of compaction acceptance criteria.

Key laboratory testing included:

- Proctor test to determine the optimum moisture content (OMC).
- Strength tests to determine the strength and stiffness after stabilisation.
- Particle size distribution (PSD) to determine the grading of the chalk fill before cement mixing and after for assessment of the potential effects of pulverisation on the mix.

The testing methods used for the respective tests were specified in the trial specification (ASC, 2022).

5.3.4 Pulverisation

A pulverisation trial was performed at the temporary excavation area to test the effectiveness of the selected pulverisation method intended for the trial embankment construction. The method involved a two-stage pulverisation process. In the first stage, the chalk layer underwent a single pass of rotavation without the addition of cement. This initial step was carried out to break up and loosen the material.

In the second stage, a single rotavation pass was also performed but with cement added to the chalk fill. The rotavation was carefully controlled, maintaining a fixed machine speed and ensuring an overlap equal to the width of the machine's wheel to achieve consistent coverage. The percentage target cement addition rate was verified using tray checks prior to the machine being used for embankment construction. Additionally, the mixing depth was checked before commencing construction of the trial embankment.

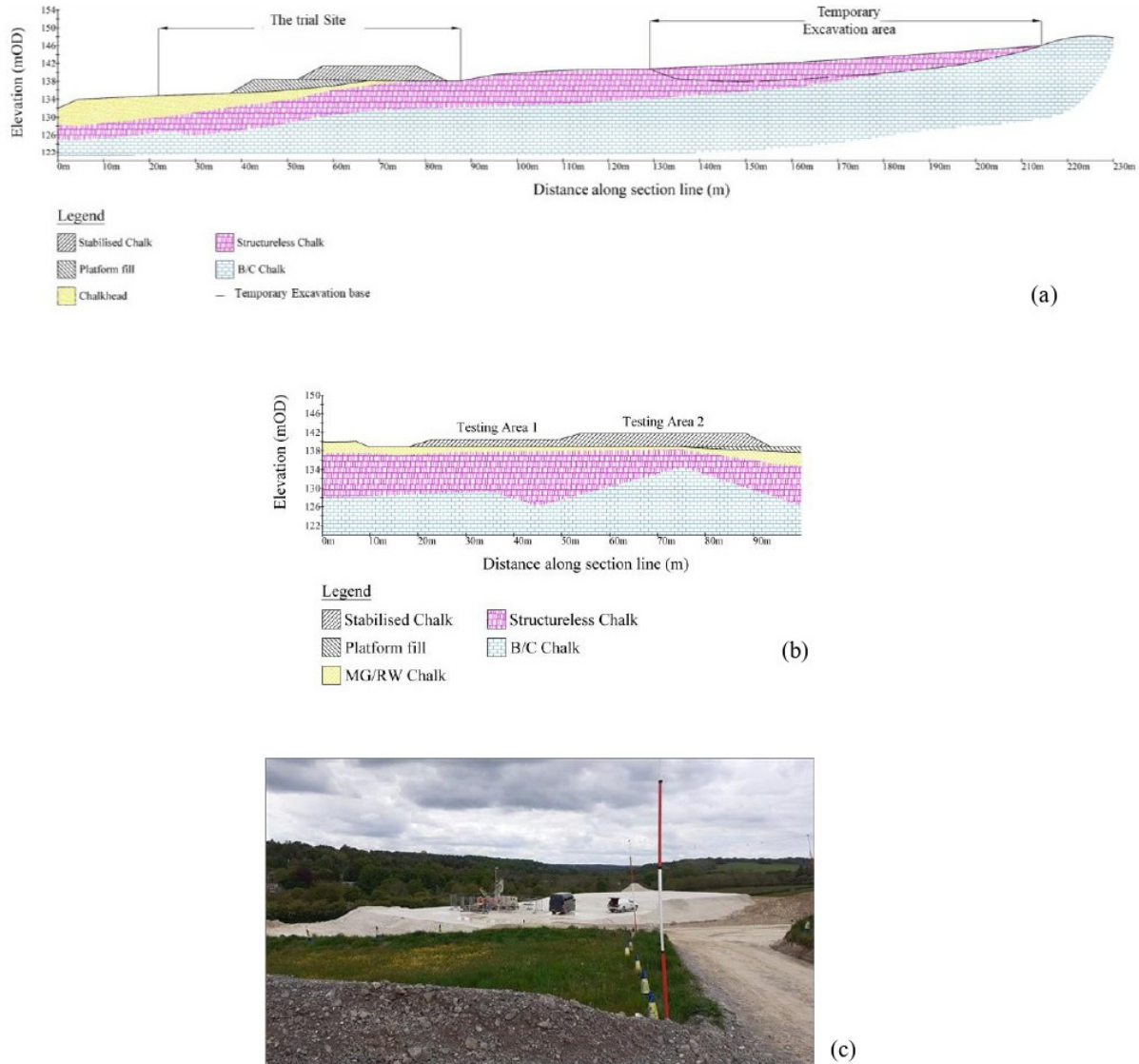


Figure 9. As-built trial embankment sections: (a) Long section showing temporary excavation area and trial site; (b) Long section showing testing Areas 1 & 2; (c) Photo showing completed trial embankment.

6. Analysis and results

6.1 Dynamic performance

Direct (downhole, crosshole) and indirect (continuous surface wave, multi-channel analysis surface wave) geophysical surveys were performed to measure the V_s of the embankment fill and the foundation strata to denote their dynamic stiffness. Downhole (DH) and crosshole (CH) surveys were performed at finished level of layer 8 for Area 1 and layer 13 for Area 2 on completion of the trial embankment construction. Continuous surface wave (CSW) and multi-channel surface wave (MASW) were performed at foundation level and at various layers, as shown in Figure 9.

The recorded V_s results from the different methods are summarised as V_{shv} in Table 2.

The V_{shv} results obtained from the direct and indirect geophysical surveys are also shown as V_s profiles in Figures 11 and 12 for testing Areas 1 and 2, respectively.

The results obtained from the different geophysical testing methods demonstrate that the Class 9M fill consistently exceeded the minimum V_s of 300 m/s, assumed for the embankment fill during design. In addition, based on a detailed analysis of the results, V_s of 470 m/s and 570 m/s are found to reliably represent the dynamic stiffness of the Class 9M fill with 2% and 3% cement content, respectively. These values highlight the significant improvement in dynamic properties achieved by the addition of cement. Thus, they demonstrate the acceptability of the Class 9M fill as a viable material for HSR embankment construction.

Table 2. Summary of V_{shv} obtained from geophysical testing.

CSW (m/s)	MASW (m/s)	DH (m/s)	CH (m/s)
400-500	500-600	400-600*	500-700

*Lower bound value is influenced by the averaging methodology considering the less stiff foundation material below the trial embankment.

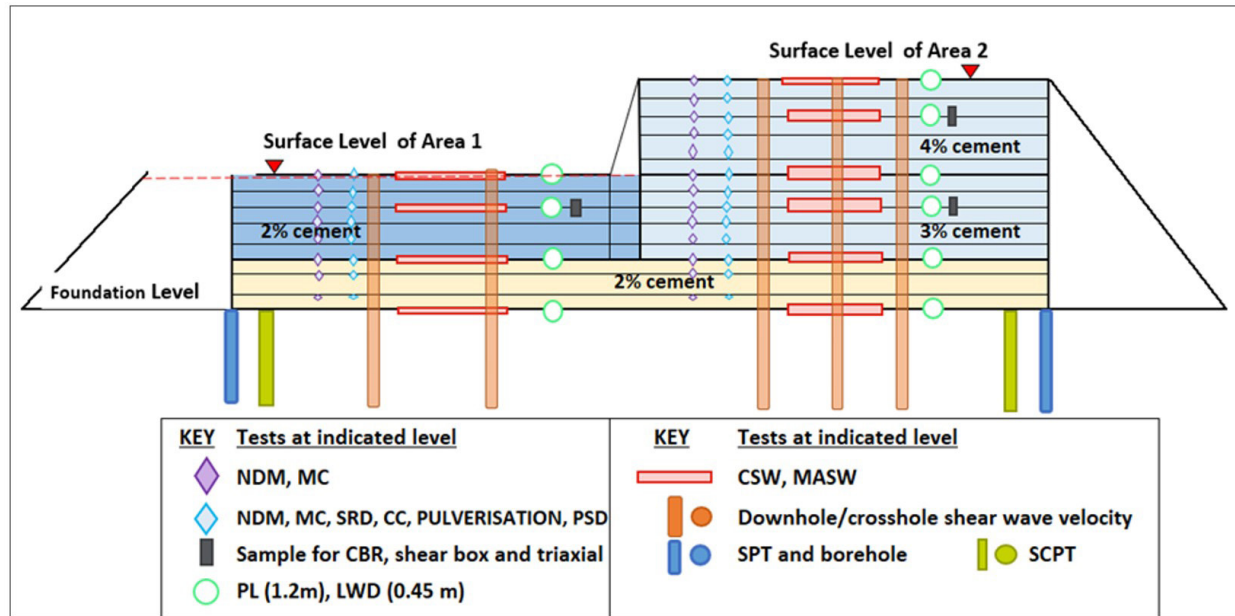


Figure 10. Testing locations and schedule for trial embankment.

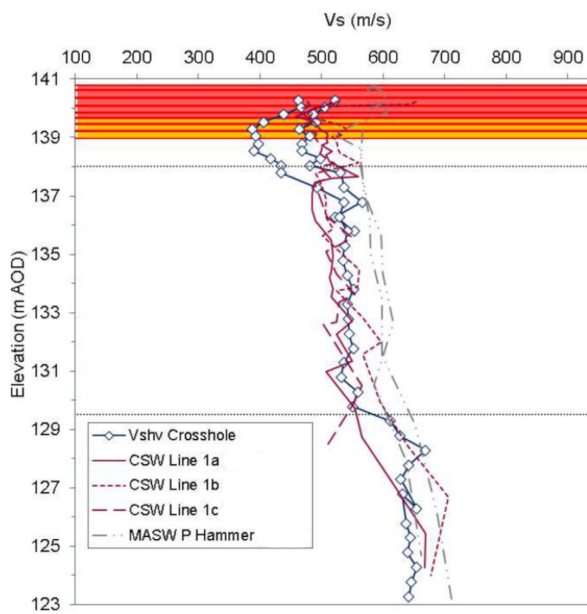


Figure 11. Summary inverted V_s profiles at testing Area 1.

In summary, the testing results confirmed that the structureless Chalk Class 3B1 improved using a minimum of

2% cement mix is acceptable for re-use as HSR embankment fill, since it satisfies the required dynamic performance criteria, characterised by a minimum V_R value, measured as an equivalent dynamic material stiffness V_s .

6.2 Static performance

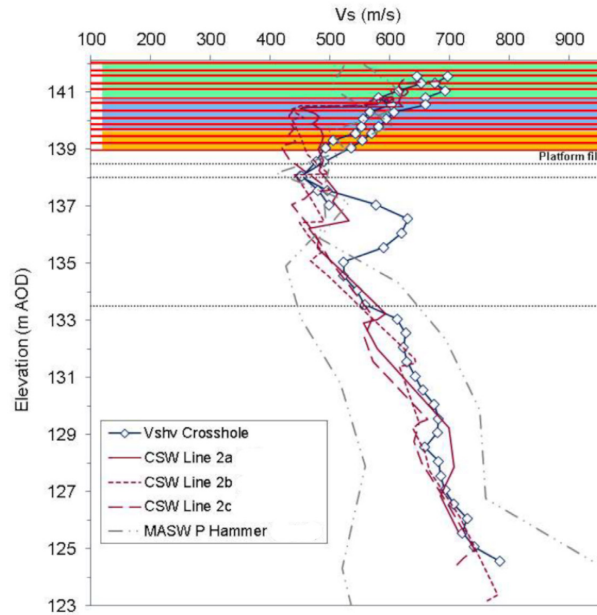
The PLT is considered a reliable method to determine the static stiffness of soils. Considering the depth of influence of 1.2 m for a 600 mm plate used for the testing (AFNOR, 2000), the test locations were carefully selected to ensure that reliable data is obtained for each cement mix. In testing Area 1, seven PLT tests were performed from the finished level of layer 8. In testing Area 2, a total of 14 PLT tests were carried out. Of these, seven tests were undertaken during construction from the finished level of layer 8 and the remaining seven were performed from the finished level of layer 13.

PLT tests were performed 24 hours after completion of fill placement and compaction. Testing was also undertaken between days 18 and 21 after completion of the embankment trial construction to study the potential effect of curing time on the long-term stiffness of the Class 9M fill. The results are presented as a range for each cement mix in Table 3.

Table 3. Summary of E_{v2} results for different cement mix for Class 9M fill.

Cement mix (%)	E_{v2}^+ Range (MPa)	E_{v2}^+ Mean (MPa)	E_{v2}^{++} Range (MPa)	E_{v2}^{++} Mean (MPa)
Dc + 2	65-409	136	65-409	136
Dc + 3	107-375	188	107-375	188
Dc + 4	115-450	244	115-4500	>>500

*Measured after 24hrs; **Measured between 18 and 21 days.

**Figure 12.** Summary inverted V_s profiles at testing Area 2.

The analysis highlights the performance and behaviour of Class 9M fill formed of Dc grade Chalk in relation to static stiffness and the influencing factors of cement content, time, and foundation strata. The findings indicate that the Class 9M fill achieved E_{v2} results exceeding the minimum limiting static stiffness criteria, confirming its suitability as an HSR embankment fill material. A clear trend emerged, showing that the static stiffness of the Class 9M fill improved with higher cement content. However, the rate and extent of improvement were governed by the cement content.

For Class 9M fill with 4% cement, an increase in stiffness was observed over time, suggesting an ongoing stabilization process. In contrast, the 2% and 3% cement mixes demonstrated significant early improvement, with stiffness levels appearing to plateau after 24 hours. This indicates that lower cement content accelerates the stabilisation process, achieving its full effect within a shorter timeframe, while higher cement content results in continued strength gain, and ultimately higher stiffness, over a longer curing period.

Another key observation was the relationship between stiffness and the depth of influence during testing. The stiffness

of the Class 9M fill was greater when the tested depth was entirely within the fill, pointing to the potential impact of the foundation strata on the overall performance of the embankment fill mass. This is further supported by the lower E_{v2} results recorded for the stabilized chalk fill with 2% cement mix, placed on the levelling fill platform. These lower results confirmed the influence of the foundation on the long-term performance of the HSR embankment fill mass.

In summary, based on the testing results, structureless Chalk grade Dc, stabilised with 2 to 3% cement, is considered suitable for use as both UEF and LEF in HSR embankments. Structureless Chalk grade Dm, stabilised with 2% cement, could potentially be used as LEF. However, its suitability would need to be assessed on a case-by-case basis, taking into consideration the embankment height and the specific characteristics of the underlying foundation strata.

6.3 Compaction requirements

6.3.1 Compaction optimisation

Compaction optimisation trials were performed during placement and compaction of layers 1, 2 and 3 of the testing areas to determine a suitable number of passes. Nine NDG records were taken in:

- layer 1 - after 4, 6 and 8 passes
- layer 2 - after 6, 8 and 10 passes
- layer 3 - after 8 and 10 passes

Variable MC was measured in each layer, ranging from an average of 21% in layer 1 to about 22% in layer 2, increasing to 25.5% in Layer 3. This indicated that compaction between layers could not be assessed collectively, as MC skewed the degree of compaction, and the AV achieved. It was also noted that material with MC below 23% showed a reduced performance in AV . Consequently, layers 1 and 2 were excluded from the compaction optimisation assessment.

From a review of the compaction optimisation trial results on layer 3, an increase in compaction was observed between 8 and 10 passes. Based on this, 10 passes using a 19 tons smooth drum roller traveling at < 4 km/h were adopted for the compaction of the remaining layers forming testing Area 1 and testing Area 2 of the trial embankment. Compaction optimisation on layer 3 is presented, as Figure 13.

6.3.2 Assessment of compaction acceptance criteria

Proctor tests were undertaken in the laboratory, using the standard 2.5 kg and the modified 4.5 kg rammer, to determine the *OMC* and maximum dry density (*MDD*). A cement-stabilised chalk fill with 2% binder content was used for the Proctor tests.

The field *DD* values achieved during compaction optimisation trials were compared to the *MDD* values obtained from laboratory Proctor tests. The field *DD* are found to closely match the *MDD* determined in the laboratory from the standard (2.5 kg) Proctor tests. As a result, the compaction of the trial embankment was benchmarked against the *MDD* values from the standard (2.5 kg) Proctor tests.

Also, from a comparative analysis of the Proctor tests, it was observed that grade Dm chalk fill consistently achieved a higher *MDD* than grade Dc chalk fill. The difference between the two grades is approximately 0.04 mg/m³ (or 40 kg/m³). Using the trial Proctor test data, the following *MDD* values were determined:

- For grade Dm Chalk: *MDD* = 1.54 mg/m³
- For grade Dc Chalk: *MDD* = 1.50 mg/m³

The trial Proctor test results are shown on Figure 14.

The degree of compaction (*DC*) of the grades Dm and Dc chalk has been calculated against these respective *MDD* values.

Using the Proctor test results in Figure 13, the *OMC* was derived as 25% for grade Dm chalk and 27% for grade Dc chalk. It should be noted that based on the average *MC* readings recorded on each layer, nearly 50% of the layers forming the trial embankment (i.e. layers 2,4,7 and 8) were dry.

A data screening process was applied during the analysis of the compaction results from the trial embankment construction and testing, filtering out the following data points:

- Datapoints where cement content did not equate to 2%.
- Datapoints on layers where the number of passes of the roller did not equate to 10 passes, as determined from the optimised compaction trial.
- Data points below an *MC* of 23%, as most of this data failed to meet the limiting criteria of *AV* ≤ 5%.
- Data points where NDG recordings were taken at a depth below the 200 mm specified as a minimum for measurement in each layer.

The resulting credible datasets from the embankment trial are presented as a histogram in Figure 15.

Analysis of the refined dataset revealed that 90% of the data points exceeded a degree of compaction (*DC*) value of 98.7%. This finding demonstrates that the *DC* required to achieve the minimum static and dynamic stiffness for Class 9M chalk as an HSR embankment fill is consistent with the minimum acceptance compaction criteria outlined in the HS2 technical standard based on the modified Proctor (refer to Table 1).

A similar screening process was followed during the interpretation of the *AV* results. The credible datasets are presented in Figure 16.

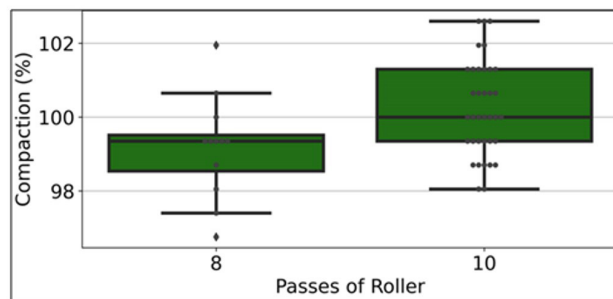


Figure 13. Trial compaction optimisation results.

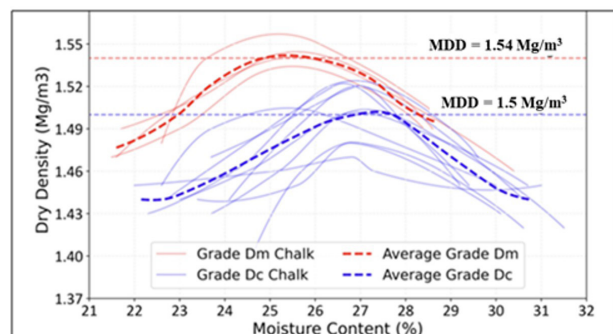


Figure 14. Trial standard Proctor test results.

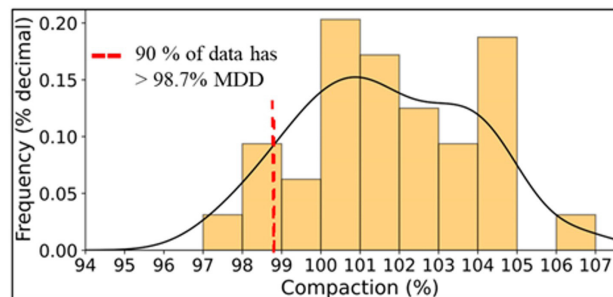


Figure 15. Histogram showing the distribution of *DC* for Class 9M fill.

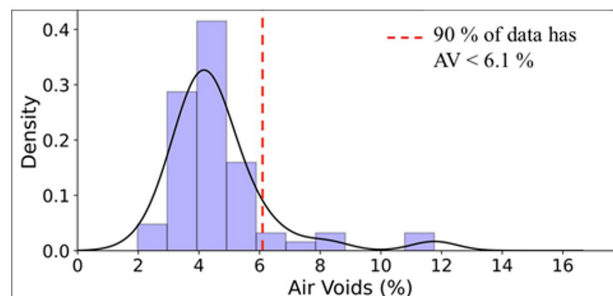


Figure 16. Histogram showing the distribution of *AV* for Class 9M fill.

Assessment of the refined dataset showed 90% have an average AV less than 6.1%. This is higher than the maximum 5% criteria outlined in HS2 technical standard (See Table 1).

The field trial results suggest that achieving the required minimum static and dynamic stiffnesses (E_{V2} and V_R) for a Class 9M chalk fill can be achieved with reduced compaction efforts. Specifically, the results indicate that the compaction criteria for AV can be relaxed by approximately 20%. This finding points to a potential opportunity to optimise the compaction process, reducing the effort and resources needed while still meeting HS2 performance requirements.

6.4 Pulverisation

The chalk pulverisation during construction of the embankment was applied into two stages. During the first stage, placement and pulverisation of the chalk fill occurred



Figure 17. Example of cement mixing of a pulverised chalk.

on the same day. No change in MC was observed, indicating that the material's moisture condition remained stable. In the second stage, cement was added to the pulverised chalk fill to produce the stabilised chalk mix. The results obtained showed that the DoP achieved generally ranged between 39% and 90%, with a discernible trend apparent between the 55 to 80% range.

The analysis of the refined datasets determined that a DoP of 60% is an appropriate and practical target for chalk, and was subsequently recommended for HSR embankments formed using cement-stabilised Class 9M chalk fill.

An example of a pulverised chalk being subject to cement mixing is shown in Figure 17.

6.5 Impact of layer thickness

In accordance with HS2 SCEW (HS2, 2022):

- All fill materials shall be deposited in layers not exceeding 250 mm uncompacted thickness.
- For cement and/or lime-stabilised materials, the upper limit of 250 mm may be increased if field trials demonstrate that thicker layers can still achieve the performance requirements.

The thickness of each of the placed and compacted fill layers was measured during construction of the trial embankment. The thickness was found to range from 134 mm to 287 mm, with six of the 13 layers exceeding 250 mm.

To assess the potential influence of layer thickness on compaction performance, individual compaction readings were normalised by dividing each value by the overall average. This process yielded a single representative normalised compaction value for each compacted layer. These values are plotted against layer thickness, as shown on Figure 18.

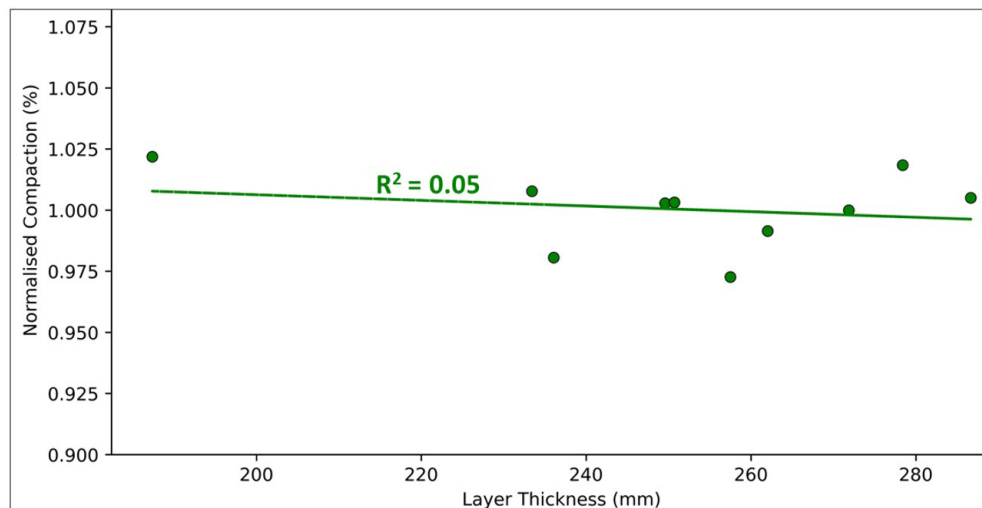


Figure 18. Normalized compaction vs layer thickness.

The results indicated a slight reduction in compaction with increasing layer thickness. However, the decrease was considered minor, as less than 1% reduction was observed across compacted layer thicknesses ranging from 200 mm to 250 mm. This suggests that layer thickness has a relatively minimal impact on compaction performance. This interpretation is supported statistically by the coefficient of determination (R^2) value of just 0.05, indicating a weak correlation between layer thickness and compaction performance. Other factors, such as MC , are likely to exert more significant influence on compaction performance based on evidence from the trial.

6.6 Minimum percent cement content for stabilised fill

The SCEW (HS2, 2022) requires that cement stabilised fills Class 9 must have a minimum of 2% cement content by weight, unless shown to the “*Contracts Engineering Director by field trials that a lesser percentage gives the required performance*”. It has been demonstrated during the trial that structureless Chalk Class 3B1 requires improvement using between 2 to 3% to meet the suitability criteria for re-use as HSR embankment fill.

6.7 Earthwork fills for transition to structures

A robust transition from mainline earthworks to concrete structures is needed. To achieve this, an interface earthwork zone, comprising engineered fill that creates a consistent gradual increase in dynamic response across the transition between the stiffness characteristics of the earthwork and the concrete structure, is required (HS2, 2020). The earthwork fill at the transition to structure is defined as UEF* in the same HS2 standard. As agreed with HS2 during the design development, the minimum static stiffness, E_{v2} , to be achieved for the UEF*, is the lesser of:

- +25% stiffer than UEF; and
- 120 MPa, but never less stiff than UEF.

Based on the high static performance of the Class 9M chalk fill during the trial for 3% and 4% cement content, an opportunity for further optimisation was identified. Structureless Chalk grade Dc mixed with 3 to 4% cement was found to be suitable for use as UEF* in the earthwork transitions to structures.

7. Conclusions

The material field trial embankment results performed as part of the innovative large scale earthworks strategy, developed specifically for the C23 section of HS2, demonstrated that the site-won structureless Chalk grade Dm/Dc (Class 3B1) can be effectively improved using controlled cement mix to produce a cement-stabilised Class 9M chalk fill. The trialed Class 9M fill produced with a cement content of 2 to 3% achieved both the static and dynamic performance requirements set out in the TS-E (HS2, 2019), confirming its suitability for re-use as HSR mainline embankment fill.

Based on the trial results, the following optimisations have been identified and are currently being implemented in the construction of mainline earthworks and earthworks to transitions:

- For mainline embankments (UEF and LEF), the compaction acceptance criterion for the Class 9M chalk fill, in terms of AV , has been relaxed from $AV \leq 5\%$ (prescribed in the TS-E) to $AV \leq 6\%$, as demonstrated by the trial compaction performance.
- Class 9M chalk fill is used as LEF for mainline embankments, with a relaxed compaction acceptance criterion of $DC \geq 95\% MDD$ based on standard Proctor.
- For earthwork transitions to structures, Class 9M chalk fill is used as UEF*, with a specified cement content of 3 to 4% to satisfy the minimum static stiffness criteria set out in HS2 standard for approach to transitions (HS2, 2020).

These optimisations have been used to establish revised compaction performance criteria for the Class 9M chalk fill, as outlined in Table 4.

Furthermore, the following key requirements were identified as needing to be observed during construction to preserve the quality of the structureless Chalk Class 3B1 when producing the cement-stabilised Class 9M chalk fill:

- Only structureless Chalk grade Dc should be used for producing a cement stabilised chalk fill Class 9M for UEF and UEF*.
- Structureless Chalk Class 3B1 should not be stockpiled due to drying effects, reducing the material’s ability to achieve the AV acceptability criteria (TRL, 1992).

Table 4. Revised compaction acceptance criteria for class 9M fill as HSR embankment fill.

Embankment (layer)	Material	E_{v2}	Min. compaction acceptability criteria
		(MPa)	(Modified Proctor)
UEF	Stabilised	≥ 60 at surface	$DC \geq 98\% MDD$
		≥ 45 at base (1m above LEF)	$AV \leq 6\%$
LEF	Stabilised	To be determined from design	$DC \geq 95\% MDD$
			$AV \leq 6\%$

- On classifying the structureless Chalk fill as either a grade Dm (LEF) or grade Dc chalk (LEF/UEF/UEF*), local Proctor testing should be undertaken to determine an appropriate compaction curve and *MDD*, ensuring adequate assessment of relative compaction for the asset-specific earthwork embankment.

The knowledge and learning insights acquired from this novel field trial are expected to be fully reflected in the new guidance being developed for earthworks using stabilised chalk fill, as part of the ongoing revision of the CIRIA C574 report. Thus, it will contribute to the building of a technical legacy for future earthworks projects in the UK and elsewhere.

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Declaration of interest

The authors have no conflicts of interest to declare. All co-authors have observed and affirmed the contents of the paper and there is no financial interest to report.

Authors' contributions

Ouarda Boumendjel-Game: methodology, data collection & curation, data analysis, data interpretation, writing – review & editing, writing – original draft. Harry Saroglou: methodology, data interpretation, review and editing. Matthew Fleetwood: methodology, investigation, data collection. Mike Black: data collection, review & editing.

Data availability

All data produced or examined in the course of the current study are included in this article.

Declaration of use of generative artificial intelligence

This work was prepared without the assistance of any generative artificial intelligence (GenAI) tools or services. All aspects of the manuscript were developed solely by the authors, who take full responsibility for the content of this publication.

List of symbols and abbreviations

<i>AV</i>	Air voids
<i>CC</i>	Core cutter
<i>CH</i>	Crosshole seismic test
<i>CSW</i>	Continuous surface wave

<i>DC</i>	Degree of compaction
<i>DD</i>	Dry density
<i>DH</i>	Downhole seismic test
<i>DoP</i>	Degree of pulverisation
<i>DS</i>	Design speed for HS2
<i>E&R</i>	Excavate and replace
<i>E_{v2}</i>	Vertical deformation modulus
<i>GI</i>	Ground investigation
<i>HS2</i>	High Speed 2
<i>HSR</i>	High-speed rail
<i>IDD</i>	Intact dry density
<i>LEF</i>	Lower embankment fill
<i>MASW</i>	Multi-channel analysis surface wave
<i>MC</i>	Moisture content
<i>MDD</i>	Maximum dry density
<i>NDG</i>	Nuclear density gauge
<i>OMC</i>	Optimum moisture content
<i>PLT</i>	Plate load test
<i>PSD</i>	Particle size distribution
<i>SCEW</i>	Specification for civil engineering works – 600series earthworks
<i>SRD</i>	Sand replacement density
<i>TS-E</i>	Technical standard – earthworks
<i>UEF</i>	Upper embankment fill
<i>UEF*</i>	Upper embankment fill at transition
<i>V_R</i>	Rayleigh Wave Velocity
<i>V_s</i>	Shear wave velocity
<i>V_{shv}</i>	Shear wave velocity from direct and indirect field testing

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A coupled hydromechanical model including fracture propagation along the dam foundation interface for gravity dam's stability assessment

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Article

Keywords

Gravity dam
Hydromechanical coupled model
Fracture propagation
Safety assessment

Abstract

Most gravity dam failures occur due to sliding along the dam/foundation interface, rock mass discontinuities, or rock mass layers of lower strength. The possibility of sliding of a dam is usually evaluated based on simplified limit equilibrium techniques. In this study an explicit time-stepping small displacement algorithm, Parmac2D-Fflow, is used to assess the safety of gravity dams. This algorithm is based on a discrete representation of discontinuities, simulates the hydromechanical interaction, and considers softening constitutive laws that are closer to the actual behaviour of the dam/foundation interface. Seepage flow along the dam/foundation interface is only allowed to occur after contact failure, making it possible to model a coupled propagation failure along the dam/foundation interface due to a hypothetical dam overtopping scenario. For two gravity dams with different heights, the numerical results predicted with a coupled/fracture propagation model are compared with those obtained with a coupled/fully fractured model and with an uncoupled analysis. The results presented highlight the relevance of considering a coupled hydro-mechanical model for dam safety analysis and show that with a coupled-fracture propagation model slightly higher safety factors are predicted.

1. Introduction

Concrete gravity dams are built on rock foundations, and stability analysis involves identifying potential sliding block mechanisms that could lead to failure. The behaviour of the rock mass is primarily governed by sliding either along the dam/concrete interface or along weak, sub-horizontal layers in the foundation, near the base of the dam (Azevedo et al., 2021; Farinha et al., 2022). The geological and geotechnical characteristics of the foundation area play a crucial role in the dam's performance, and seepage studies are commonly conducted to assess the dam's global stability (Santos et al., 2023). These studies are sometimes complemented with sensitivity analysis of dam model outputs (He et al., 2022; Li, 2021).

The hydromechanical behaviour of fractured rock masses is critical and must be carefully considered, with the behaviour of discontinuities represented by realistic constitutive models (Farinha et al., 2022; Zhou et al., 2022). For concrete dam foundations, discontinuum models that simulate hydromechanical interactions offer a valuable approach for assessing dam stability. These models account for actual shear displacements, the aperture of discontinuities, and the water pressure distribution within the foundation.

Regarding the failure mechanism that involves sliding and the possibility of crack formation and propagation along the dam base, studies have been performed to investigate the interface properties and associated failure processes between dam concrete and rock (Dong et al., 2016, 2024; Li et al., 2022; Tian et al., 2015) and its effect on the stability of dams (Krounis et al., 2015).

This study uses a coupled hydromechanical model to evaluate the stability of gravity dams, accounting for the opening and closing behaviour of discontinuities, as well as the influence of water pressure within the foundation joints and along the dam/foundation interface. The model incorporates interface elements that simulate a reduction in cohesion and tensile strength at both concrete/concrete and concrete/rock interfaces. Stability assessments are performed on two gravity dams of varying heights. The safety factors against sliding are calculated using a gradual load amplification approach, applying a softening behaviour to joint elements that simulate the discontinuities (concrete/concrete and concrete/rock) (Azevedo et al., 2010) and (Azevedo & Lemos, 2022). Two different assumptions for the behaviour of the foundation rock mass are considered: either linear elastic or brittle failure. A critical evaluation of the results is provided in the context of sliding safety.

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2. Numerical modelling approach

2.1. Coupled hydromechanical model

This study was carried out with *Parma2D-Fflow* (Farinha et al., 2017; Azevedo et al., 2021), which allows fully coupled hydromechanical analysis. The mechanical domain is divided into blocks, which are internally divided into a mesh of plane triangular elements to simulate their deformability. The interaction between these deformable blocks, which simulate both the dam and the foundation, is done using joint finite elements under a small displacement hypothesis (Figure 1a). The hydraulic model is perfectly superimposed on the mechanical model and thus for each joint element (JE) there is a corresponding seepage channel (SC), as shown in Figure 1b), with a hydraulic node (HN) at each end of the seepage channel.

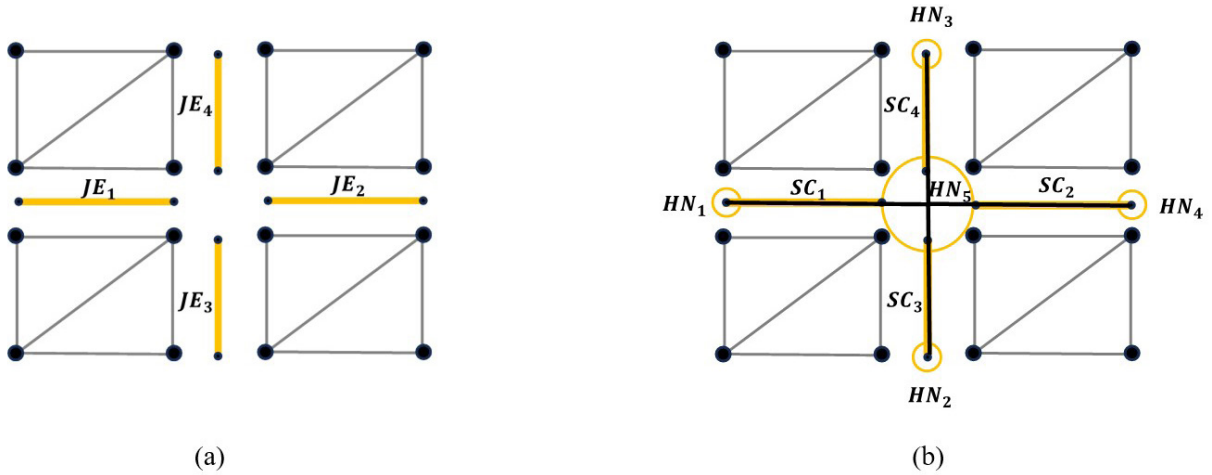


Figure 1. Hydraulic and mechanical models superimposed: (a) Mechanical model joint elements (JE); (b) hydraulic model nodes (HN) and seepage channels (SC).

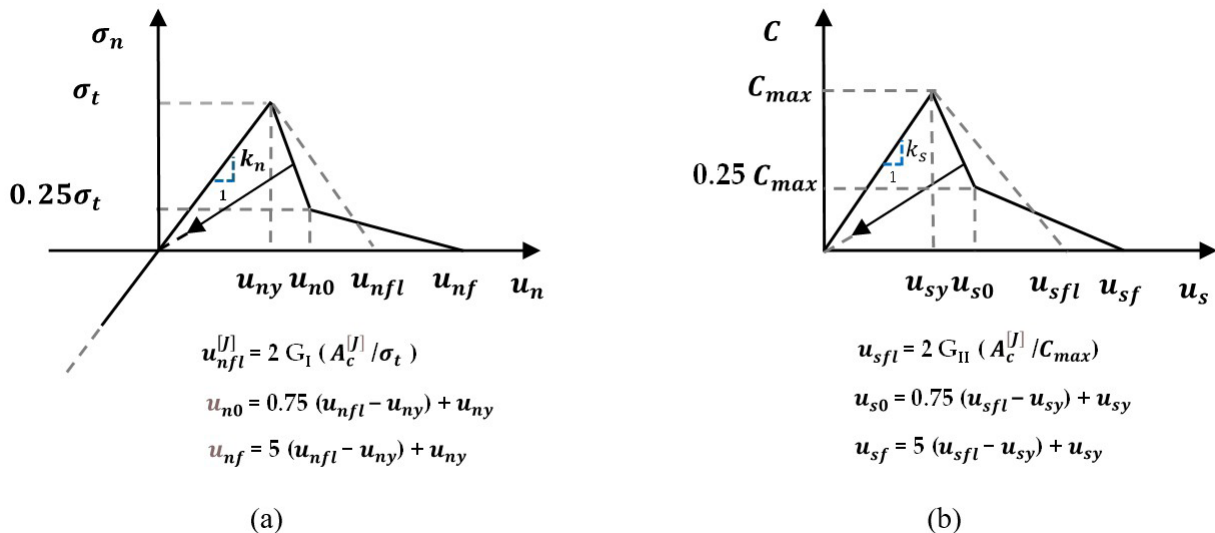


Figure 2. Bilinear vectorial-softening interface model (BL): a) normal direction; and b) shear direction.

3. Stability analysis of gravity dams

3.1. Model description

3.1.1 Geometry and boundary conditions

This study examined four different models, each representing one of two large gravity dams with varying

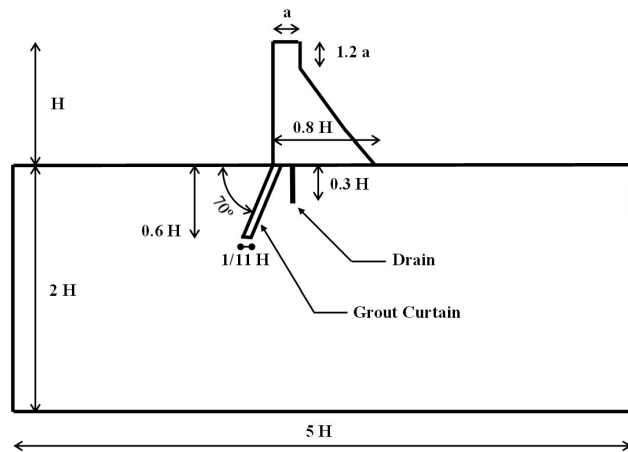


Figure 3. Dam/foundation system model geometry.

heights (15 m and 30 m), with their foundations on rock masses characterized by two distinct fracture patterns. Figure 3 shows the schematic layout of these models, where all dimensions are expressed in relation to the dam height (H) and the top thickness of the dam (a).

As shown in Figure 4, the construction joints within the concrete dam structure were simulated using a set of horizontal discontinuities, with a spacing of 3.0 m, and the foundation rock masses were represented with different sets of discontinuities:

- The first model, called “reg”, includes a set of continuous horizontal discontinuities spaced 5.0 m apart, along with a second set of vertical cross-joints. These vertical joints have an average spacing of 5.0 m perpendicular to the joint traces, with a standard deviation from the mean of 1.0 m.
- The second model, called “dip”, features two sets of continuous orthogonal discontinuities that make angles of 15° and 105° with the horizontal.

It is important to highlight that in the “dip” model, the alignment, and the continuity of the two sets of flat discontinuities in the foundation promote the development of potential failure mechanisms, particularly the formation of sliding wedges beneath the dam.

In addition to the sets of discontinuities, Figure 4 also shows the internal mesh of both dam and foundation blocks, composed of triangular plane finite elements.

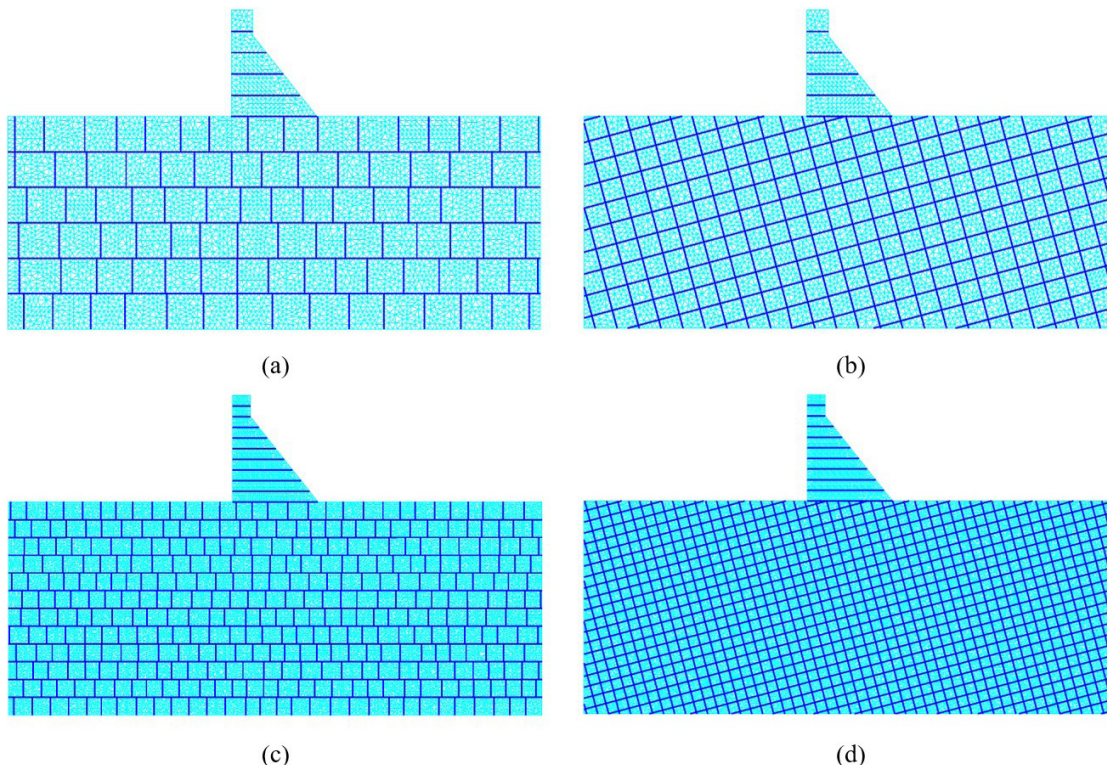


Figure 4. Dam/foundation system discontinuous models: (a) model 15-reg; (b) model 15-dip 15 m high gravity dam; (c) model 30-reg; (d) model 30-dip 30 m high gravity dam. In each block a triangular plane finite element discretization is adopted.

Figure 5 shows the hydraulic part of the models, based on the assumption that water seepage occurs exclusively along the dam/foundation interface and through the network of discontinuities within the foundation.

Table 1 shows the number of mechanical and hydraulic elements included the four different models. It should be mentioned that, for both dam heights, the average edge length of the triangular elements is 0.6 m.

Regarding the mechanical boundary conditions, the bottom of the model was restrained and horizontal displacements on the lateral boundaries were restricted. For the hydraulic boundary conditions, impermeability was assumed at both the base and the sides of the model, and the drainage system was simulated by assigning a water pressure along the drain axis equal to 1/3 of the hydraulic head upstream from the dam.

3.1.2 Material properties

The dam concrete and the foundation rock blocks are modelled with elastic behaviour, with the material properties shown in Table 2 which include Young's modulus (E),

Poisson's coefficient (ν), and density (ρ). In the concrete region, joints are assigned a normal stiffness (k_n) of 60.6 GPa/m and a shear stiffness (k_s) of 24.2 GPa/m. For both the dam/foundation interface and the discontinuities within the rock mass, the stiffness values are $k_n = 120.8$ GPa/m and $k_s = 51.5$ GPa/m. When non-linear behaviour is considered in the rock mass discontinuities, the model assumes zero cohesion and tensile strength, with a friction angle of 45° .

For the hydraulic apertures of the seepage channels, the following values were used: $a_0 = 1.668 \times 10^{-4}$ m, $a_{res} = 1/3 \times a_0$ and $a_{max} = 5 \times a_0$ (a_0 is the joint aperture at nominal zero normal stress, a_{res} is the residual aperture and a_{max} is the maximum hydraulic aperture). The permeability of the dam/foundation interface was assumed to be half that of the discontinuities within the foundation. The grout curtain was modelled with a permeability 2.5 times lower than that of the surrounding rock. The construction joints within the dam were treated as impermeable. Table 3 shows the contact model parameters of the adopted contact interface bilinear softening constitutive law.

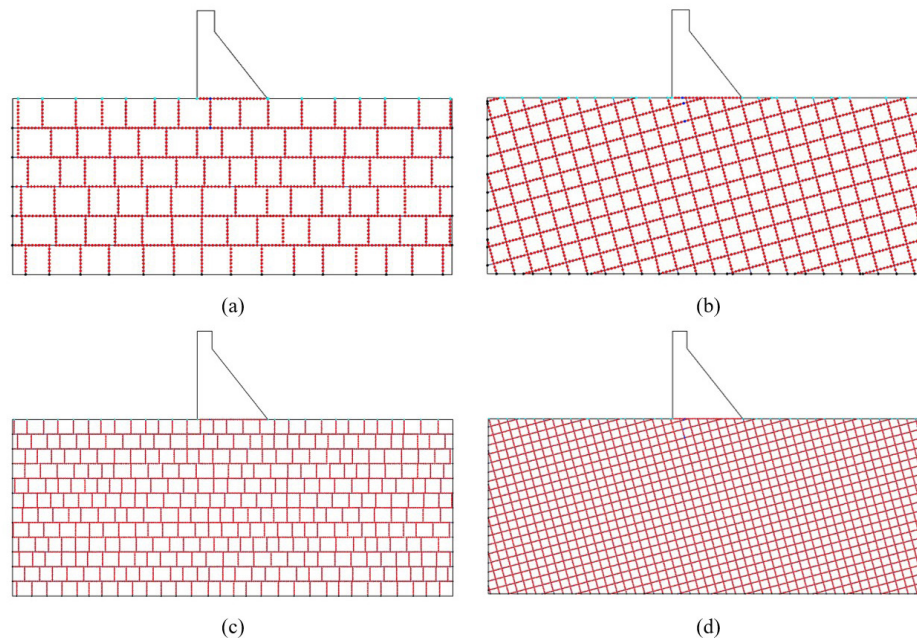


Figure 5. Foundation hydraulic models. Hydraulic head assumed on the rock mass surface upstream and downstream from the dam (light blue dots); hydraulic head assumed at the drainage system (dark blue dots); hydraulic channels (red dots); impervious lateral and bottom model boundaries (black dots): (a) model 15-reg; (b) model 15-dip; (c) Model 30-reg; (d) model 30-dip.

Table 1. Number of different elements in each model.

Model	a^*	b^*	c^*	d^*	e^*
15-reg	6531	9629	1420	1322	1379
15-dip	8063	10058	2550	2299	2509
30-reg	25883	38549	5843	5367	5659
30-dip	31784	40178	10221	9115	10037

* a : mechanical nodal points; b : triangular elements; c : joint elements; d : hydraulic nodal points; e : seepage channels.

Table 2. Material mechanical properties (after Dong et al., 2016).

Material	E (GPa)	ν (-)	ρ (kN/m ³)
Dam concrete	30.3	0.24	2400
Rock mass	64.4	0.20	2700

Table 3. Softening model parameters for the two different discontinuities: concrete/rock and concrete/concrete.

σ_t (MPa)	G_I (Nm/m ²)	C_{max} (MPa)	G_{II} (Nm/m ²)	μ_c (-)
2.9	250	8.0	2500.0	1.0

σ_t : maximum tensile strength; G_I : fracture energy in mode I; C_{max} : maximum cohesion stress; G_{II} : fracture energy in mode II; μ_c : friction coefficient.

3.2 Stability assessment

The models developed were used to evaluate the sliding safety of the dam/foundation system, considering the possibility of sliding occurring not only at the dam/foundation interface but also within the discontinuities of the foundation rock mass. The analysis was performed in two stages. In the first stage, a hydromechanical calculation was carried out, taking into account the weight of the dam, assuming that the water table was at ground level with an initial in situ horizontal-to-vertical effective stress ratio of 0.5. In the second stage, the hydrostatic pressure was applied at the upstream face of the dam and at the base of the reservoir, under the assumption that all the discontinuities exhibited linear elastic behaviour. This second stage involved a mechanical analysis, followed by a hydromechanical analysis. In cases where non-linear behaviour of the discontinuities was assumed, this was only considered after equilibrium had been reached. Throughout all simulations, the reservoir was assumed to be at the dam crest level.

For each gravity dam the results obtained using a coupled model with fracture propagation were compared to those from a coupled model assuming fully developed fractures, as well as to an uncoupled analysis in which water pressures remained fixed at the values corresponding to the initial equilibrium state (no hydraulic computation was performed in this model). In the coupled simulations, the hydraulic head upstream from the dam was gradually increased given the water height above the dam crest. In the fracture propagation model, seepage along the dam/foundation interface was permitted only after damage had initiated in the corresponding joint elements.

The safety of concrete dams is commonly expressed through a factor of safety, which can be determined either by following a strength reduction of the discontinuities involved in potential failure mechanisms (Farinha et al., 2017) or by increasing the hydrostatic load on the upstream face of the dam (Azevedo et al., 2021; Enzell et al., 2023), the latter being adopted in this work.

Figure 6 shows the hydraulic head contours for the 30-reg model assuming a brittle model for the foundation behaviour, for the case where seepage is considered along the dam foundation interface since the beginning of the hydromechanical analysis and for the case where seepage along the dam/foundation interface is just allowed after damage initiation at each joint element. As shown, at the vicinity of the dam foundation the hydraulic head is slightly different on both calculation hypothesis, but outside this area the hydraulic head distribution is quite similar. On both models the effect of the drainage system on the hydraulic head is clearly visible. Figure 7 shows the distribution of water pressures along the base of the dam obtained for the two seepage assumptions. As shown, at the beginning of the load amplification calculation there is no damage at the dam/foundation interface joint elements (zero pressure), whereas in the scenario where seepage is allowed from the beginning of the calculations the predicted pressure distribution is closer to the bi-linear uplift distribution that is usually assumed in stability design.

Figure 8 and Figure 9 show the horizontal crest displacement during the process of amplification of water pressure for both the 15 m and 30 m high gravity dams. For the four different models the load amplification analysis was carried out assuming three different hypotheses: uncoupled (UCP), coupled (CP) and coupled behaviour with fracture propagation due to water pressure (CP-FP) and two different foundation behaviours: elastic (E) and brittle model (B).

As shown in Figures 8 and 9, for both dam heights, with a fractured foundation following a brittle model a higher crest displacement is predicted when compared with an elastic foundation. In addition, with a brittle foundation the difference in behaviour when load coupling is considered (CP, CP-FP) is more noticeable when compared with the uncoupled models (UCP), where the pressures remained constant and equal to those obtained in the initial equilibrium situation.

For the Model 15-reg the CP hypothesis leads as expected to the lowest water height above the crest. For the Model 15-dip the CP hypothesis also leads to the lowest water height

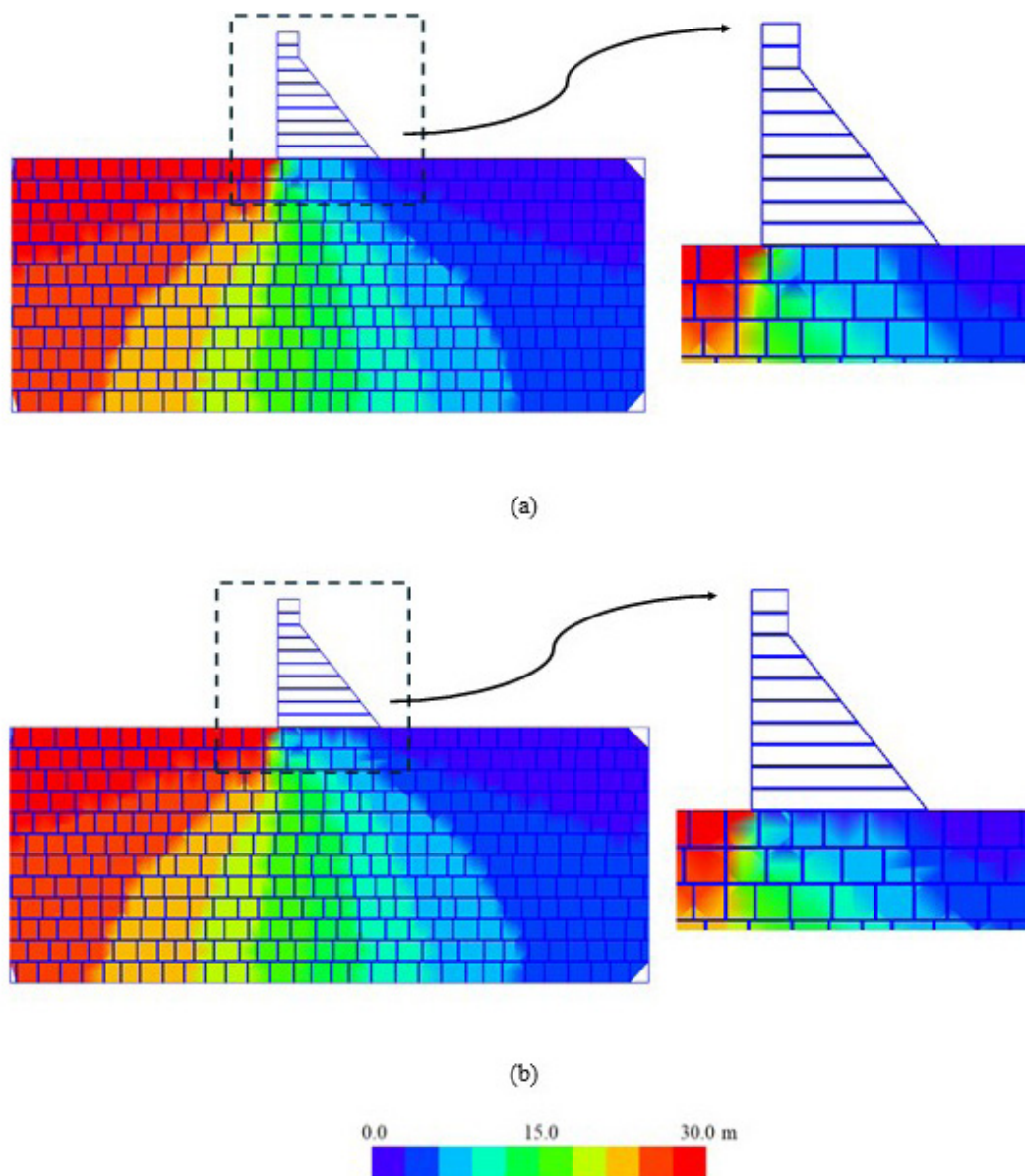


Figure 6. Hydraulic head contours: model 30-reg for a foundation with a brittle model: (a) seepage is considered along the dam foundation since the beginning of the hydromechanical analysis (S); (b) seepage along the dam/foundation interface is just allowed after damage initiation (S-FP).

above the crest, but for a fracture foundation, the predicted water height is similar to the water height predicted with a CP-FP hypothesis. When an elastic behaviour is assumed for the foundation, the CP-FP hypothesis predicts a water level above the crest higher than the value predicted with a UCP approach. When a fractured behaviour is assumed for the foundation, the UCP hypothesis predicts a water level above the crest higher than the value predicted with a CP-FP approach. A similar behaviour is predicted for the 30 m dam height. The main difference occurs in the Model 30-dip for a fractured foundation, where a slightly higher water level above the crest is predicted with the CP model when

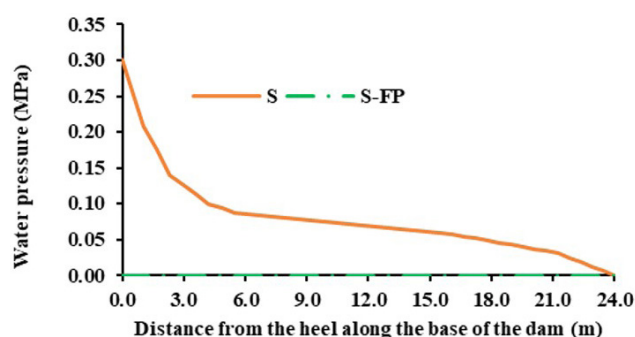


Figure 7. Water pressures along the dam/foundation interface: model 30-reg for a foundation with a brittle model.

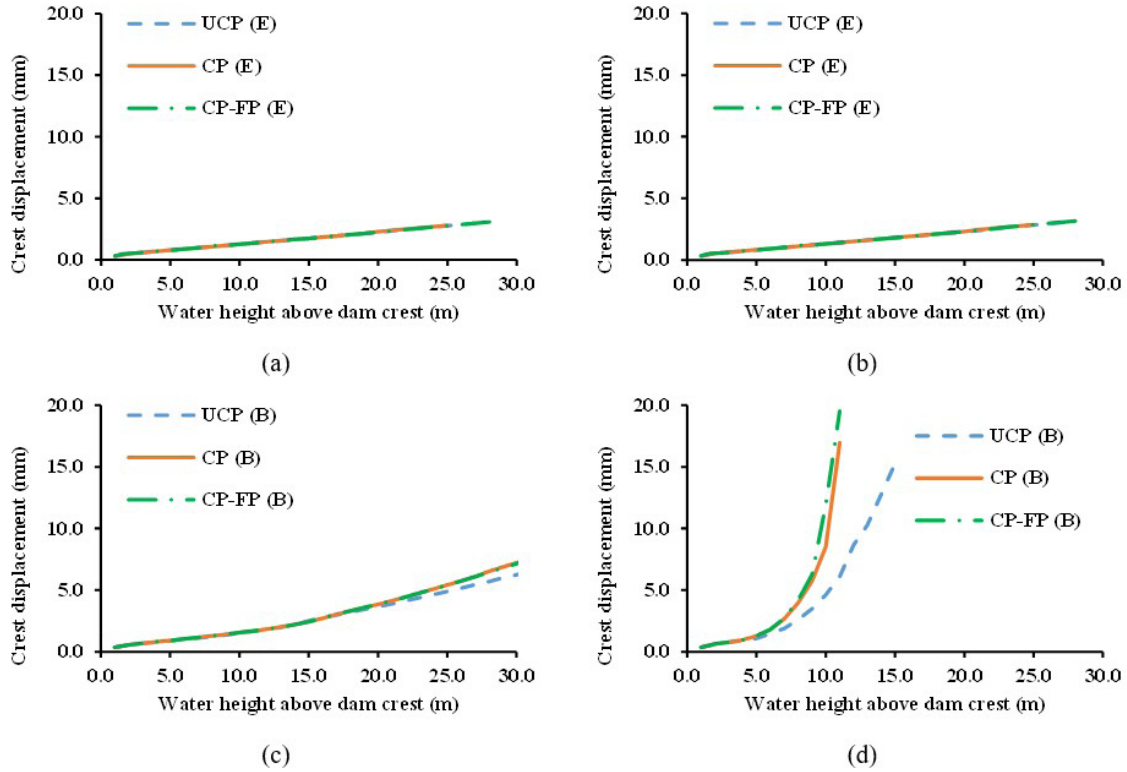


Figure 8. Horizontal crest displacement during the process of amplification of water pressure for: (a) model 15-reg (E); (b) model 15-dip (E), Model 15-reg; (c) model 15-reg (B); (d) model 15-dip (B).

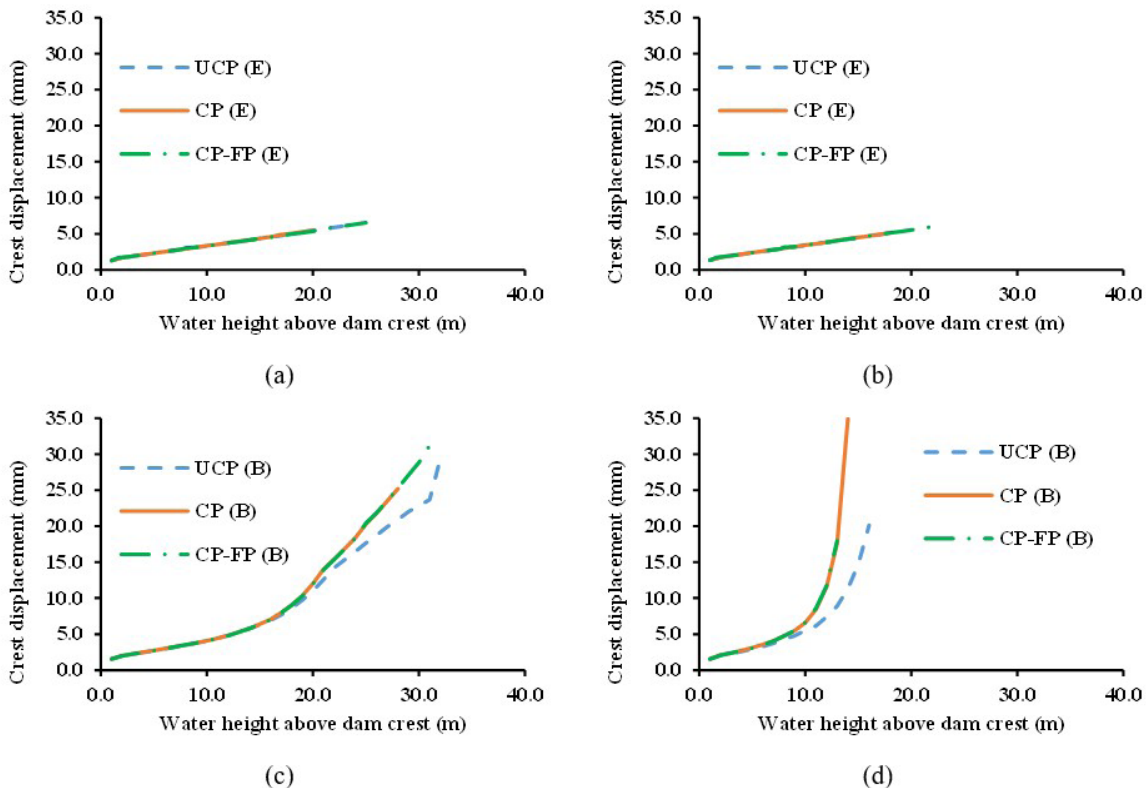


Figure 9. Horizontal crest displacement during the process of amplification of water pressure for: (a) model 30-reg (E); (b) model 30-dip (E), Model 30-reg; (c) model 30-reg (B); (d) Model 30-dip (B).

compared with the water level predicted with the CP-FP hypothesis, this is mainly due to the fact that seepage at the dam/foundation occurs due to damage only at the highest water value which originates additional difficulties to converge to a stable equilibrium position.

Table 4 shows the safety factors calculated for the four different models based on numerical analyses performed under three different assumptions: uncoupled (UCP), coupled (CP) and coupled with water pressure-induced fracture propagation (CP-FP), and two different foundation behaviours: elastic (E) and brittle model (F).

Assuming an elastic foundation model (E) the safety factor obtained with the simplified uncoupled model (UCP) is always higher than that obtained with a coupled model (CP), in which a permeable dam/foundation interface is considered from the beginning of the load amplification process. When using the coupled model that incorporates fracture propagation (CP-FP) along the dam–foundation interface, the resulting safety factors are, as anticipated, higher than those obtained with the standard coupled model (CP) and also exceed those derived from the simplified uncoupled model (UCP).

Assuming a fractured foundation following a brittle model (B) the safety factor calculated with the simplified uncoupled model (UCP) is always the highest. The coupled model that includes fracture propagation (CP-FP) along the dam/foundation interface leads to a higher safety factor

compared with the coupled model (CP), which assumes the interface is permeable from the start of the load amplification process, except for model 30-dip.

As expected, for a brittle foundation (B) in which the orientation of the discontinuity sets favours the formation of failure mechanisms (15-dip and 30-dip), the predicted safety factors are, for all analysis assumptions, consistently lower than those obtained for the models with a regular foundation (15-reg and 30-reg).

Overall, the results here presented indicate that with an uncoupled model (UCP) the calculated safety factor may be higher than that obtained with a coupled fracture propagation model (CP-FP). The results presented also show that the coupled model (CP), that considers the dam/foundation interface permeable from the start of the safety factor calculation, leads to a safety factor equal or lower than that predicted with the coupled fracture propagation model (CP-FP), except for model 30-dip.

Figure 10 shows the expected predicted sliding failure mode for the 30 m high dam assuming an elastic foundation for the coupled model taking into account fracture propagation (CP-FP). A similar failure is predicted with the UCP and CP models. For both foundation geometries the failure mode is mostly due to the sliding of the dam at the concrete/rock interface with a less significant rotation about the downstream foot, which is slightly more noticeable at the onset of failure for the coupled approaches (CP, CP-FP).

Table 4. Safety factors: elastic foundation (E) and fractured foundation (F).

Model	15-reg	15-dip	30-reg	30-dip
UCP (E)	4.60	4.73	2.53	2.27
CP (E)	4.33	4.33	2.33	2.20
CP-FP (E)	4.87	4.87	2.67	2.47
UCP (F)	5.80	3.00	3.13	2.07
CP (F)	5.13	2.47	2.87	1.93
CP-FP (F)	5.67	2.47	3.07	1.87

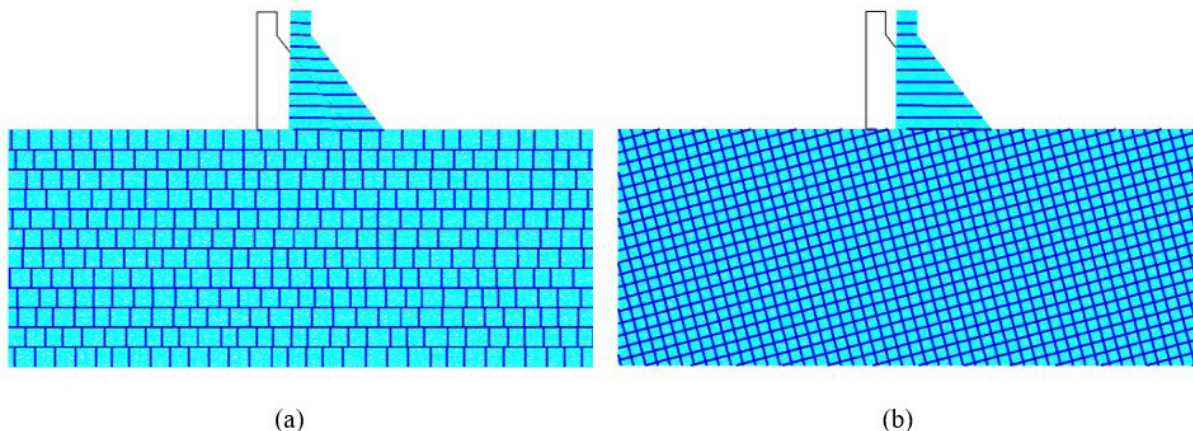


Figure 10. Failure modes for the 30 m high dam with elastic foundation for the maximum load amplification factors: (a) model 30-reg (CP-FP), displacement magnified 2.5 times; (b) model 30-dip (CP-FP), displacement magnified 5.0 times.

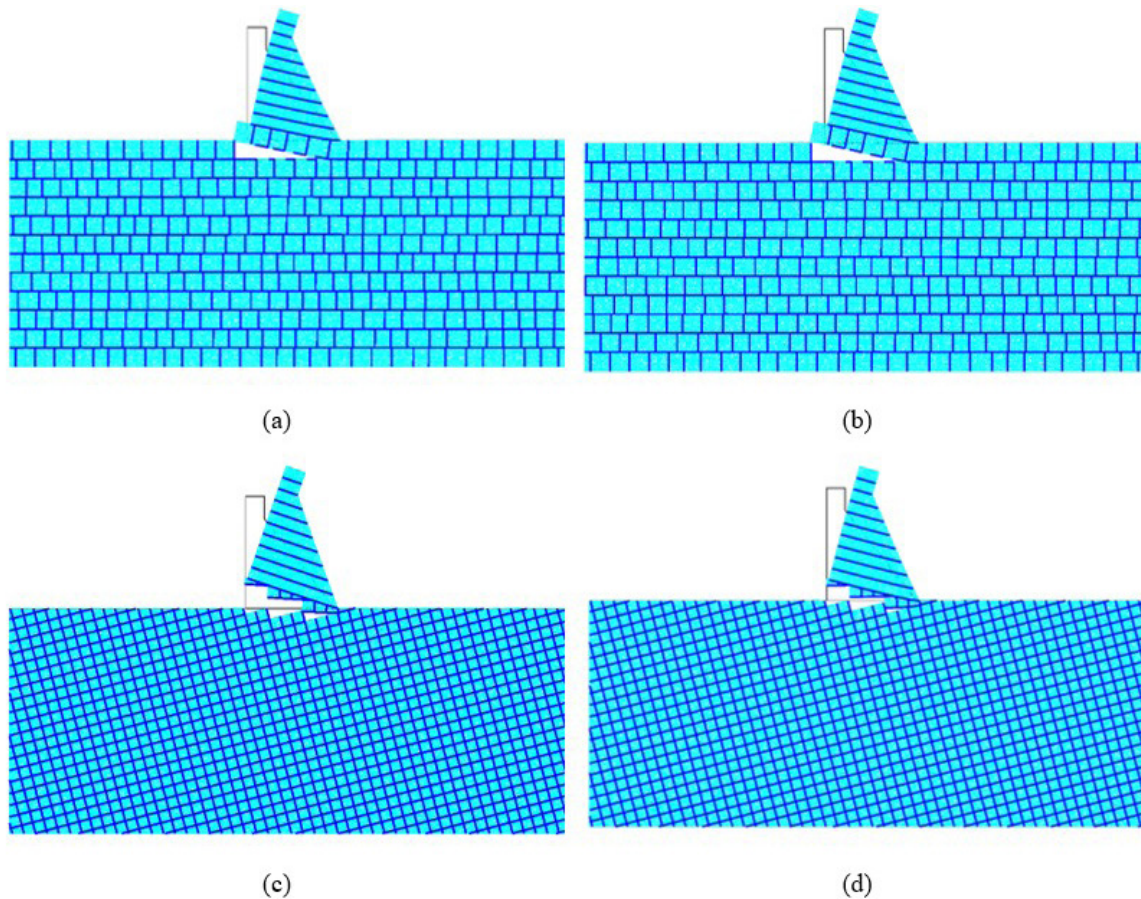


Figure 11. Failure modes for the 30 m high dam with brittle foundation for the maximum load amplification factors: (a) model 30-reg (CP), displacement magnified 300 times; (b) model 30-reg (CP-FP), displacement magnified 350 times; (c) model 30-dip (CP), displacement magnified 75.0 times; (d) model 30-dip (CP-FP), displacement magnified 250 times.

Figure 11 shows the predicted failure modes for a fractured foundation, for the 30 m high dam using both the coupled model (CP) and the coupled model taking into account fracture propagation (CP-FP). Under both hypothesis and dam geometries the predicted failure modes are more complex than the sliding failure mode shown in Figure 10, as the sliding of the dam is accompanied with a more meaningful rotation at the toe of the dam and failure mode involves opening of discontinuities in the foundation rock mass near the heel of the dam.

As shown in Figures 10 and 11, the failure modes obtained, for this set of contact strength parameters, dam heights, and foundation geometries, with the CP and CP-FP approaches are similar. This means that although in the CP-FP approach water seepage only takes place as crack occurs, the fracture propagation influence is not meaningful in the final failure mode.

4. Conclusion

The safety evaluation of gravity dams should follow the proposed framework that considers a load amplification

approach using a coupled hydromechanical model, material properties closer to the real material response and foundation geometries and behaviours as close as possible to the reality. Compared to the traditional strength reduction approach, typically used in foundation design, and referenced in current dam safety design guidelines, this strategy yields more realistic safety factors and failure mechanisms. In the context of coupled analyses, assuming an initially impervious dam/foundation interface may offer a more accurate representation of actual conditions. The results presented show that slightly higher safety factors are predicted when fracture propagation is accounted for during the load amplification process.

The outcomes of safety assessments for existing dams under overtopping conditions, whether caused by floods exceeding design expectations or by the effects of climate change, and under seismic loading, can support the establishment of alert thresholds for use in predictive models. The proposed methodology can be adopted to identify and anticipate potential failure scenarios, and additionally it can be used in the development of a comprehensive failure analysis database.

Ongoing research is focused on evaluating the performance of the coupled model with fracture propagation across a wide range of material properties, including those for concrete, rock mass and corresponding interfaces, as well as different material behaviours, including nonlinear behaviour at both the dam and foundation areas, dam geometries and the consideration of a downstream rock wedge that enhance dam stability.

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Declaration of interest

The authors have no conflicts of interest to declare. All co-authors have observed and affirmed the contents of the paper and there is no financial interest to report.

Authors' contributions

Maria Luísa Braga Farinha: conceptualization, methodology, validation, supervision, project administration, writing – original draft, writing – review & editing. Nuno Monteiro Azevedo: conceptualization, software, methodology, validation, supervision, project administration, writing – original draft, writing – review & editing. Sérgio Oliveira: conceptualization, validation, software, writing – review & editing.

Data availability

The datasets generated analyzed in the course of the current study are available from the corresponding author upon request.

Declaration of use of generative artificial intelligence

This work was prepared without the assistance of any generative artificial intelligence (GenAI) tools or services. All aspects of the manuscript were developed solely by the authors, who take full responsibility for the content of this publication.

List of symbols and abbreviations

a_0	Joint aperture at nominal zero normal stress
a_{max}	Maximum hydraulic aperture
a_{res}	Residual aperture
k_n	Joint normal stiffness
k_s	Joint shear stiffness
B	Brittle model

C_{max}	Maximum cohesion stress
CP	Coupled model
CP-FP	Coupled model with fracture propagation
E	Elastic model
E	Young's modulus
G_I	Fracture energy in mode I
G_{II}	Fracture energy in mode II
H	Height of the dam
HN	Hydraulic node
JE	Joint Element
SC	Seepage channel
UCP	Uncoupled model
μ_c	Friction coefficient
ν	Poisson's ratio
ρ	Density
σ_t	Maximum tensile strength

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Estimation of the permeability coefficient of recycled concrete aggregates using machine learning algorithms: AdaBoost and artificial neural networks

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Article

Keywords

Permeability coefficient
Machine learning
Recycled concrete aggregate
AdaBoost
Artificial neural network

Abstract

The development of the construction sector in recent decades and the systematic reduction of natural aggregate (NA) resources have obliged the search for alternatives and substitutes for these essential construction materials. Using anthropogenic aggregates as replacements for natural aggregates provides an opportunity to reduce landfills by recycling waste materials from construction and demolition (CDW), for civil engineering applications. It may also involve risks that undoubtedly include estimating geotechnical parameters based on solutions developed to date for natural aggregates. A significant factor affecting these estimates is that anthropogenic aggregates could have different properties and chemical compositions. This study analyzed the possibility of estimating the coefficient of permeability for recycled concrete aggregate (RCA) using machine learning algorithms - AdaBoost and artificial neural networks. The AdaBoost model demonstrated higher accuracy (R^2 was 0.959 for training data and 0.886 for test data), relative to neural networks (R^2 was 0.682 and 0.640). Shapley Additive Explanations (SHAP) analysis, which increased the interpretability of the results, showed that AdaBoost better reflected the influence of hydraulic properties such as gradient or bulk density, while the neural networks were more sensitive to aggregate grain size. The results indicate the effectiveness of machine learning algorithms in evaluating the coefficient of permeability for recycled materials and represent a promising perspective for supporting the design of sustainable construction solutions.

1. Introduction

Sustainability in the construction sector is becoming increasingly crucial as environmental concerns, regulatory policies, and resource limitations drive the industry toward more eco-friendly solutions. The construction sector is one of the largest consumers of natural resources and simultaneously generates significant waste, making the transition to a circular economy essential (Cassiani et al., 2021; Vieira et al., 2016). One key strategy for achieving this transformation is reusing construction materials, particularly recycled concrete aggregates (RCA), which substitute natural aggregates in various engineering applications. The utilization of RCA helps mitigate the environmental impact of construction activities by reducing landfill waste, minimizing the depletion of natural aggregate sources, and lowering transportation emissions (Danish & Mosaberpanah, 2022; Stengel & Kustermann, 2024; Yadav & Kumar, 2024). However, challenges remain in fully integrating RCA into construction applications due to

variations in its physical and mechanical properties, including its increased porosity and potential impact on concrete strength (Dash et al., 2023; Hatungimana et al., 2020).

The history of RCA utilization dates back to the post-World War II reconstruction era, when Europe faced a significant shortage of raw materials, prompting the need to repurpose demolition waste (Buck, 1973). Today, the process of producing RCA involves crushing concrete from demolished structures, removing impurities such as rebar, bricks, and soft materials like wood, glass, or polystyrene, and grading the resulting material into fractions ranging from 0 to 63 mm (Oikonomou, 2005). RCA's mechanical and hydraulic properties, including density, moisture content, porosity, water absorption, grain size distribution, and filtration characteristics, determine its suitability for specific applications. The permeability coefficient is significant in road construction and dam engineering, as it influences drainage capacity and the material's ability to control water flow within infrastructure layers (Elhakim, 2016; Sas et al., 2019; Wang et al., 2008).

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Life cycle assessments (LCA) comparing 100% recycled aggregate concrete with conventional concrete indicate that RCA can significantly reduce environmental impact (Stengel & Kustermann, 2024). Studies have shown that incorporating RCA into construction can lower greenhouse gas emissions, reduce the need for raw material extraction, and decrease the amount of construction and demolition waste sent to landfills (Moccia et al., 2022). Nevertheless, despite its environmental benefits, RCA is not yet widely used in structural concrete due to its strength and durability concerns. Increased porosity and residual cement paste on RCA particles can reduce compressive strength, making it necessary to explore methods for enhancing its mechanical performance (Dash et al., 2023). Construction authorities recognize the potential of RCA, and its adoption is encouraged through sustainability-driven policies aimed at reducing construction waste and promoting circular economy initiatives (Nedeljković et al., 2021).

One of the primary challenges in expanding the use of RCA is the lack of effective characterization methods that can accurately predict its mechanical, physical, and filtration properties. Traditional laboratory and field tests for determining the permeability coefficient are expensive and time-consuming, limiting their feasibility for large-scale applications. Machine learning (ML) offers a promising alternative by enabling the prediction of permeability characteristics based on easily measurable physical properties, reducing the reliance on costly experimental procedures (Dzięcioł & Sas, 2023; Mamirov et al., 2022; Nguyen et al., 2024). The potential of ML-based models in sustainable construction extends beyond predictive accuracy. By integrating ML techniques with RCA assessment, engineers and researchers can optimize material selection, improve parameter control and reduce the uncertainty associated with recycled aggregate performance. This approach aligns with the broader shift toward digital transformation in construction, where data-driven methodologies enhance efficiency and sustainability (Albuquerque et al., 2022; Trach et al., 2022). However, further research is needed to refine ML models for RCA characterization, taking into account additional material parameters such as cement paste content, microstructural properties, long-term durability assessments, or specifically the permeability coefficient (Dzięcioł & Sas, 2023; Sua-iam & Makul, 2023; Zhou et al., 2024). Moreover, hybrid ML models that combine multiple algorithms could further enhance prediction robustness and reliability.

This study aims to demonstrate the predictive capabilities of AdaBoost and Neural Network algorithms in estimating the permeability coefficient of RCA. By comparing these two approaches, the research seeks to evaluate their accuracy, generalization ability, and suitability for practical applications in construction. Additionally, SHAP analysis will identify the key features influencing permeability predictions, providing insights into the underlying physical mechanisms governing RCA behavior (Hart, 1989; Zargari Marandi, 2024).

The findings of this study contribute to the growing body of knowledge on sustainable material reuse, supporting the integration of ML-driven methods in the circular economy framework for construction.

2. Materials and methods

The recycled concrete aggregate used in the study originated from the demolition of a repaired road surface. A constant head method was applied to test the coefficient of permeability. The grain size range is presented below in Table 1.

Figure 1 presents the results of the coefficient of permeability tests with a compaction energy of 0.59 J/cm³ and 2.65 J/cm³. The boxplot illustrates the distribution of permeability values for both energy levels, with a notable decrease in permeability observed at the higher compaction energy. The mean permeability coefficient at 0.59 J/cm³ is approximately 0.007 m/s, whereas at 2.65 J/cm³, it is significantly lower, around 0.0041 m/s. Increasing the energy of compaction reduces the permeability of the soil due to the compaction of the soil structure and the reduction of voids.

Table 1. A summary of the material's grain size range.

Description and unit	Minimum	Maximum
Particle size (d_3), [mm]	0.15	0.17
Particle size (d_{10}), [mm]	0.28	0.30
Particle size (d_{17}), [mm]	0.60	0.80
Particle size (d_{20}), [mm]	0.98	1.30
Particle size (d_{30}), [mm]	1.90	2.20
Particle size (d_{50}), [mm]	3.30	3.90
Particle size (d_{60}), [mm]	5.50	7.80
Particle size (d_{90}), [mm]	9.80	12.50

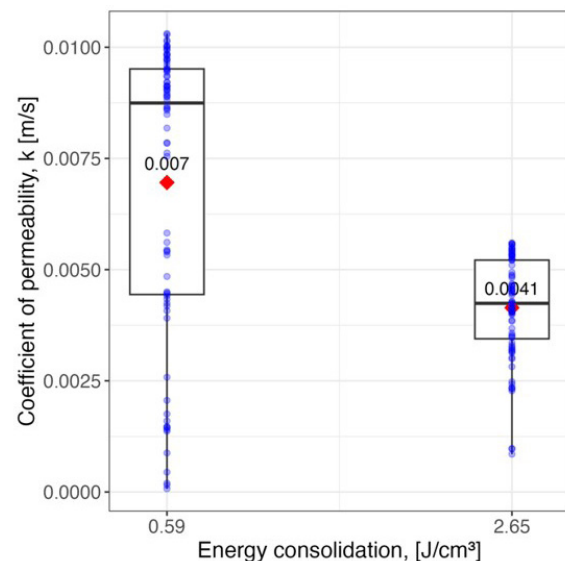


Figure 1. The results of the coefficient of permeability tests.

Figure 2 presents a 3D scatter plot correlating filtration velocity, gradient, and compaction energy. The colour gradient represents variations in filtration velocity, with higher values observed at lower compaction energy levels. The data points show a nonlinear trend, where permeability decreases as the compaction energy increases.

A total of 160 laboratory measurements of the coefficient of permeability were performed, with each observation including the particle size distribution, compaction energy, and hydraulic gradient values. These data formed the basis for training and testing the machine learning models. The input features selected for the model were: the grain size parameters d_5 to d_{90} , compaction energy, volumetric density, porosity, grain size curvature index (C_c), homogeneity index (C_u) and hydraulic gradient. The permeability coefficient was used as the target variable. The choice of features was based on established relationships in soil mechanics, particle size distribution affects pore structure and flow paths, compaction energy influences the density and thus void ratio. In contrast, the hydraulic gradient controls flow behavior in the medium. The dataset was randomly split into a training set (80%) and a test set (20%). To validate the model's robustness, 10-fold cross-validation was applied during the training process. This study implemented two machine learning algorithms to estimate the permeability coefficient of recycled concrete aggregates: Artificial Neural Networks (ANN) and AdaBoost. The Artificial Neural Network (ANN) model was implemented as a feedforward architecture trained using backpropagation. Its structure, including the number of neurons and layers, was optimized through cross-validation. The AdaBoost algorithm, using decision tree stumps as weak learners, underwent hyperparameter tuning with grid search. The results were verified with the use of error analysis, for individual models were estimated:

Root Mean Square Error (RMSE):

$$RMSE = \sqrt{\frac{1}{N} \sum_{i=1}^N (y_i - \hat{y})^2} \quad (1)$$

Mean Squared Residuals (MSR):

$$MSR = \frac{\sum_{i=1}^n (\hat{y}_i - y_i)^2}{n} \quad (2)$$

Mean Absolute Error (MAE):

$$MAE = \frac{1}{n} \sum_{i=1}^n |y_i - \hat{y}_i| \quad (3)$$

Coefficient of determination (R^2):

$$R^2 = \frac{\sum_{i=1}^N (\hat{y}_i - \bar{y})^2}{\sum_{i=1}^N (y_i - \bar{y})^2} \quad (4)$$

To enhance the interpretability of the machine learning models developed in this study, SHAP (Shapley Additive Explanations) analysis was applied. SHAP is a widely adopted explanatory technique for interpreting predictions made by machine learning algorithms. It quantifies the contribution of each input feature to a model's output by assigning importance scores, thereby offering detailed insight into the internal decision-making process of the model (Dzięcioł & Sas, 2024; Hart, 1989). In this context, SHAP enabled the identification of the most influential parameters affecting the predicted permeability coefficient of recycled concrete aggregates (RCA), revealing distinct patterns between the AdaBoost and Neural Network algorithms.

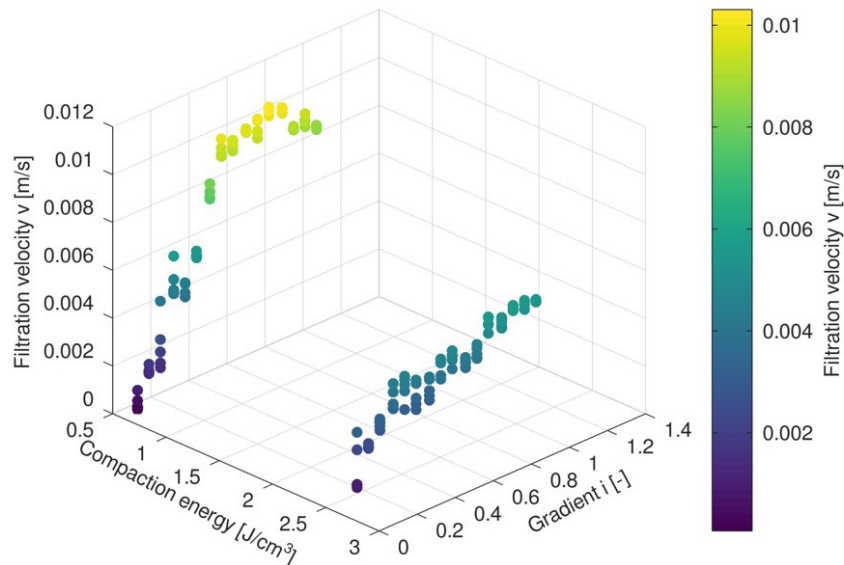


Figure 2. Results of filtration velocity versus gradient and compaction energy tests.

3. Results

Table 2 presents the prediction results of the permeability coefficient for both algorithms. The following physical characteristics of the investigated Recycled Concrete Aggregate (RCA) were used to determine models based on AdaBoost and Neural Network algorithms: grain sizes at selected percentage content, including smaller sizes (d_5 , d_{10} , d_{17} , d_{20} , d_{30} , d_{50} , d_{60} , d_{90}), grain size curvature index, homogeneity index, volumetric density, porosity, and compaction energy. In the case of the training sample, better results of fitting to the observation data were obtained for the AdaBoost algorithm, in its case the R^2 was 0.959, for Neural Network the R^2 was 0.682.

In the test sample (Table 3) the R^2 results obtained for the algorithms were lower for AdaBoost the coefficient of determination was 0.886, and for Neural Network it was 0.640.

Figure 3 presents a comparison between the predicted and actual permeability coefficient (k) values obtained using the AdaBoost regression model. The left plot corresponds to

the training set, while the right plot represents the test set. The coefficient of determination (R^2) for the training set is 0.959, indicating an excellent fit of the model to the training data. The test set shows a slightly lower R^2 value of 0.886, suggesting that the model generalizes well to unseen data, though with minor deviations. The strong alignment of data points along the regression line in both cases highlights the high accuracy of AdaBoost in predicting the permeability coefficient.

Figure 4 illustrates the performance of a Neural Network model in predicting the coefficient of permeability. Similar to Figure 3, the left plot represents the training set, while the right plot corresponds to the test set. The training set exhibits an R^2 value of 0.682, indicating a moderate fit to the data. However, the test set has an even lower R^2 value of 0.640, reflecting a weaker generalization capability than AdaBoost. The increased spread of data points around the regression line suggests that the Neural Network model struggles to accurately capture the permeability coefficient trends, potentially due to insufficient training data or the complexity of the underlying physical processes.

Table 2. Evaluation of the training set.

Model	RMSE	MSR	MAE	R^2
AdaBoost	0.001	3.04×10^{-7}	3.62×10^{-4}	0.959
Neural Network	0.002	2.39×10^{-6}	1.26×10^{-3}	0.682

Table 3. Evaluation of the test set.

Model	RMSE	MSR	MAE	R^2
AdaBoost	0.001	8.07×10^{-7}	6.34×10^{-4}	0.886
Neural Network	0.002	2.53×10^{-6}	1.30×10^{-3}	0.640

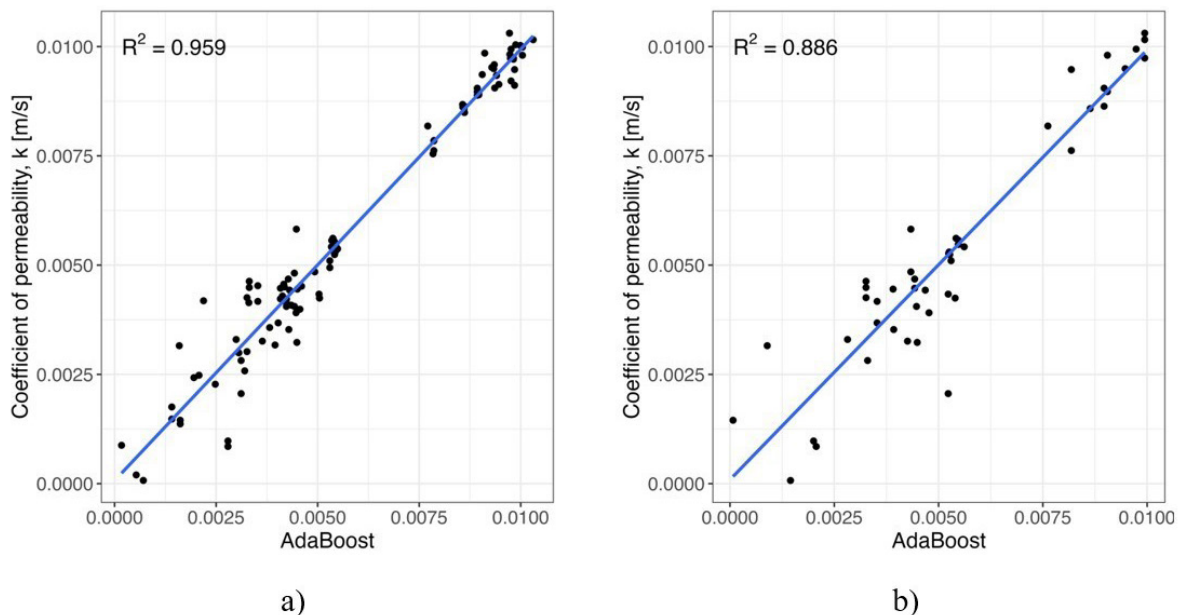
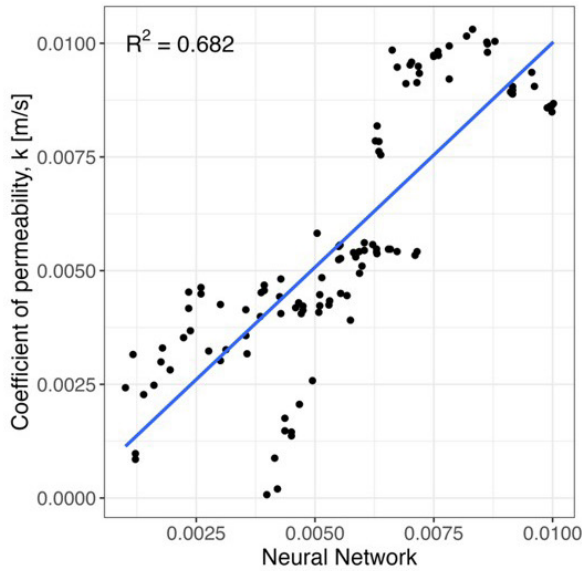
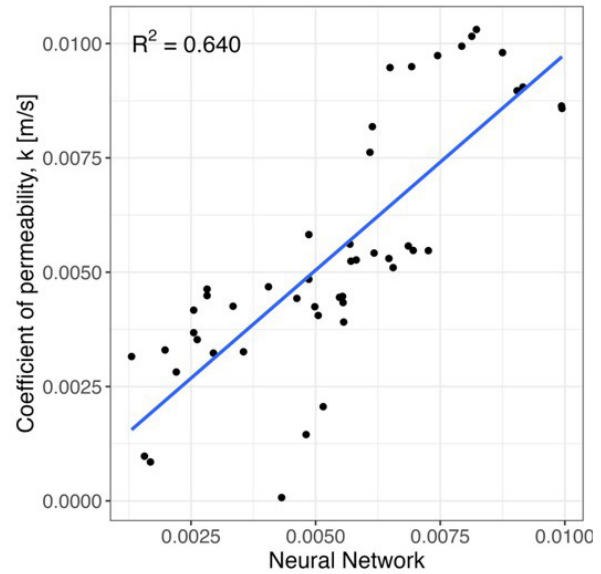


Figure 3. AdaBoost prediction results: (a) training set; (b) test set.

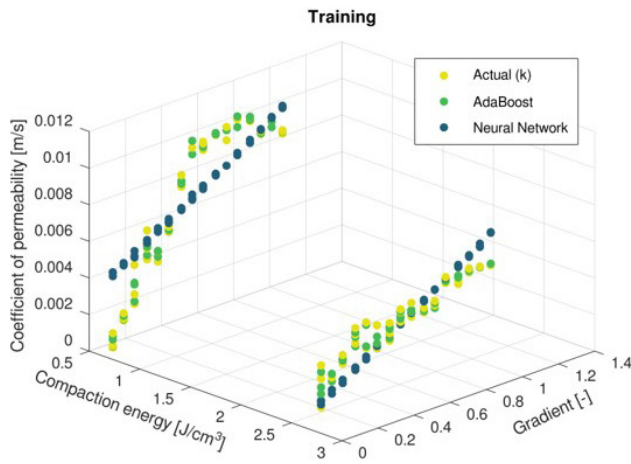


a)

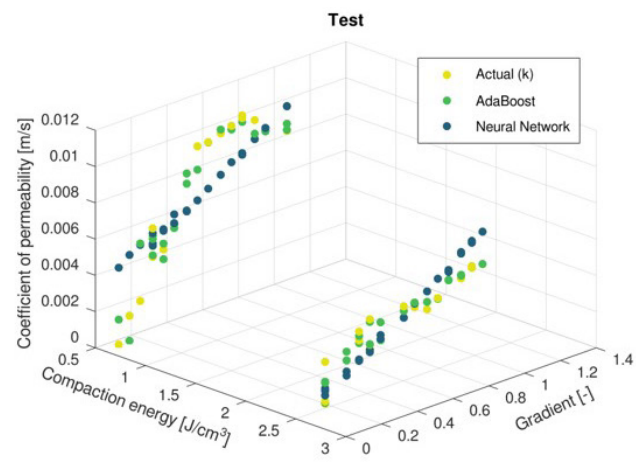


b)

Figure 4. Neural Network prediction results: (a) training set; (b) test set.



a)



b)

Figure 5. Comparison of actual permeability coefficient (k) values with predictions from AdaBoost and Neural Network models: (a) training set; (b) test set.

The R^2 results for both algorithms translate into a distance of the individual results from the reference line, thus larger residuals' values. Comparing all the graphs, one can notice a greater concentration of results around the reference line in the case of the AdaBoost algorithm, which harmonizes with the obtained high R^2 results and low error results for this algorithm. Figure 5 compares the actual and predicted permeability coefficient values using AdaBoost and Neural Network models for training and test sets.

The left plot represents the training set, while the right plot corresponds to the test set. The yellow markers represent actual permeability coefficient values, while the green and blue markers correspond to AdaBoost and Neural Network predictions. In both plots, the AdaBoost model aligns more closely with the actual permeability coefficient values than the Neural Network model, particularly at lower permeability coefficient values. This further confirms the predictive performance of AdaBoost observed in previous analyses.

In the test set (right plot), the Neural Network model shows more deviation from actual values, especially at higher permeability ranges, indicating potential generalization issues. In contrast, the AdaBoost predictions remain consistently aligned with observed permeability coefficient trends, suggesting better adaptability to unseen data.

Figure 6 presents the prediction error distribution for the AdaBoost and Neural Network models in both the training (top) and test (bottom) datasets. The x-axis represents the actual permeability coefficient, while the y-axis shows the prediction error, calculated as the difference between predicted and actual permeability values. In the training dataset (top plot), the AdaBoost model (green points) exhibits a more concentrated distribution of errors around zero, indicating a better fit to the training data. The Neural Network model (blue points) shows more variability, particularly for higher permeability values, suggesting a greater prediction error. Both models show slightly increased error variability in the test dataset (bottom plot), which is expected due to the model's exposure to unseen data. However, the AdaBoost model maintains a lower spread of errors, reinforcing its superior generalization ability compared to the Neural Network. The Neural Network model shows higher prediction deviations for lower and higher permeability values, which may indicate a mismatch or limitation in capturing complex dependencies on the permeability coefficient.

Figure 7 presents the residual error distributions for the AdaBoost and Neural Network models in both the training and test datasets. The residual error represents the difference between the predicted and actual permeability coefficient values. The histograms provide insights into the error distributions and model performance. The top row shows the residuals

for the training dataset, with AdaBoost on the left (green) and Neural Network on the right (blue). The bottom row displays the residuals for the test dataset, with the same model assignments. In the training set, the AdaBoost model exhibits a more symmetrical and concentrated residual distribution around zero, indicating a well-fitted model with minimal bias. The Neural Network model, however, has a broader spread and an asymmetric shape, suggesting a higher degree of error. In the test set, the AdaBoost residuals remain relatively centered around zero, though with a slightly broader distribution than the training set, which is expected due to the model's exposure to unseen data. The Neural Network residuals, however, display an even wider spread and more extreme errors, reinforcing the findings from previous figures that this model struggles with generalization.

To assess potential overfitting of the AdaBoost model, which achieved a very high R^2 coefficient, an in-depth statistical analysis was applied. As a result, it was found that the limited decrease in performance on the test set, as well as the concentrated distribution of residuals (Figure 7) and low prediction error (Figure 6), support the conclusion that AdaBoost retains strong generalization ability. Compared to the neural network model, which shows a lower R^2 and wider distribution of residuals on the test set, AdaBoost performs more reliably on unseen data.

Figures 8 and 9 in this study leverage SHAP to evaluate feature importance for permeability coefficient prediction models trained using both machine learning algorithms. SHAP estimates the influence of each feature on model predictions, allowing for a deeper understanding of key factors affecting permeability.

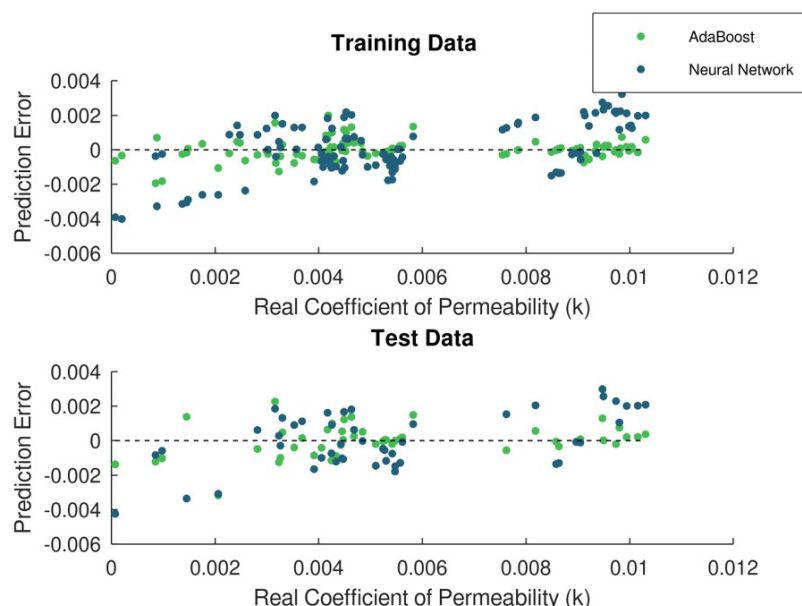


Figure 6. Prediction error comparison for the AdaBoost and Neural Network models in the training dataset (top) and test dataset (bottom).

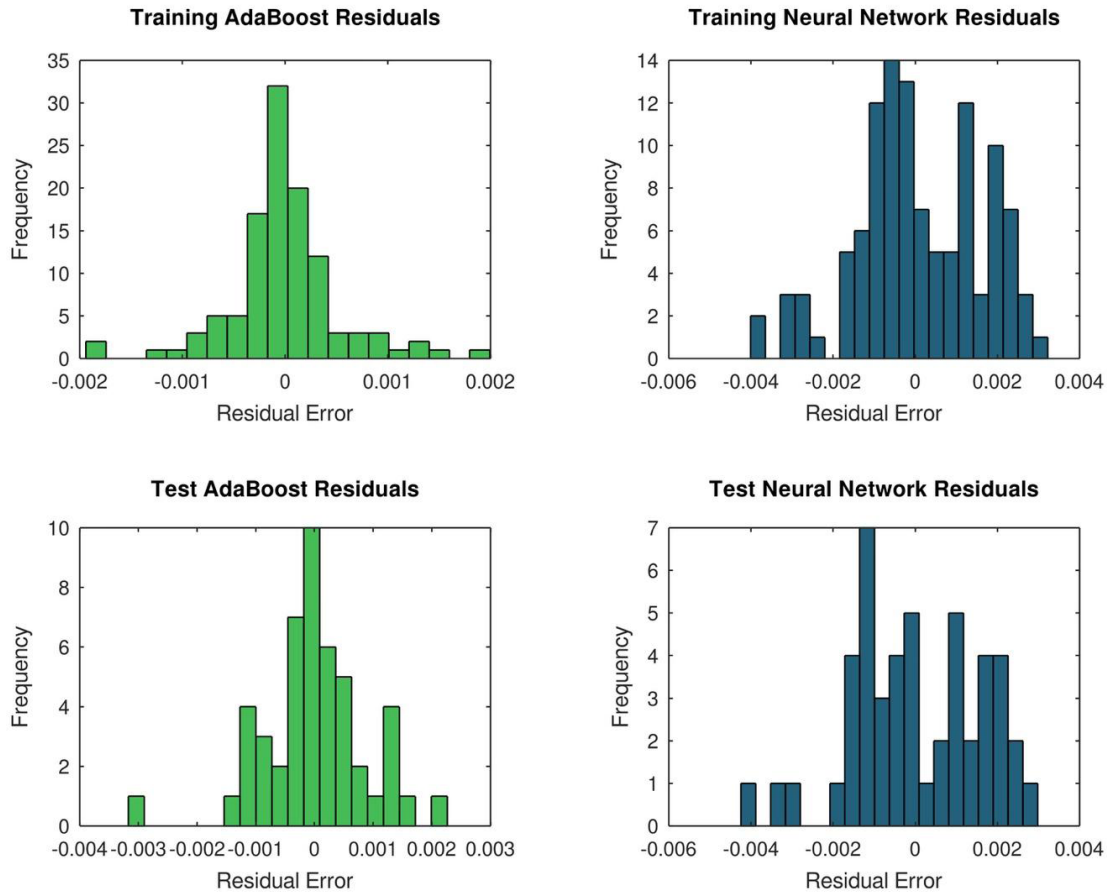


Figure 7. Residual error distributions for AdaBoost (left) and Neural Network (right) models in the training dataset (top) and test dataset (bottom).

Figure 8 presents the SHAP-based feature importance analysis for the AdaBoost regression model. The results indicate that the hydraulic gradient is the most influential feature, exhibiting the highest SHAP value and thus the strongest contribution to the model's predictions. Volumetric density also demonstrates a measurable influence, underscoring the role of compaction in controlling permeability. In contrast, features related to fine-grained particle size distribution (d_{10} , d_{30} , d_{17} , d_5) show limited overall impact. Nevertheless, these features may contribute to subtle variations in model output, affecting the precision of individual predictions.

Figure 9 illustrates the feature importance analysis for the Neural Network model using SHAP values. In contrast to Figure 8, the Neural Network model exhibits a stronger dependence on particle size variables. Features such as d_{20} , d_{50} , and d_5 have the highest SHAP values, indicating their crucial role in permeability predictions. This suggests that the Neural Network model relies more heavily on granular structure than hydraulic properties. Additionally, porosity (n) and particle diameters (d_{60} , d_{90}) contribute to permeability predictions, albeit to a lesser extent. The variance in SHAP values across different particle sizes suggests that the Neural Network model captures complex, nonlinear relationships in the dataset, making it more sensitive to granular properties.

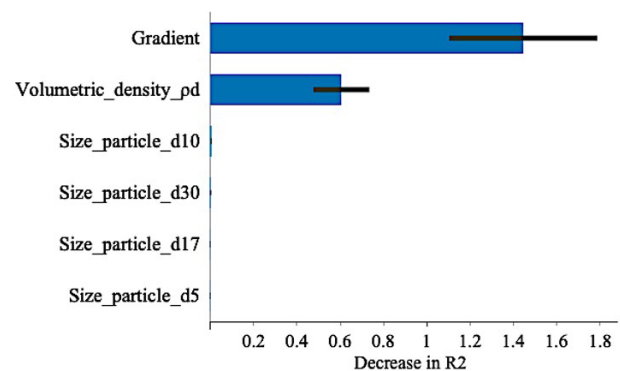


Figure 8. Feature Importance – AdaBoost.

The SHAP-based feature importance analysis revealed notable differences in how each model utilizes input features to predict permeability. In the case of AdaBoost, features related to hydraulic conditions and compaction (e.g., gradient, volumetric density, compaction energy) were dominant, while particle size distribution appeared to have a limited impact. In contrast, the Neural Network model assigned higher importance to granular parameters such as d_{20} , d_{50} , and d_5 , which aligns more closely with classical geotechnical understanding of

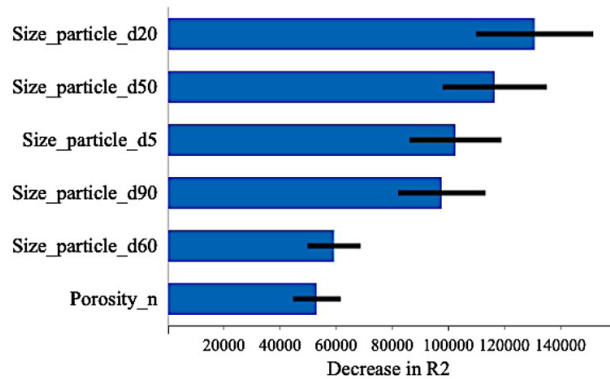


Figure 9. Feature Importance – Neural Network.

how pore structure affects flow. This divergence suggests that while AdaBoost may rely more on global patterns in the data, including strongly expressed relationships with compaction and gradient, the Neural Network model is potentially more sensitive to microstructural variation captured by the grain size distribution. However, despite this capacity to model detailed dependencies, the Neural Network exhibited lower predictive performance compared to AdaBoost. This may be attributed to factors such as limited training data and high input dimensionality, which may impair the model's ability to generalize effectively to unseen data.

4. Conclusions

This study explored sustainable practices in recycling concrete aggregates, focusing on predicting the permeability coefficient for recycled concrete using machine learning techniques. Two regression models, AdaBoost and Neural Networks, were evaluated and compared. Results showed that AdaBoost outperformed the Neural Network model, achieving an R^2 of 0.959 for the training set and 0.886 for the test set, indicating strong generalization capabilities. In contrast, the Neural Network model exhibited lower accuracy, with R^2 values of 0.682 (training) and 0.640 (test), suggesting limited generalization despite its ability to capture complex permeability-related patterns. To improve interpretability, Shapley Additive Explanations (SHAP) were applied, allowing detailed analysis of feature importance. In the AdaBoost model, hydraulic properties such as gradient and volumetric density emerged as key features, highlighting the role of compaction and hydraulic conditions in determining permeability behavior. Conversely, the Neural Network model relied predominantly on particle size variables (d_{20} , d_{50} , d_3), indicating a greater sensitivity to granular structure than hydraulic conditions. The findings confirm that machine learning offers a data-driven approach to predicting the permeability of recycled concrete aggregates, supporting sustainable material design and reuse strategies. By applying SHAP for feature interpretation, the study

enhances understanding of the predictive process and improves the interpretability of machine learning models of machine learning models in geotechnical applications. The results suggest that AdaBoost is better suited for permeability prediction in the studied dataset due to its high accuracy and adaptation to hydraulic properties. At the same time, while still useful, neural networks may require further tuning or working with new data ranges to improve generalization.

These insights have significant implications for the sustainable use of recycled concrete aggregates, reinforcing the importance of AI-driven approaches in optimizing material performance. By identifying the dominant predictors for selecting and refining machine learning models in civil engineering applications. Furthermore, the ability to analyze feature contributions through SHAP facilitates model validation and ensures that predictions are consistent with established permeability theories.

The results demonstrate strong predictive performance, particularly for the AdaBoost model. It is important to note that the models were trained on RCA samples with specific origins and characteristics. As such, the generalizability of these models to all recycled concrete aggregates cannot be assumed without additional calibration or retraining using local data. From a design perspective, the observed prediction errors, particularly for AdaBoost, fall within ranges considered acceptable for preliminary geotechnical assessments. These models could therefore support early-stage evaluations, material screening, or comparative studies. However, further validation would be required for critical design applications or regulatory compliance to ensure reliability under varying field conditions and RCA compositions.

While this study demonstrates the potential of machine learning in permeability prediction, future research should further explore additional factors such as chemical composition or temperature effects to refine predictive models. Optimizing Neural Network architectures and exploring hybrid machine learning approaches could enhance accuracy and generalization. Validating these models in real-world construction scenarios would further strengthen their applicability in sustainable infrastructure projects.

In conclusion, this study highlights the effectiveness of machine learning in sustainable materials engineering, providing a scientifically sound methodology for optimizing recycled concrete applications. By improving the accuracy of permeability prediction, these insights contribute to developing sustainable construction practices, reducing environmental impacts and promoting the principles of a circular economy in the construction industry.

Declaration of interest

The authors have no conflicts of interest to declare. All co-authors have observed and affirmed the contents of the paper and there is no financial interest to report.

Authors' contributions

Justyna Dzięcioł: conceptualization, data curation, visualization, writing – original draft, methodology, supervision, validation, writing – review & editing. Wojciech Sas: supervision.

Data availability

Datasets generated and analyzed in the current study can be made available upon inquiry to the corresponding author.

Declaration of use of generative artificial intelligence

This work was prepared without the help of generative artificial intelligence (GenAI).

List of symbols and abbreviations

i	Index of observation in the dataset
n	Total number of observations in the dataset
y_i	Actual observed value of the permeability coefficient
$\hat{y}_i$	Predicted value of the permeability coefficient
$\bar{y}$	Mean value of the actual permeability coefficient
N	Total number of samples considered for evaluation

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Experimental research of the effects of adding different chemical reagents to clayey soil on improving its physical and mechanical properties

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Article

Keywords

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Abstract

Clayey soil in its natural state usually has a low bearing capacity, high compressibility, and high sensitivity to changes in water content. These properties represent an obstacle to the use of this type of soil in construction industry projects, so it is necessary to stabilize it before its application for construction purposes. This paper presents the results of a comparative study of the effects of chemical stabilization of clayey soils on improving soil properties by considering a variety of chemical reagents. In addition to commonly used chemical reagents such as calcium carbonate (lime), alternative materials such as magnesium carbonate, sodium silicate, and potassium hydroxide were also considered. With an aim to determine the optimal content of reagent in a mixture with clayey soil, each of the selected chemical reagents was considered with three different percentage shares in the mixture. Given that the permanent improvement of soil properties is of utmost importance in geotechnical engineering, for this purpose, changes in the crucial physical and mechanical properties of the treated clayey soil (Atterberg limits, unconfined compressive strength, and shear strength) were also monitored over time after the chemical treatment. The research results revealed that the selected chemical reagents have different efficiencies on the considered physical and mechanical properties of clayey soil, whereby each of them contributed to the soil improvement. This opens up the possibility of applying the selected stabilizers within the framework of the chemical soil stabilization technique, primarily in the field of roadway construction.

1. Introduction

Clayey soils pose a challenge in the roadway construction due to their properties, caused by the mineralogical composition of clay and water content, in terms of low bearing capacity, high compressibility, and high sensitivity to water content, which make them prone to cracking or swelling (Marinković et al., 2021). This makes them unsuitable for direct application in roadway construction projects. The problem can be overcome by applying soil stabilization techniques aimed at improving physical and mechanical properties of the soil, among which the soil stabilization using chemical reagents is one of the most commonly applied. The development of this technique dates back thousands of years, from the early attempts of the ancient Greeks and Romans to use materials such as lime and ash to improve soil properties (Han, 2015). This engineering technique has evolved throughout history, enabling the construction of stable and long-lasting structures on a wide range of soil types.

Chemical soil stabilization techniques have been enhanced over time with the use of new types of reagents. Recently, in order to reduce the impact on the environment, research has focused on reagents acceptable from an ecological point of view.

The effects of a stabilization technique depend largely on both the type of reagent and the type of soil to which it is applied, as well as on the amount of reagent added and the time elapsed since the reagent and soil were mixed. The reactions between the reagent and the clayey soil occur through two main processes: a rapid ion exchange reaction process, known as soil modification, and a slower pozzolanic reaction process, known as soil stabilization or hardening (Maaitah, 2012).

Soil stabilizers are usually classified into two categories - those commonly used in the roadway construction (lime, cement, fly ash, bituminous products, etc.) and alternative ones that do not have a commercial use for the aforementioned purpose. Considering the most commonly used reagents on the one hand, these materials are most often used for soil

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stabilization in engineering practice, in more than 80% of cases (Cristelo et al., 2009; Shon et al., 2010; Onyelowe & Okafor, 2012; Zafirovski et al., 2024) and have been proven to be effective in improving soil properties. On the other hand, alternative materials, which do not yet have widespread commercial use in geotechnics, are still insufficiently investigated and have recently attracted increasing attention from researchers. The application of new chemical stabilizers has opened up new opportunities to address the challenges posed by clayey soils, offering more sustainable and cost-effective solutions for construction industry projects. However, in previous studies, the emphasis has mainly been on the effects of combining an alternative reagent with some of the commonly used materials listed above, according to the specific project needs and local soil characteristics (Kamon & Nontananandh, 1991; Vakili et al., 2016; Firoozi et al., 2017; Behnood, 2018; Andavan & Pagadala, 2020; Silva et al., 2025), whereas research into the individual effects of alternative materials was represented to a much lesser extent (Marto et al., 2014; Davidovic et al., 2012; Moayedi et al., 2012; Latifi et al., 2016; Miranda Neto & Mahler, 2017; Singh et al., 2020; Seco et al., 2022; Silva et al., 2022).

In the present research, in addition to previously known chemical reagents, such as lime (calcium carbonate), alternative reagents such as magnesium carbonate, sodium silicate, and potassium hydroxide were also examined in the chemical stabilization of clayey soil. On the basis of the study on the soil Atterberg limits and unconfined compressive strength (*UCS*), monitored at four different time intervals after the soil treatment (3, 7, 14, and 28 days), the optimal content for each of the considered reagents in the mixture with clayey soil was determined. In the subsequent stage of the research, direct shear tests were carried out on the soil samples prepared with the optimal content for each of the considered reagents, in order to identify the individual effect of each reagent on the shear strength parameters of the clayey soil. The effects of the reagents were observed 3, 14, and 28 days after the chemical stabilization, thus assessing both the short-term effects and the long-term effects of stabilization.

2. Materials and methods considered in the research

2.1 Natural clayey soil

The clayey soil considered in the research originates from the Crvena Reka borrow pit, along the E-80 Niš–Dimitrovgrad Highway route in the south-eastern part of the Republic of Serbia. During the construction of the Highway, a landslide occurred at this location. The remediation measures involved the construction of a retaining structure as a curtain of piles. Samples of clayey soil for the research were taken during the construction of bored piles, from the depth of the registered sliding surface.

The mineralogical composition of the natural clayey soil was determined by X-Ray Diffraction (XRD) analysis at the Laboratory for General and Inorganic Chemistry at the Faculty of Science and Mathematics of the University of Niš (Serbia). It consists of clay minerals (montmorillonite ($\text{CaO}_2(\text{Al,Mg})_2\text{SiO}_{10}(\text{OH})_{24}\text{H}_2\text{O}$) – 15%, illite ($\text{KA}_2\text{Si}_3\text{AlO}_{10}(\text{OH})_2$) – 11%, and clinocllore ($\text{Mg}_5\text{Al}(\text{Si,Al})_4\text{O}_{10}(\text{OH})_8$) – 10%) in combination with other minerals (calcite (CaCO_3) – 39% and quartz (SiO_2) – 25%). The results of this research should indicate the potential of reusing the clayey soil material for the roadway construction purposes after mixing with a certain chemical reagent.

The physical and mechanical properties of the natural clayey soil, which was used in this research, were determined by tests carried out at the Laboratory for Geotechnics of the Faculty of Civil Engineering and Architecture of the University of Niš (Serbia). The results are shown in Table 1.

2.2 Reagents

One of the most commonly used reagents for soil stabilization is lime (calcium carbonate) - $\text{Ca}(\text{OH})_2$. This natural mineral, often found in the form of limestone, has been used for decades in a wide range of construction industry projects. Lime is considered a versatile and cost-effective solution, primarily due to its neutralizing effect on soil acidity. When added to acidic soils, lime reacts with the acidic compounds, increasing the *pH* value of the soil and improving its physico-mechanical properties. This stabilizer also stimulates the flocculation of soil particles, thereby

Table 1. Properties of the natural clayey soil used in the research.

Property symbol (unit)		Value
Grain size distribution	Gravel (%)	1.2
	Sand (%)	4.9
	Silt (%)	40.6
	Clay (%)	53.3
USCS soil classification (–)		CL
G_s (–)		2.705
C_u (–)		8.0
C_c (–)		2.0
MDD (g/cm^3)		1.903
OMC (%)		18.5
LL (%)		49
PL (%)		23
PI (%)		26
pH (–)		9.5
M_v (MPa)		12.945
CBR (%)		2.71
s (%)		2.91
UCS (kPa)		205
c' (kPa)		16.9
ϕ' (°)		16.5

Legend: see List of symbols and abbreviations.

reducing the swelling potential and enhancing the shear strength of the soil. According to Bell (1996), all types of clay minerals react with lime. In addition, clayey soils with medium to high plasticity, where the plasticity index *PI* is greater than 15%, are most suitable for stabilization with lime.

While lime represents the standard selection for soil stabilization, several less commonly used materials for chemical stabilization have shown potential for soil stabilization in specific applications, as presented below.

Magnesium carbonate (MgCO_3) is a viable option for improving the physical and mechanical properties of soils, particularly those with a high clay or silt content. This reagent has been shown to be effective in increasing the cohesion, reducing the plasticity, and enhancing the load-bearing capacity of the treated soil (Yi et al., 2013; Taha Jawad et al., 2014; Seco et al., 2017).

Sodium silicate (Na_2SiO_3), also known as water glass, has been used to stabilize soils under specific conditions. It contributes to reducing the permeability and increasing the cohesion of fine-grained soils. Sodium silicate forms chemical bonds with soil particles, reducing their susceptibility to moisture-induced expansion and contraction (Hurley & Thornburn, 1972; Maaitah, 2012). Research by Pakir et al. (2015) has shown the successful application of this reagent to improve the properties of marine clay.

Potassium hydroxide (KOH) is a highly alkaline substance that can be used to stabilize soils under certain conditions. It is particularly effective in improving the bearing capacity and compaction of clayey soils. The addition of potassium hydroxide initiates reactions that increase the strength of the soil and reduce its sensitivity to changes in moisture content (Elkhebu et al., 2018).

The selected reagents with the considered percentage shares in the mixture with respect to the dry weight of the soil sample (3%, 5%, and 7% each), along with the corresponding labels of the examined mixtures, are summarized in Table 2.

Overall, the primary goal of the present research was to examine the potential of each of the selected reagents alone (i.e., without combining them mutually) in terms of improving the Atterberg limits, unconfined compressive strength, and the shear strength parameters (cohesion and internal friction angle) of the clayey soil, which are of a paramount importance in geotechnical engineering and roadway construction.

2.3 Methods of experimental research

The soil material from the site was brought to the laboratory. Selected reagents were added individually to the clay material with the considered percentage shares in the mixture (according to Table 2) in relation to the dry weight of the soil.

The Atterberg limits were tested according to the national standard SRPS EN ISO 14688-12:2018 (SRPS EN ISO, 2018) harmonized with the European norms (EN).

The Liquid Limit (*LL*) was determined by applying the fall cone method and the Plastic Limit (*PL*) using the thread rolling method. The samples were dried, crushed, and sieved through a 425 μm sieve, and then mixed with each of the four selected reagents at three different percentage shares, in order to determine the effect of each of them on the change in the values of the Liquid Limit and the Plastic Limit, as well as the associated Plasticity Index (*PI*).

The samples for the unconfined compressive strength (*UCS*) test and the direct shear test were prepared with a moisture content equal to the optimal moisture content of natural soil according to the Proctor test (*OMC* = 18.5%, Table 1), for the purpose of comparability of the results of natural and chemically stabilized clayey soil. The samples were compacted during the standard Proctor test with compaction energy of 600 kJ/m^3 . The treated samples were wrapped in a plastic foil and stored in a closed plastic box until the corresponding time for conducting the test, in order to maintain the same moisture content of the sample over time.

UCS testing was applied to all samples in accordance with the national standard SRPS EN ISO 13286-41:2022 (SRPS EN ISO, 2022), which is in line with the European standards. The cylindrical sample was 15 cm in height and 10 cm in diameter. Each sample was compressed until reaching the peak load, where the applied load was recorded by the data acquisition system. The *UCS* test was used to determine the compressive strength of the soil in natural conditions and the chemically treated soil for each of the four reagents considered with three different percentage shares. The obtained result represents the mean value of three tested samples.

In terms of obtaining the shear strength parameters, direct shear tests were performed according to the national standard SRPS EN ISO 17892-10:2019 (SRPS EN ISO, 2019) harmonized with the actual European norms, on samples with the optimal content for each of the selected reagents, which was determined based on the *UCS* test results.

Table 2. Overview of the tested mixtures (samples) with the selected contents of examined reagents.

Reagent	Percentage share	Mix label
Lime	3%	L3
	5%	L5
	7%	L7
Magnesium carbonate	3%	MC3
	5%	MC5
	7%	MC7
Sodium silicate	3%	SS3
	5%	SS5
	7%	SS7
Potassium hydroxide	3%	PH3
	5%	PH5
	7%	PH7

The samples were square in shape at the base, measuring 6.0 cm × 6.0 cm, and 2.0 cm high. The shear rate was 0.0083 mm/min. For each of the considered reagents and for each of the considered time intervals after the soil treatment, in accordance with the specified standard, three specimens from one soil sample were prepared for shearing under three different vertical pressures (a constant normal stress of 50 kPa, 100 kPa, and 200 kPa during the tests), to allow the shear strength parameters to be determined. During shearing, draining of the samples was enabled, and as a result, the effective shear strength parameters (effective cohesion c' and effective internal friction angle ϕ') were obtained.

The complete experimental research and accompanying tests were carried out at the Laboratory for Geotechnics of the Faculty of Civil Engineering and Architecture of the University of Niš. Table 3 summarizes the experimental program.

3. Comparative analysis and discussion of results

3.1 Atterberg limits

Table 4 summarizes the values of consistency limits obtained for clayey soil mixtures treated with three different

percentages (3%, 5%, and 7%) of four selected reagents, at four curing intervals (3, 7, 14, and 28 days after the treatment). For comparative purposes, it is important to note that the natural (untreated) clayey soil exhibited the values of LL , PL , and PI of 49%, 23%, and 26%, respectively, as indicated in Table 1.

For all reagents considered, the value of LL remained almost unchanged, with no significant changes observed either due to increased reagent content or over time. The most pronounced increase in LL value was observed in the case of sodium-silicate treated samples (SS) after 3 days of curing. By the 7th day, LL values of all samples became comparable, and this pattern continued at the subsequent time intervals of 14 and 28 days. The most substantial increase in LL occurred with the addition of 7% magnesium carbonate (MC7) after 28 days, where the LL reached a value of 54%.

Changes in the plastic limit and the plasticity index were observed for all the tested samples throughout the curing period. The most pronounced changes in PL and PI values occurred after 28 days since the treatment. The highest PL values were recorded on soil samples treated with 7% lime (L7, $PL = 36\%$) as well as with 7% potassium hydroxide (PH7, $PL = 35\%$), which also exhibited the lowest PI values of 17% and 16%, respectively. The other samples resulted

Table 3. Summary of the experimental program.

Reagent and percentage share	Laboratory test and testing time interval after chemical stabilization		
	Atterberg limits (after 3, 7, 14, and 28 days)	UCS test (after 3, 7, 14, and 28 days)	Direct shear test (after 3, 14, and 28 days)
Lime	3%, 5%, 7%	3%, 5%, 7%	optimal content (5%)
Magnesium carbonate	3%, 5%, 7%	3%, 5%, 7%	optimal content (5%)
Sodium silicate	3%, 5%, 7%	3%, 5%, 7%	optimal content (5%)
Potassium hydroxide	3%, 5%, 7%	3%, 5%, 7%	optimal content (7%)

Table 4. Values of the consistency limits of the clayey soil after the chemical stabilization with the selected reagents and with the considered percentage shares.

Mix label	After 3 days			After 7 days			After 14 days			After 28 days		
	LL	PL	PI	LL	PL	PI	LL	PL	PI	LL	PL	PI
(%)												
Natural soil	49	23	26									
L3	49	23	26	49	25	24	51	29	22	50	31	19
L5	51	28	23	52	30	22	50	30	20	53	34	19
L7	51	34	17	53	37	17	52	34	18	53	36	17
MC3	50	26	24	48	29	19	50	27	23	49	24	25
MC5	49	28	21	51	29	22	51	29	22	50	28	22
MC7	52	34	18	52	36	16	52	32	20	54	34	20
SS3	50	29	21	50	23	27	49	28	21	50	25	25
SS5	52	29	23	50	30	20	52	34	18	50	27	23
SS7	53	36	17	51	35	16	51	31	20	52	33	19
PH3	49	24	25	51	28	23	51	27	24	51	25	26
PH5	50	29	23	51	32	19	51	30	21	51	29	22
PH7	52	35	17	49	35	14	50	30	20	51	35	16

Legend: see List of symbols and abbreviations.

in *PI* values ranging between 19% and 26% after 28 days, indicating minimal variation in outcomes regardless of the type of chemical reagent used.

Given that the *LL* values remained unchanged, the increasing trend in *PL* values and the decreasing trend in *PI* values over time can be viewed positively, in terms that the treated clayey soil maintains a semi-solid state of consistency even at higher water content.

3.2 Unconfined Compressive Strength (*UCS*)

The clayey soil samples were tested with different amounts of reagents (3%, 5%, and 7%) and with different curing times, specifically at intervals of 3, 7, 14, and 28 days after the chemical treatment. The results of the *UCS* tests for the treated clayey soil are presented in Figure 1, showing the effects of the selected reagents at the considered percentage shares and over time after the treatment (lime – Figure 1a, magnesium carbonate – Figure 1b, sodium silicate – Figure 1c, and potassium hydroxide – Figure 1d). Each data point represents the average value of three tested samples per reagent, for each percentage share and curing interval.

Overall, the research revealed that each of the selected reagents contributed to the improvement of the *UCS* of the

treated soil. Lime and magnesium carbonate exhibited the most pronounced effects, with a significant increase in the *UCS* value over time. On the other hand, sodium silicate and potassium hydroxide showed a significantly weaker effect in terms of improving the *UCS* value of the soil.

The lime-treated samples demonstrated the highest *UCS* values after the treatment (Figure 1a). A clear improvement in terms of *UCS* was observed as the content of lime in the mixture with the clayey soil increased. The results showed a significant increase in strength when the lime content increased from 3% to 5%, whereas increasing the lime content from 5% to 7% did not significantly affect the increase in *UCS* values. Regardless of the content of lime added, a substantial increase in the *UCS* values was evident over time. Specifically, after a 3-day curing period, the *UCS* value increased from 205 kPa (untreated clayey soil) to 435 kPa for clayey soil treated with 3% lime (L3), to 529 kPa for soil with 5% lime (L5), whereas the addition of 7% lime (L7) resulted in an increase in the *UCS* value to 622 kPa. After 28 days of curing, the *UCS* value increased to 896 kPa, 1145 kPa, and 1178 kPa for the treatments with 3%, 5%, and 7% lime, respectively. A similar trend of increasing *UCS* values over time with the addition of lime as a stabilizer to residual granite soil was registered in the study of Cristelo et al. (2009).

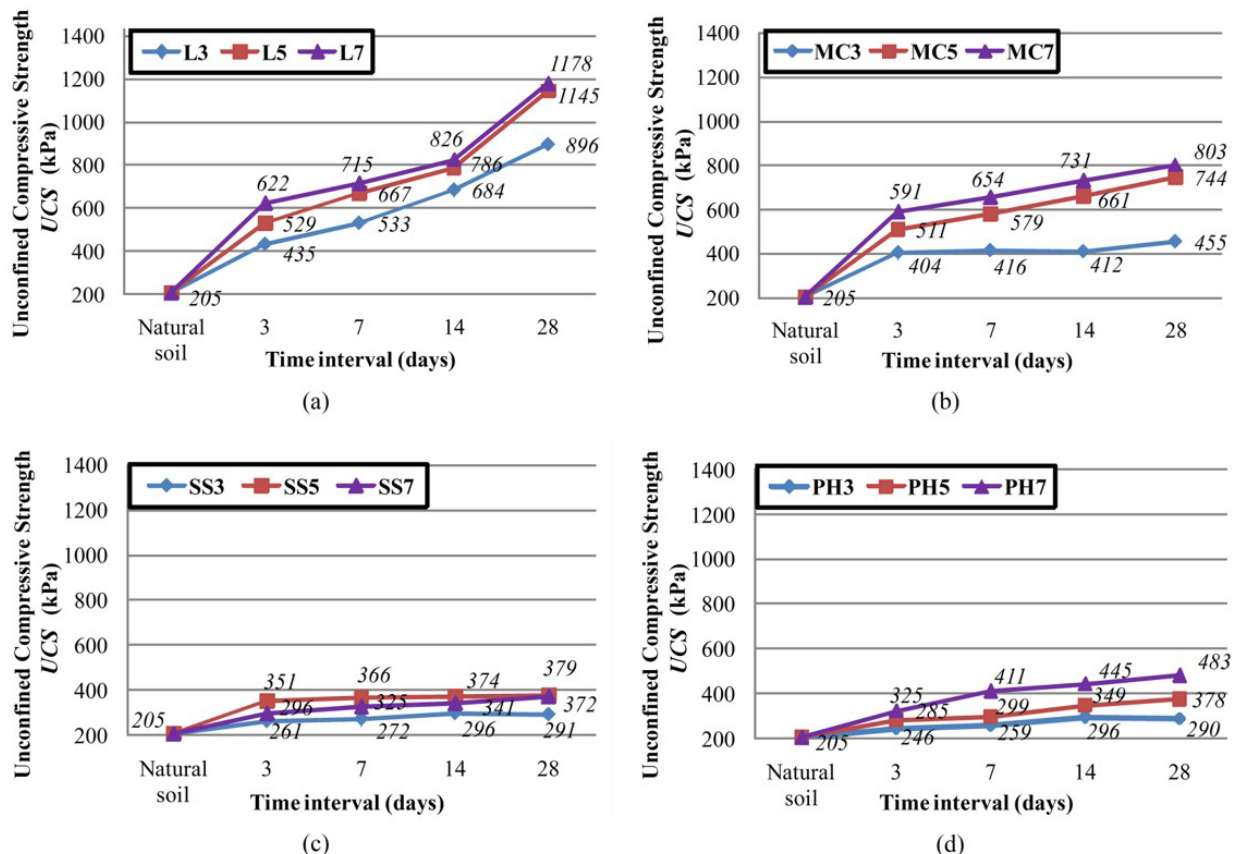


Figure 1. Change in the *UCS* values of the clayey soil over time after the chemical stabilization with the selected reagents and with the considered percentage shares: (a) lime, (b) magnesium carbonate, (c) sodium silicate, and (d) potassium hydroxide.

The addition of magnesium carbonate yielded results comparable to those obtained with the addition of lime, particularly at 5% and 7% percentage shares, but this similarity was only observed during the first 14 days of curing. By the end of 28 days, the samples treated with magnesium carbonate resulted with *UCS* values approximately 30% lower than those treated with lime (Figure 1b). Similar to the case of adding lime, the increase in *UCS* values was more pronounced when the magnesium carbonate content was increased from 3% to 5% than from 5% to 7%. For example, after 3 days of curing, the recorded *UCS* values were 404 kPa for the addition of 3% magnesium carbonate (MC3), 511 kPa for the addition of 5% magnesium carbonate (MC5), and 591 kPa for the addition of 7% of this reagent (MC7). After 28 days of curing, the measured *UCS* values were 455 kPa, 744 MPa, and 803 kPa for magnesium carbonate contents of 3%, 5%, and 7%, respectively. The trend of a rapid increase in the *UCS* value in the initial curing period was also observed in the study by Yi et al. (2013), although in their study soil carbonation was achieved in a different way – by permeation of gaseous CO₂ through prepared reactive MgO-treated soil samples.

The measured *UCS* values achieved with the addition of sodium silicate (Figure 1c), as well as potassium hydroxide (Figure 1d), were significantly lower compared to those obtained with lime and magnesium carbonate. Nevertheless, their use yet led to a measurable improvement in the unconfined compressive strength of the soil. In particular, the highest *UCS* value recorded for the addition of sodium silicate was 379 kPa, achieved with 5% reagent content after 28 days of curing. Interestingly, the results for the considered curing periods up to 28 days revealed that slightly higher *UCS* values were obtained with 5% sodium silicate content than with 7% content. The conclusion that increasing the sodium silicate content in a mixture with clayey soil results in a decrease in the *UCS* value is in line with the findings of Latifi et al. (2014) who considered tropical laterite soil (decrease in *UCS* value for sodium silicate content above 9%) and Pakir et al. (2015) who considered marine clay (decrease in *UCS* value for sodium silicate content above 6%).

The samples treated with potassium hydroxide resulted in a maximum *UCS* value of 483 kPa after 28 days at a reagent content of 7%. It was also observed that *UCS* values remained almost unchanged between 14 and 28 days, regardless of the percentage share of potassium hydroxide used. Unlike the previously elaborated reagents, potassium hydroxide exhibited a steady trend of increasing *UCS* values as the content of the reagent increased throughout all the curing intervals considered. This suggests that future research should examine the effects of using higher percentages of potassium hydroxide in a mixture with the clayey soil, in order to identify the optimal reagent content for which maximum soil strength will be achieved.

The observed increase in soil strength during the curing process can be attributed to the formation of a gel-like material with strong binding properties. This material is

formed due to chemical reactions between the soil and the reagent, which contributes to increasing the compressive strength of the treated soil.

Furthermore, extended duration of curing plays a vital role in enhancing the rate of reaction among the soil particles and the reagent. This is primarily due to the amplified positive charge within the soil–reagent mixture, which leads to the repulsion of soil particles and facilitates more effective stabilization (Latifi et al., 2014). This can also be seen from the diagrams in Figure 1, where it is observed that the uniaxial strength of the soil for all the selected reagents continues to increase even after the considered curing period of 28 days.

Based on the results obtained from the *UCS* test for samples after 28 days, the optimal content for each of the considered reagents was determined. The optimal content of reagents was selected based on the diagram in Figure 2, where it can be seen that with an increase in the content of the reagent in the mixture with the soil from 5% to 7% (for lime and magnesium carbonate), the value of *UCS* slightly increased, whereas the increased consumption of reagents is significant. The results for sodium silicate revealed that slightly higher *UCS* value was obtained with 5% sodium silicate content than with 7% content. Accordingly, for each of the above mentioned reagents, an optimal content of 5% was determined (L5, MC5, and SS5, respectively). Given the optimal content of lime, this is consistent with the conclusions of the study by Bell (1996), in which the optimal amount of lime varies between 4% and 6% depending on the type of clayey soil tested. In the case of potassium hydroxide, the optimal reagent content of 7% was chosen (PH7), since the *UCS* value increased almost linearly with the content of the reagent.

The comparative results of the *UCS* values determined on samples with the optimal content of each of the considered reagents over time can be seen in Figure 3, based on which the previously drawn conclusions regarding the order of the reagents in terms of their efficiency were confirmed.

3.3 Shear strength parameters determined by the direct shear test

The shear strength of the soil is one of the most important geotechnical properties of the soil. It is represented with the Coulomb–Mohr failure criterion, which gives the dependence of shear stress (τ) and normal stress (σ) in the form $\tau = c + \sigma \cdot \tan \phi$, where c and ϕ represent the shear strength parameters of the soil, i.e. cohesion and angle of internal friction. These parameters were determined as part of the research by the direct shear test. During shearing, draining of the samples was enabled, and as a result, the effective shear strength parameters (c' and ϕ') were obtained.

The tests were performed on samples with the optimal content for each of the selected reagents, which was determined based on the *UCS* test results. The tests were conducted for three time intervals (3, 14, and 28 days after mixing the reagent with the natural clayey soil).

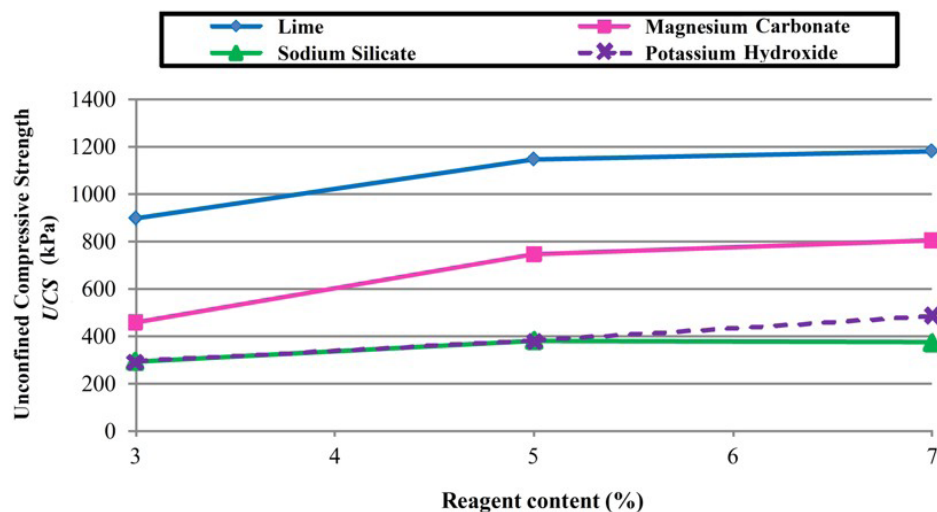


Figure 2. Variation of unconfined compressive strength of clayey soil with reagent content after 28 days.

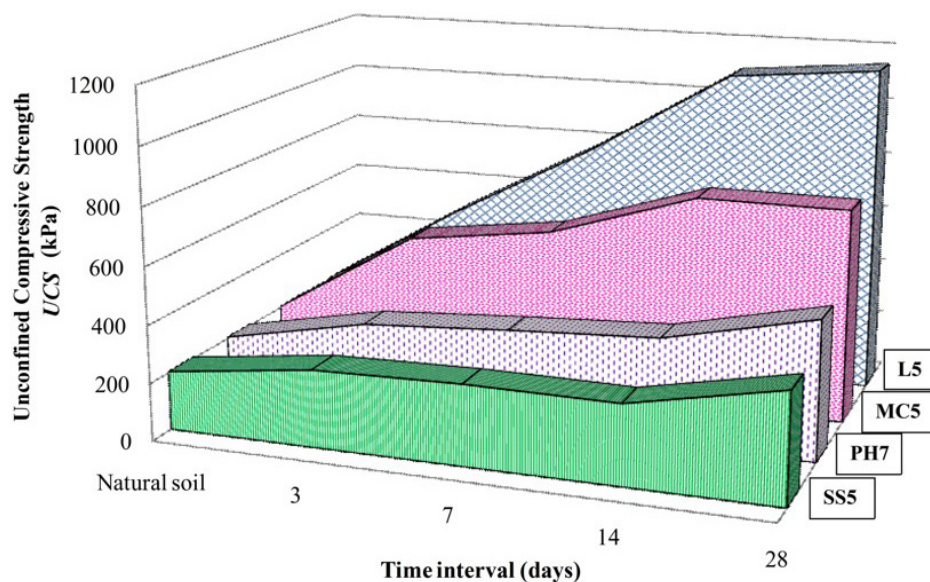


Figure 3. Comparison of the trend of change in UCS over time after the chemical stabilization of clayey soil with the corresponding optimal content for each of the considered reagents.

During the test, changes in shear stress (τ) and horizontal strain (ϵ_h) along the failure plane were recorded, which is illustrated in Figure 4. The diagrams represent the dependence of these values at a normal stress of 50 kPa, 100 kPa, and 200 kPa, after 3 days and after 28 days since the treatment of the clayey soil with the selected reagents.

The shear stress values at failure (τ) of the natural clayey soil for the applied normal stresses (σ) of 50 kPa, 100 kPa, and 200 kPa were 30.9 kPa, 49.8 kPa, and 78.1 kPa, respectively. It can be seen from Figure 4 that the shear strength increased over time in the case of all the selected reagents. All the maximum shear strength values were obtained at the test time of 28 days after the treatment,

at a normal stress of 200 kPa. Specifically, in the case of adding the optimal reagent content, the maximum shear stress value was:

- for lime (L5, Figure 4a): 167.8 kPa;
- for magnesium carbonate (MC5, Figure 4b): 119.9 kPa;
- for sodium silicate (SS5, Figure 4c): 114.0 kPa;
- for potassium hydroxide (PH7, Figure 4d): 109.4 kPa.

Furthermore, from the same figure it is clearly seen that the τ values recorded on the samples with lime addition (Figure 4a) represent the highest values of all the tested samples (Figures 4b-d). In addition to the significantly higher shear stress values, the shape of the diagrams has a pronounced peak indicating a brittle failure.

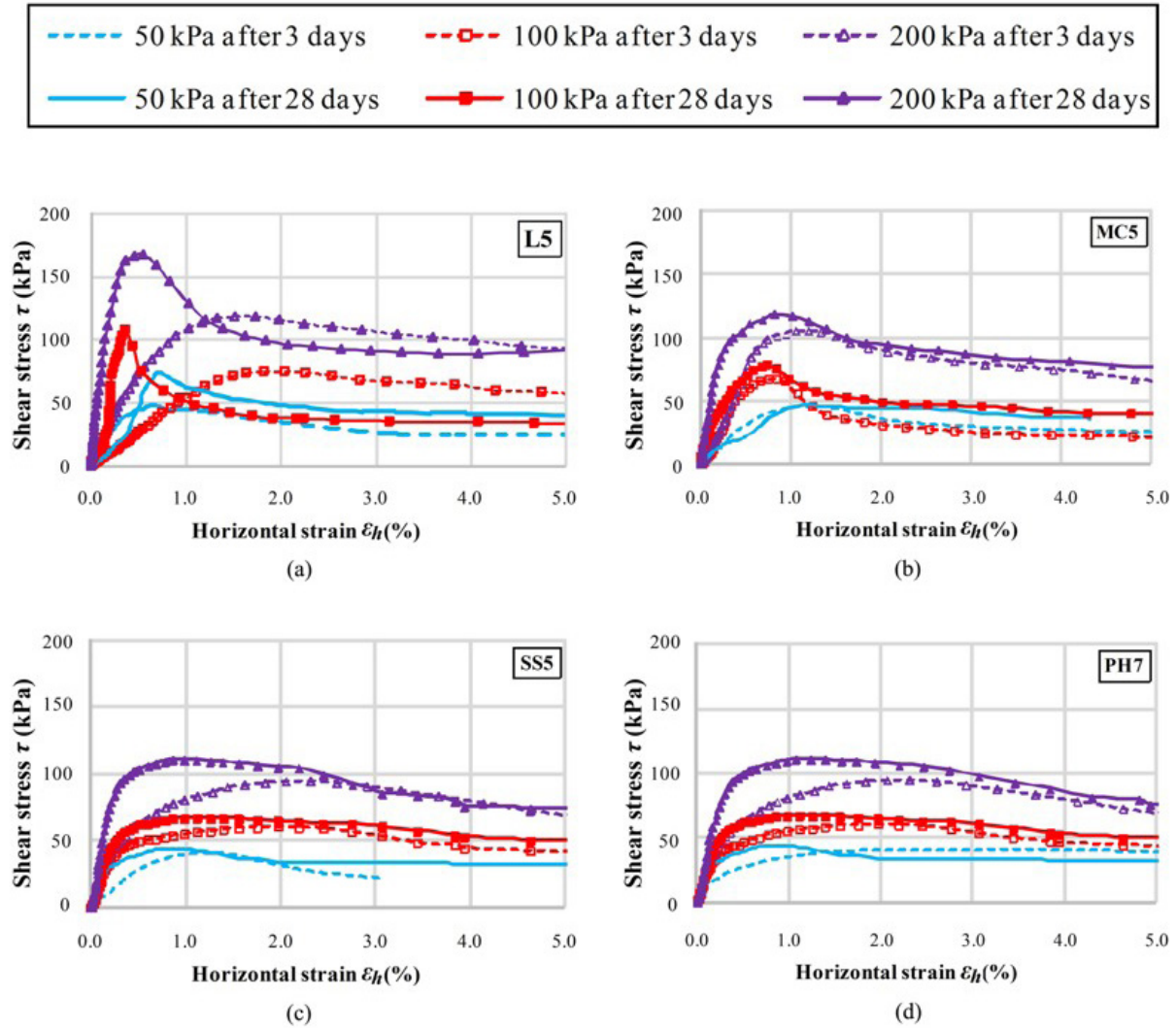


Figure 4. Shear stress–horizontal strain behavior at normal stress of 50 kPa, 100 kPa, and 200 kPa, after 3 days and after 28 days since the treatment of clayey soil with the optimal reagent content for each of the selected reagents: (a) lime, (b) magnesium carbonate, (c) sodium silicate, and (d) potassium hydroxide.

Considering the alternative reagents, only the sample with the addition of magnesium carbonate (MC5, Figure 4b) indicated the occurrence of a brittle failure. All other samples indicated a ductile failure (Figures 4c, d), i.e. without a large change in shear force, there was a significant displacement along the failure plane.

The trends of changes in the values of shear strength parameters (cohesion (c') and internal friction angle (ϕ') in relation to the natural state of the clayey soil ($c' = 16.9$ kPa and $\phi' = 16.5^\circ$) are given in Figure 5.

The lime-based reagent (L5) significantly contributed to the increase in shear strength parameters compared to the natural state of the soil. The improvement of these soil properties is noticeable for all the test time intervals. The trend of increasing shear strength parameter values with time is almost linear, both in terms of the increase in the value

of cohesion (after 28 days, the increase in the value of c' was 2.5 times) and in terms of the increase in the angle of internal friction (after 28 days, the increase in the value of ϕ' was two times).

With regard to alternative materials, approximate values of cohesion (in the range from 18.5 kPa to 26.5 kPa) and internal friction angle (in the range from 20.9 kPa to 25.8 kPa) were recorded for all the considered curing periods.

The greatest improvement in the strength parameters was recorded with the addition of magnesium carbonate (MC5), with an increase in both c' and ϕ' values of about 50% after 28 days. An increasing trend of the angle of internal friction with time can also be observed, whereas the value of cohesion registered 3 days after the treatment remained almost unchanged.

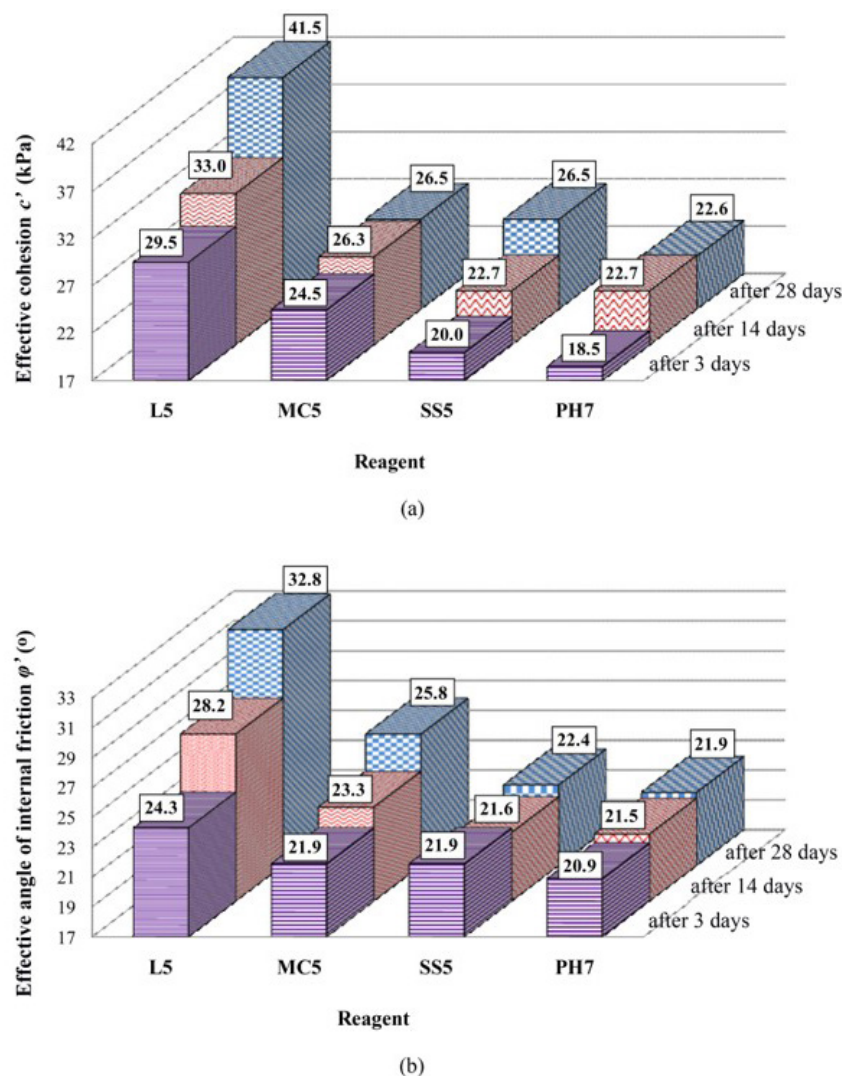


Figure 5. Comparison of the trend of change over time after the chemical stabilization of clayey soil with the corresponding optimal content for each of the selected reagents: (a) effective cohesion c' and (b) effective angle of internal friction ϕ' .

In the case of potassium hydroxide (PH7), after 3 days since the treatment, the values of cohesion and internal friction angle were slightly increased compared to the values of the parameters for the natural clayey soil. After 14 and 28 days since the treatment, the value of cohesion constantly increased (up to the value reached also with the addition of magnesium carbonate after 28 days), whereas the value of internal friction angle remained unchanged.

The addition of sodium silicate (SS5) also contributed to the improvement of the shear strength parameter values compared to the natural state of the soil – the results for the cohesion value after 14 days and 28 days were quite similar, whereas the increase in the value of the angle of internal friction was already evident during the first 3 days after the treatment and the value did not significantly change over time (the increase in both the c' value and the ϕ' value was about 35% after 28 days).

Figure 6 shows the Coulomb–Mohr failure lines for the natural clayey soil, as well as for all the considered reagents obtained by direct shear tests on samples tested 3, 14, and 28 days after the clayey soil treatment. Regarding cohesion, which represents the intercept of the Coulomb–Mohr failure line with the shear stress axis (τ), it can also be observed that the value of cohesion in the case of adding lime increases significantly over time, whereas in the case of other reagents a slightly lower increase in cohesion values was achieved.

On the other hand, there were significant changes in the value of the angle of internal friction, which represents the slope of the Coulomb–Mohr line. The illustrated lines for each of the considered reagents have a noticeably higher slope compared to the slope of the line for the natural clayey soil. In the case of lime addition, a significant change in the slope of the Coulomb–Mohr line was observed, whereas in the case of alternative reagents there is no significant change in the Coulomb–Mohr line slope.

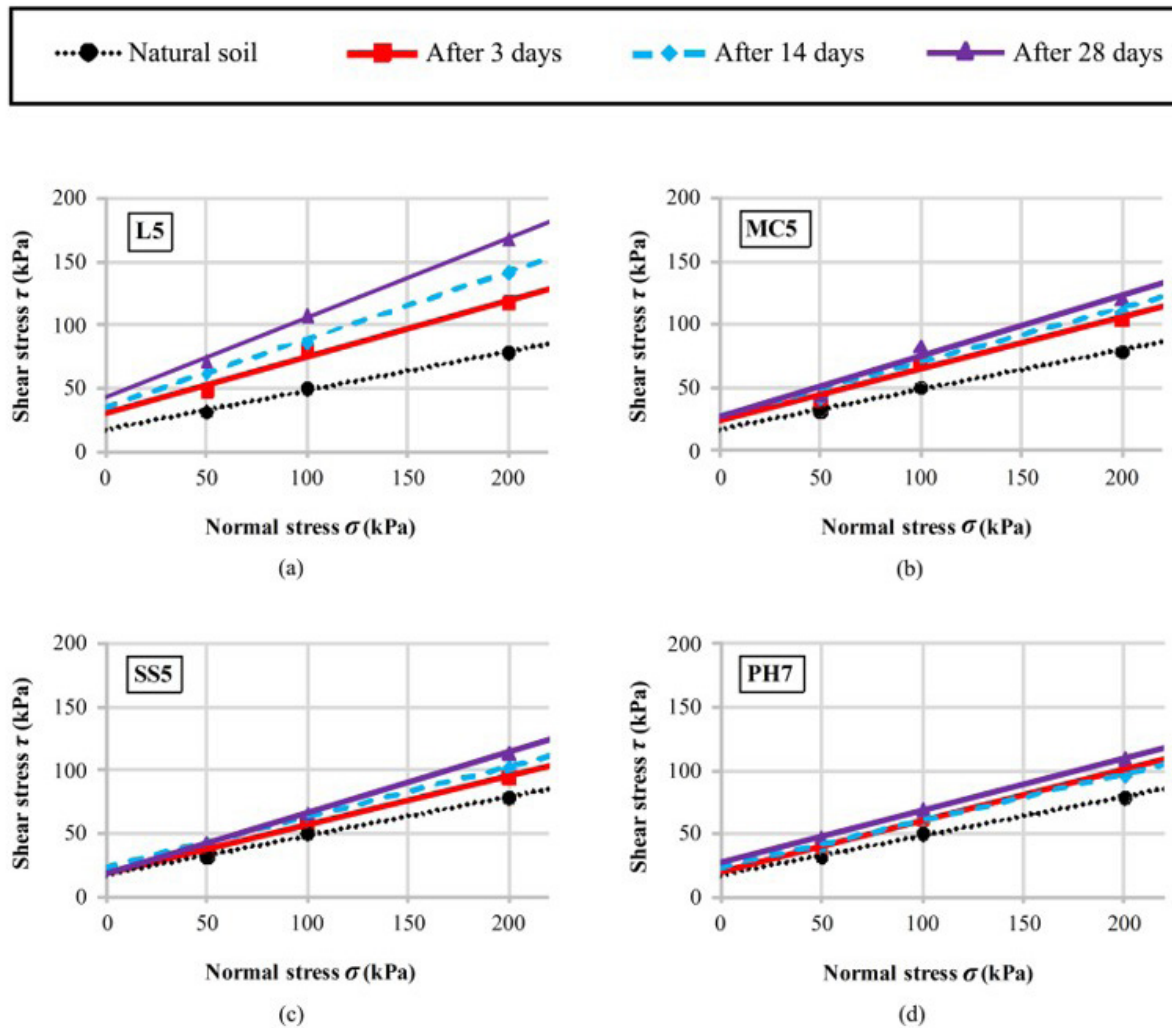


Figure 6. Comparison of the effects of each of the selected reagents on the shear strength parameters of clayey soil: (a) lime, (b) magnesium carbonate, (c) sodium silicate, and (d) potassium hydroxide.

4. Conclusions

In this research, a clayey soil of unfavorable physical and mechanical properties in its natural state was examined in laboratory before and after chemical stabilization, considering a variety of reagents, different percentages of reagents in the mixture with clay, as well as different time intervals after stabilization.

The results presented in this paper confirmed that the improvement of clayey soil properties can be successfully achieved by applying lime, magnesium carbonate, sodium silicate, and potassium hydroxide as reagents on the clayey soils. Each of the considered reagents added to the natural clayey soil contributed to an increase in the values of physical and mechanical properties of the clayey soil and thus enabled their application for various practical construction purposes.

Modification of the values of Atterberg limits, *UCS*, and shear strength parameters of the clayey soil were tested. The most important findings are as follows:

- None of the selected reagents contribute to a significant increase in the *LL* value, regardless of the content of the reagent in the mixture, as well as the time elapsed since the soil treatment. On the other hand, each of the selected reagents results in an increase in the value of *PL*. This ultimately yields a decrease in the value of *PI*, particularly over time, thus enabling the treated clayey soil to remain in a semi-solid state of consistency even with a higher water content, which has a favorable effect on its use for practical purposes in construction.
- From the aspect of increasing the *UCS* values of the treated clayey soil, it has been proven that lime and magnesium carbonate are the most effective reagents. On the other hand, sodium silicate and potassium hydroxide show a significantly weaker effect in terms of improving the *UCS* value of the clayey soil. Based on the results related to *UCS* values, it was concluded

that the optimal content of reagent for the treated soil is 5% in the case of applying lime, magnesium carbonate, or sodium silicate, and 7% in the case of applying potassium hydroxide.

- During the direct shear test, changes in the failure mode of the treated soil can be observed based on the $\varepsilon_h - \tau$ diagram. Namely, the samples with the addition of lime and the samples with the addition of magnesium carbonate had a more pronounced peak at failure compared to the other reagents, which indicates a brittle failure of the soil. In the case of sodium silicate and potassium hydroxide, ductile failure was recorded.
- An increase in the soil shear strength is observed with the addition of the optimal stabilizer content for each of the selected reagents. The greatest increase in the shear strength parameter values is achieved with the addition of lime (after 28 days, the increase in the value of c' was 2.5 times and the increase in the value of ϕ' was two times). With regard to alternative materials (magnesium carbonate, sodium silicate, and potassium hydroxide), lower increases of cohesion and internal friction angle were achieved with mutually similar values.

After conducting this research, among the tested reagents, lime with a 5% share in the mixture with the clayey soil proved to be the most effective reagent. Nevertheless, this study confirmed that alternative additives, such as magnesium carbonate, sodium silicate, and potassium hydroxide, are also substantially promising as sustainable stabilizing materials considering the examined physical and mechanical properties of the clayey soil.

In addition to significantly improving the properties of soil materials, lime is also the most cost-effective stabilizer. Compared to sodium silicate and potassium hydroxide, the costs of using lime as a chemical stabilizer are several times lower, whereas compared to magnesium carbonate they are even several dozen times lower. All reagents used in this study are commercially available and are environmental-friendly materials that do not pollute the environment. Because of that there is no health risk for personnel in laboratory during testing.

It should be noted, however, that all the results presented in this paper relate to the local clayey soil of a specific mineralogical composition. Therefore, there is a need for more comprehensive further research, which would analyze a wider range of soils with different mineralogical compositions.

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Declaration of interest

The authors have no conflicts of interest to declare. All co-authors have observed and affirmed the contents of the paper and there is no financial interest to report.

Authors' contributions

Elefterija Zlatanović: conceptualization, methodology, investigation, resources, data curation, formal analysis, writing – original draft preparation, writing – review and editing, visualization, supervision, project administration, funding acquisition. Nemanja Marinković: conceptualization, methodology, investigation, resources, data curation, formal analysis, writing – original draft preparation, writing – review and editing, visualization, supervision. Nebojša Davidović: conceptualization, methodology, investigation, resources, data curation, formal analysis, writing – original draft preparation, writing – review and editing, visualization, supervision. Zoran Bonić: conceptualization, methodology, investigation, resources, data curation, formal analysis, writing – original draft preparation, writing – review and editing, visualization, supervision, project administration. Nikola Romić: conceptualization, methodology, investigation, resources, data curation, formal analysis, writing – original draft preparation, writing – review and editing, visualization, supervision.

Data availability

The datasets generated analyzed in the course of the current study are available from the corresponding author upon request.

Declaration of use of generative artificial intelligence

This work was prepared independently, without the assistance of generative artificial intelligence (GenAI).

List of symbols and abbreviations

c	Cohesion
c'	Effective cohesion
s	Swelling
CBR	California bearing ratio
C_c	Coefficient of curvature
C_u	Coefficient of uniformity
EN	European Norms
G_s	Particle density
ISO	International Organization for Standardization
ISS	Institute for Standardization of Serbia
L	Lime
L3	Soil with 3% of lime in mixture
L5	Soil with 5% of lime in mixture

L7	Soil with 7% of lime in mixture
LL	Liquid limit
MC	Magnesium carbonate
MC3	Soil with 3% of magnesium carbonate in mixture
MC5	Soil with 5% of magnesium carbonate in mixture
MC7	Soil with 7% of magnesium carbonate in mixture
MDD	Maximum dry density
Mv	Modulus of compressibility
OMC	Optimal moisture content
PH	Potassium hydroxide
PH3	Soil with 3% of potassium hydroxide in mixture
PH5	Soil with 5% of potassium hydroxide in mixture
PH7	Soil with 7% of potassium hydroxide in mixture
PI	Plasticity index
PL	Plastic limit
SRPS	Designation of standards and related documents issued by the Institute for Standardization of the Republic of Serbia
SS	Sodium silicate
SS3	Soil with 3% of sodium silicate in mixture
SS5	Soil with 5% of sodium silicate in mixture
SS7	Soil with 7% of sodium silicate in mixture
UCS	Unconfined compressive strength
ε_h	Horizontal strain in direct shear test
σ	Normal stress
τ	Shear stress
φ	Internal friction angle
φ'	Effective internal friction angle

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Influence of anisotropic stress states on soil-structure interface behavior: a new experimental approach in a modified hollow cylinder apparatus

Sabine Gehring^{1#} , Hans Henning Stutz¹ 

Article

Keywords

Interface tests
Stress anisotropy
Sand
Hollow cylinder triaxial device

Abstract

To determine the influence of soil-structure interface behavior on the bearing capacity and deformations of geotechnical structures, suitable testing methods capable of simulating in situ conditions are required. This article introduces a novel test method for soil-structure interface tests. A hollow cylinder triaxial device is modified to conduct interface tests with four independent control variables. This enables investigations of the influence of different anisotropic stress states on the contact shear behavior of granular material. First test results on Karlsruhe fine sand are presented with a variation of the lateral normal stresses acting on the plane orthogonal to the shear (i.e., contact) plane. These tests are conducted in two modes: in mode A long samples ($d_o = h$) are used whereas in mode B short samples ($2(d_o - d_i) = 1/h$) are used. The results of the test series in mode A show that the failure occurs either in the soil (on the plane with the highest ratio of the shear and normal component of the stress vector τ / σ_N) or on the contact plane (where the shear strength is reduced). The failure in the test series with mode B is restricted to the horizontal plane due to sample geometry. The findings will be utilized in the future to test and develop various modeling methods for soil-structure interfaces.

1. Introduction

To determine the loading capacity and serviceability of a geotechnical structure, the shear strength and stiffness of the soil must be tested. Many projects include rigid structures such as piles, foundations, tunnels and retaining walls (Figure 1a). In these cases, the contact shear behavior between the soil and the rigid structure is crucial to the system's behavior: For example, the surface roughness of a pile, which rules the contact shear behavior, has a major impact on its load-bearing capability (Jardine et al., 1993). Thus, a holistic understanding of the behavior of geotechnical structures requires an in-depth understanding of soil-structure interface behavior.

This inspired several authors to investigate the soil-structure contact shear behavior (e. g., Potyondy, 1961; Kishida & Uesugi, 1987; Stutz et al. 2018). The following major contributing factors have been identified: normal stress on the shear plane (Meier et al., 2023), soil density (Wernick, 1978) and ratio of the surface roughness to the grain size of soil (Uesugi & Kishida, 1986). The maximum shear stress which can be mobilized in interface tests increases with an increasing surface roughness and is limited by the soil shear strength (e.g., Kishida & Uesugi, 1987; Konkol, 2025).

If the surface roughness of an interface is so high that the soil shear strength is reached in interface tests this situation is often called a fully rough interface.

These conclusions were made from test results generated in different devices for interface testing (see Figure 2). For a clear description of different boundary conditions and their influence on the sample's stress state, the following definition of the stress components of the stress tensor of the sample are used in the current work according to Figure 1b:

$$\sigma_{ij} = \begin{bmatrix} \sigma_{11} & \tau_{12} & \tau_{31} \\ \tau_{12} & \sigma_{22} & \tau_{32} \\ \tau_{13} & \tau_{23} & \sigma_{33} \end{bmatrix} = \begin{bmatrix} \sigma_{PP} & \tau_{12} & \tau_{31} \\ \tau_{12} & \sigma_{OP,1} & 0 \\ \tau_{13} & 0 & \sigma_{OP,2} \end{bmatrix} \quad (1)$$

σ_{PP} is the normal stress on the plane parallel to the contact surface. The orthogonal planes are chosen such that there is no shear stress τ_{23} and the normal stress components on these planes are called $\sigma_{OP,1}$ and $\sigma_{OP,2}$.

Most of the laboratory interface tests are conducted using modified direct shear tests (Figure 2a, e.g., Potyondy, 1961; Frost & Han, 1999; Stutz & Martinez, 2021) or simple shear test devices (Figure 2b, e.g., Kishida & Uesugi, 1987). In these tests (as well as in modified ring shear tests, Figure 2c,

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e.g., Lemos & Vaughan, 2000), different boundary conditions are applied on the plane parallel to the contact plane (“parallel plane” in Figure 1b). For example, the stress component σ_{PP} of the stress tensor in the sample (see Equation 1 and Figure 1b) can be controlled. On the “orthogonal planes” (Figure 1b), the deformation is restricted by the frames or stacked rings in the devices. Thus, the stress components $\sigma_{OP,1}$ and $\sigma_{OP,2}$ of the tensor cannot be controlled and are unknown. Because of those test boundary conditions, information regarding the impact of various boundary conditions on the orthogonal plane is scarce. First tests toward the influence of the three-dimensional stress state were conducted by Corfdir et al. (2004). They developed a new test device and controlled one normal stress component $\sigma_{OP,1}$ on an orthogonal plane in the sample in addition to the normal stress component σ_{PP} on the parallel plane. Corfdir et al. (2004) used a rotating cylinder inside a hollow cylinder-shaped sample as contact plate for the interface (Figure 2d). With an annular confining cell, the normal stress on the shear

plane is applied and with an axial piston, one additional normal stress $\sigma_{OP,1}$ can be controlled. Another approach by Tejchman & Wu (1995) and Liu et al. (2024) is an inclined rigid surface in a triaxial or biaxial device (see Figure 2e). In such tests, the normal stress components $\sigma_{OP,1}$ and $\sigma_{OP,2}$ on the orthogonal planes can be calculated, but however, not independently controlled.

Investigations on the influence of different stress states and the entire 3D-stress tensor on contact shear behavior are not only crucial for gaining a better understanding of interface behavior, but is also useful for evaluating different modeling methods for soil-structure interaction. In general, two different approaches to develop constitutive models for interfaces exist in the literature (see Figure 1c). The first approach uses only the stress vector on the shear plane and the relative displacement at the interface (e.g., Ghionna & Mortara, 2002; Stutz et al., 2014). Because soil shear behavior and contact shear behavior are similar, the second approach uses the idea of deriving interface models from

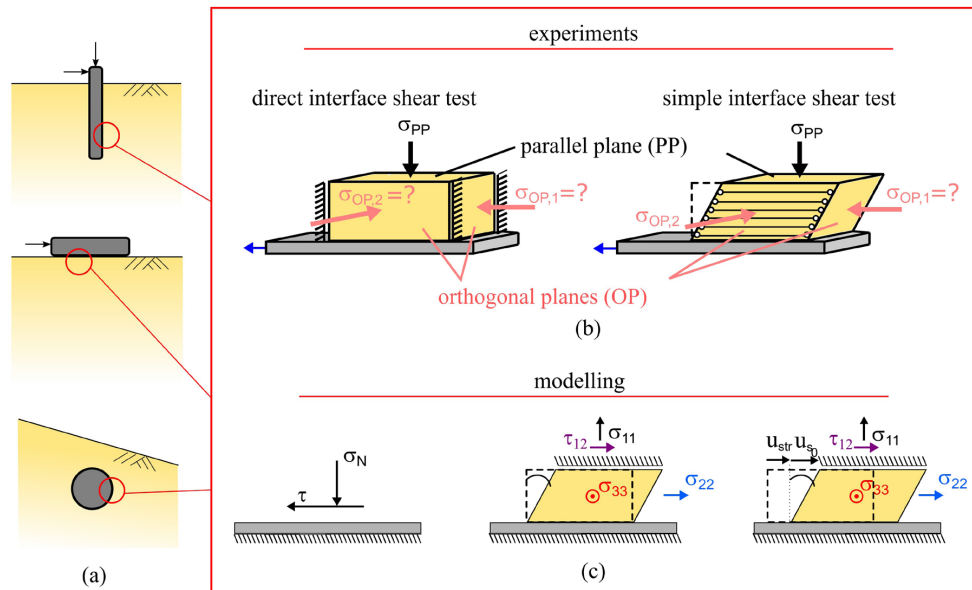


Figure 1. Interface shear behavior: (a) practical examples, (b) unknown stress on orthogonal plane for common interface tests, (c) modeling approaches for interface behavior.

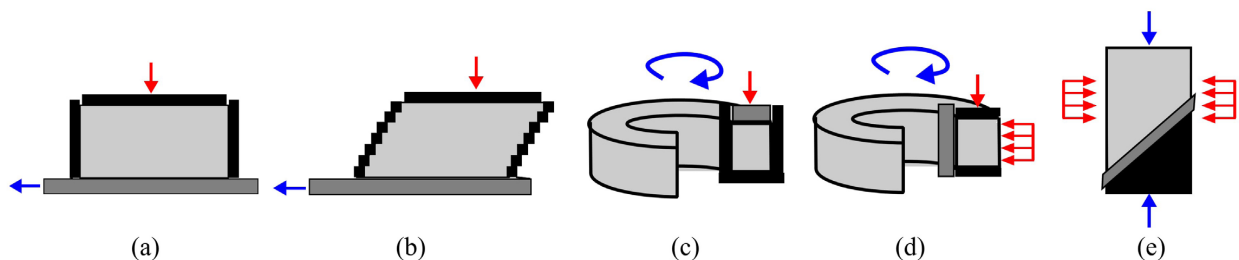


Figure 2. Different test setups for interface tests: (a) modified direct shear apparatus, (b) modified simple shear device, (c) modified ring shear apparatus, (d) rotating interface in sample ring and annular confining cell, (e) inclined contact surface in triaxial or biaxial device.

existing soil models by reducing the shear strength of the model (e.g., Gehring et al., 2023). This is supported by the experimental observation, that the relative displacement between contact surface and soil consists of localized shear strains in a shear band in the soil (DeJong et al., 2006). Using this method, the entire 3D stress tensor in the sample is considered. The last-mentioned approach for contact modeling is based on different assumptions concerning the stress state at the interface: equal normal stresses ($\sigma_{PP} = \sigma_{OP,1} = \sigma_{OP,2}$, Arnold & Herle, 2006), negligible deformations at the orthogonal plane ($\sigma_{PP} \neq \sigma_{OP,1} = \sigma_{OP,2}$, Stutz et al., 2017) or a coupling with neighboring soil elements in FEM ($\sigma_{PP} \neq \sigma_{OP,1} \neq \sigma_{OP,2}$, Staubach et al., 2022). Furthermore, recent research incorporated the consideration of stick and slip conditions between the surface and the soil in combination with the second approach (Niebler et al., 2025). Thereby, the total displacement of the interface is divided into the displacement due to sliding of the contact plate u_{str} and the displacement due to the shearing of the soil u_{so} (see Figure 1c). To choose a suitable method and verify or contradict the existing approaches, experimental evidence with various stress states is required.

Despite these needs, data on the influence of different stress states and boundary conditions on the orthogonal plane is limited and existing methods can only control one stress component on an orthogonal plane $\sigma_{OP,1}$. Therefore, a novel test method is presented here. This method allows to control two stress components $\sigma_{OP,1}$ and $\sigma_{OP,2}$ on planes orthogonal to the shear plane. To conduct such interface tests, a hollow cylinder triaxial device is modified with a contact plate. The modified device, due to its four independent control parameters, offers diverse opportunities for interface testing with various stress states. The first test series on Karlsruhe fine sand with different stress conditions and contact plates with

varying roughness properties are conducted and interpreted concerning the failure mechanism.

2. Materials and test setup

2.1 Test material (soil and surface)

The experiments were conducted on medium dense samples of Karlsruhe fine sand. A grain size distribution for Karlsruhe fine sand is shown in Figure 3a and index properties are listed in Table 1. The grain size distribution is uniformly graded. The grain shape can be characterized with an aspect ratio of 1.289 (ratio between longest length of the grain and the length orthogonal to it) and is illustrated in Figure 3b. For the sample preparation, the membranes are placed in a mold and the sand is air pluviated between the membranes (Figure 3c). A nozzle with a diameter of 4 mm is used for pluviation and is moved with a constant distance to the sample top.

The samples have an outer diameter of 100 mm, and inner diameter of 60 mm and a height of 100 mm (mode A). In the test series with short samples (mode B), the sample height is reduced to 20 mm. In the modified hollow cylinder triaxial device for interface tests, ring plates are used as contact surfaces (for the test setup see Section 2.2).

Table 1. Index quantities of Karlsruhe fine sand (Wichtmann & Triantafyllidis, 2016).

Karlsruhe fine sand	
grain density ρ_s	2.65 g/cm ³
e_{min}	0.677
e_{max}	1.054
Aspect ratio	1.289

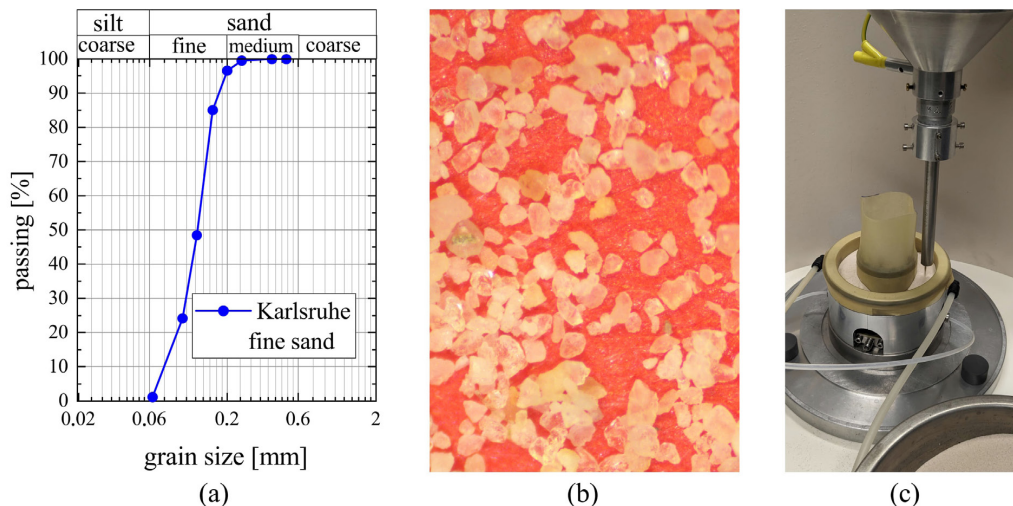


Figure 3. Karlsruhe fine sand (KFS). (a) grain size distribution of Karlsruhe fine sand, (b) microscopy of grains, (c) sample preparation (air pluviation) exemplary for mode B.

Steel rings with different roughness properties are used in the presented test series. They are displayed in Figure 4a, b and c and listed in Table 2.

The roughness property R_a is equivalent to the center line average (CLA) and is defined as the average deviation from the mean line of the surface (see Figure 4d). It is important to mention, that the surfaces of the contact plates K1 and K3 contain two porous stones and six screws (Figure 4a). The measured roughness was obtained between the screws. K11 was developed as an improved contact plate without screws. This provides a uniform roughness across the entire contact area and is therefore consistent with the roughness measurements (Figure 4b). The roughness R_a of the plates was measured with a digital microscope on three spots on the plates and averaged thereafter.

2.2 Test setup and procedure

In the modified hollow cylinder triaxial device, a ring-shaped sample is sheared against a contact plate by

rotating the bottom plate against the top plate (see Figure 5). In common hollow cylinder triaxial tests, both end plates are fully rough and manufactured with steel fins or coarse grains (Broms & Casbarian, 1965; Hight et al., 1983). The aim is to ensure a transfer of the displacement of the end plates to the sample with nonslip traction and full mobilization of the soil strength. To conduct interface tests with this device, a changeable top plate was manufactured. In the modified device, interface tests with contact plates with different

Table 2. Overview of used contact plates.

contact plate	roughness R_a (excluding screws)	manufacturing of surface
K0	$>230\mu\text{m}$, fully rough	glued sand grains (with screws)
K3	$>0.7\mu\text{m}$	Fine sand blasted (with screws)
K11	$\approx 0.7\mu\text{m}$	polished (no screws)

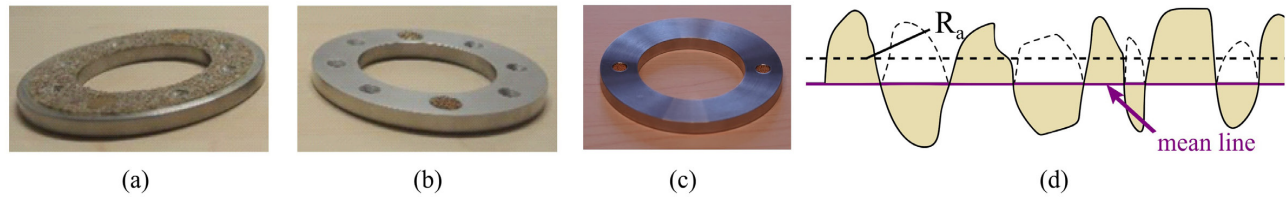


Figure 4. Photographs of contact plates K0 (a), K3 (b) with screws and K11 (c) without screws, (d) schematic of average roughness value R_a .

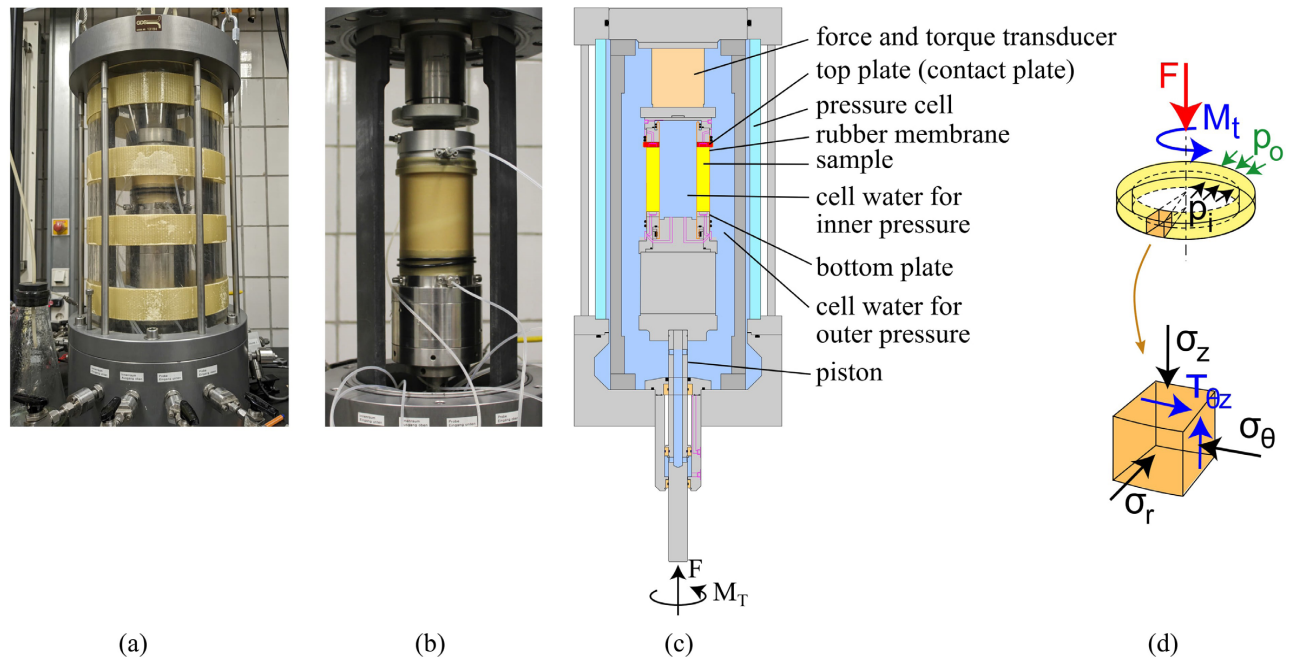


Figure 5. Modified hollow cylinder device for interface tests. (a) photograph of mode B with pressure cell, (b) photograph of mode A without pressure cell, (c) schematic of setup mode A, (d) loading and stress state of the sample.

roughness properties can be performed. The top plate was further enhanced with a screw-free variant (K11) to obtain a uniform surface. The sample is covered with membranes on the outer and inner surface area. The outer and inner cell water is separated, so it is possible to control four independent control parameters: the axial force or displacement, the torsional momentum or rotation, the outer cell pressure, and the inner cell pressure. Two of them are used to apply a normal stress on the shear plane (axial force) and a shear displacement to the interface (torsion). This leaves two more control parameters to control the normal stresses $\sigma_{OP,1}$ and $\sigma_{OP,2}$ on the orthogonal planes and enables studies of the interface behavior with complex stress states.

In Figure 5d a schematic soil element with the components of the stress tensor in the sample is displayed at the bottom with the globally measured forces and pressures acting on the whole sample above. The stress elements form the following stress tensor and its transfer to the stress tensor defined above the interface:

$$\sigma_{ij} = \begin{bmatrix} \sigma_z & 0 & \tau_{\theta z} \\ 0 & \sigma_r & 0 \\ \tau_{z\theta} & 0 & \sigma_\theta \end{bmatrix} = \begin{bmatrix} \sigma_{PP} & 0 & \tau_{31} \\ 0 & \sigma_{OP,1} & 0 \\ \tau_{13} & 0 & \sigma_{OP,2} \end{bmatrix} \quad (2)$$

The following equations show the relationships between the global measured values and the stress components for average stresses with the assumption of equal work of the global measured forces and the stresses acting in the sample (see Saada, 1988):

$$\sigma_z = \frac{F_{ax}}{\pi(r_o^2 - r_i^2)} + \frac{p_o r_o^2 - p_i r_i^2}{r_o^2 - r_i^2} \quad (3)$$

$$\sigma_r = \frac{p_o r_o + p_i r_i}{r_o + r_i} \quad (4)$$

$$\sigma_\theta = \frac{p_o r_o - p_i r_i}{r_o - r_i} \quad (5)$$

$$\tau_{\theta z} = \frac{3M_t}{2\pi(r_o^3 - r_i^3)} \quad (6)$$

From these stress components, the principal stresses can be calculated:

$$\sigma_1 = \frac{\sigma_z + \sigma_\theta}{2} + \sqrt{\left(\frac{\sigma_z - \sigma_\theta}{2}\right)^2 + \tau_{\theta z}^2} \quad (7)$$

$$\sigma_2 = \sigma_r, \quad (8)$$

$$\sigma_3 = \frac{\sigma_z + \sigma_\theta}{2} - \sqrt{\left(\frac{\sigma_z - \sigma_\theta}{2}\right)^2 + \tau_{\theta z}^2} \quad (9)$$

The ratio of the principal stresses is often depicted with the coefficient b :

$$b = \frac{\sigma_2 - \sigma_3}{\sigma_1 - \sigma_3} \quad (10)$$

The Roscoe invariants are defined as:

$$\text{mean pressure } p = \frac{1}{3}(\sigma_z + \sigma_r + \sigma_\theta) \quad (11)$$

$$q = \sqrt{\frac{\text{deviator stress}}{2}} = \sqrt{\frac{\sigma_z^2 + \sigma_r^2 + \sigma_\theta^2 - \sigma_z \sigma_r - \sigma_z \sigma_\theta - \sigma_r \sigma_\theta + 3\tau_{\theta z}^2}{2}} \quad (12)$$

$$\text{Lode angle } \theta = \cot \sqrt{3} \cdot \frac{b}{2-b} \quad (13)$$

After the sample preparation (see Section 2.1) the samples were saturated. For this purpose, the samples were first flushed with CO₂, flushed with water under atmospheric pressure, then flushed with water and a backpressure of 200 kPa and in the end the backpressure was increased to 500 kPa. The saturation was verified with a B-value greater than 0.97. After this, the samples were consolidated to the initial stress state. With an open drainage the axial force F_{ax} and the outer and inner cell pressures were increased up to the desired values. The reached effective values of σ'_z, σ'_r and σ'_θ were kept constant during the test for all experiments.

All tests were conducted with the same vertical stress $\sigma'_z = \sigma'_{pp}$ of 100 kPa (normal stress on the parallel plane). The normal stresses on the orthogonal plane (radial stress $\sigma'_r = \sigma'_{OP,1}$ and tangential stress $\sigma'_\theta = \sigma'_{OP,2}$) were varied from test to test. These stresses were kept constant during the entire test. After reaching the test specific initial stress state, the drained contact shearing phase was started by rotating the bottom endplate. The bottom plate was rotated with a constant velocity of $\theta = 0.2$ deg/min. This leads to an increase of the shear stress on the horizontal plane $\tau_{\theta z}$.

3. Results and discussion

All tests are conducted with medium dense samples with a target relative density $I_{D,0} \approx 0.6$ after the sample preparation. The actual relative density can be found in Table 3 together with the boundary conditions of the presented tests. $I_{D,0}$ and e_0 represents the initial density and the void ratio directly after the sample preparation and before the consolidation.

3.1 Influence of membrane stiffness

During the shear phase (torsion) of the test, not only the sample is deformed but also the membranes. Since the membranes are fixed at the bottom and top caps, they are deformed due to rotation of the end plates and therefore contribute to the globally measured forces.

In test series B01:K3 and B02:K0 experiments with short samples with a height of 20 mm are conducted. Especially for these tests and a rotation of the bottom plate up to 40°, the influence of the membrane stiffness cannot be neglected: The measured moment during torsion tests results not only from the mobilized friction on the sample but also from stresses in the membrane.

In the literature, one method to correct the measured data results from the shear stress emerging from elastic membrane deformation due to torsion:

$$\tau_M = \frac{t_M E_M (r_o^3 + r_i^3) \theta}{(r_o^3 - r_i^3) h} \quad (14)$$

with the global measured shear stress due to membrane deformation τ_M , the thickness t_M and the elastic modulus E_M of the membranes, the rotation angle θ and the height h and inner r_i and outer radius r_o of the specimen (e.g. Koseki et al., 2005; Kiyota et al. 2008). Koseki et al. (2005) and Kiyota et al. (2008) used dummy tests with water-filled membranes to validate this relationship and to develop further correction formulas for large deformations. Especially when large deformations occur, these tests with water-filled membranes show different deformation patterns than the drained tests in the present work. Therefore, a new correction method is developed to compensate for the effect of the membrane stiffness on the test results. Tests on soil samples with different membranes (V15 and V17) are used

to capture the actual membrane (or sample) deformation and a mathematical correction is developed. The following membrane types were used here:

- Latex membrane, stiffness $E_{\text{latex}} = 1387 \text{ kPa}$, thickness $t_{\text{latex}} = 0.55$ (average of inner and outer membrane in test V15)
- Butyl membrane, stiffness $E_{\text{butyl}} = 1706 \text{ kPa}$, thickness $t_{\text{butyl}} = 0.64$ (average of inner and outer membrane in test V17)

The membrane stiffness was obtained with tension tests using dummy samples to maintain the shape of the membrane and restrict the lateral strain of the membrane. The calculated shear stress from the globally measured momentum of tests V15 and V17 is shown in Figure 6. The tangential displacement s_t represents the displacement of a point on the average circumference on the bottom plate along its average circumference. The shear stress of both tests increases and reaches at approximately 5 mm tangential displacement a more shallow, linear path. The linear increase of test V17 with butyl membranes follows a steeper linear path and therefore reaches a higher shear stress. This shows a larger influence of the membrane stiffness resulting from a larger stiffness of the butyl membranes. From this difference, the stress induced by membrane deformation is calculated for these tests and transferred to a general correction function.

If the membrane deformation is similar in two tests and in the elastic range, the additional global shear stress or measured torque resulting from the membrane stiffness is proportional to the thickness and stiffness of the membranes (see Equation 14). With two different types of membrane with different thickness and stiffness, the ratio of the additional measured global shear stress can be estimated. Here, the global shear stress induced by the membrane stiffness in the test with latex membranes τ_M^{latex} and in the test with butyl membranes τ_M^{butyl} have the following relationship:

Table 3. Overview of presented tests.

series	No.	mode	contact plate	$I_{D,0}$	e_0	σ'_z	$\sigma'_\theta = \sigma'_r$	p'
A01:K11	V26	A	K11 (smooth)	0.59	0.83	100 kPa	50 kPa	67 kPa
A01:K11	V28	A	K11	0.58	0.837	100 kPa	100 kPa	100 kPa
A01:K11	V29	A	K11	0.61	0.82	100 kPa	150 kPa	133 kPa
A01:K11	V30	A	K11	0.61	0.82	100 kPa	150 kPa	133 kPa
A02:K0	V02	A	K0 (rough)	0.70	0.79	100 kPa	100 kPa	100 kPa
A02:K0	V31	A	K0	0.66	0.80	100 kPa	150 kPa	133 kPa
A02:K0	V32	A	K0	0.62	0.82	100 kPa	50 kPa	67 kPa
B01:K3	V08	B	K3 (less rough)	0.62	0.82	100 kPa	100 kPa	100 kPa
B01:K3	V09	B	K3	0.63	0.82	100 kPa	150 kPa	133 kPa
B01:K3	V10	B	K3	0.63	0.82	100 kPa	50 kPa	67 kPa
B01:K3	V21	B	K3	0.56	0.85	100 kPa	175 kPa	150 kPa
B02:K0	V15	B	K0 (rough)	0.76	0.77	100 kPa	100 kPa	100 kPa
B02:K0	V18	B	K0	0.49	0.87	100 kPa	150 kPa	133 kPa
B02:K0	V20	B	K0	0.65	0.81	100 kPa	50 kPa	67 kPa

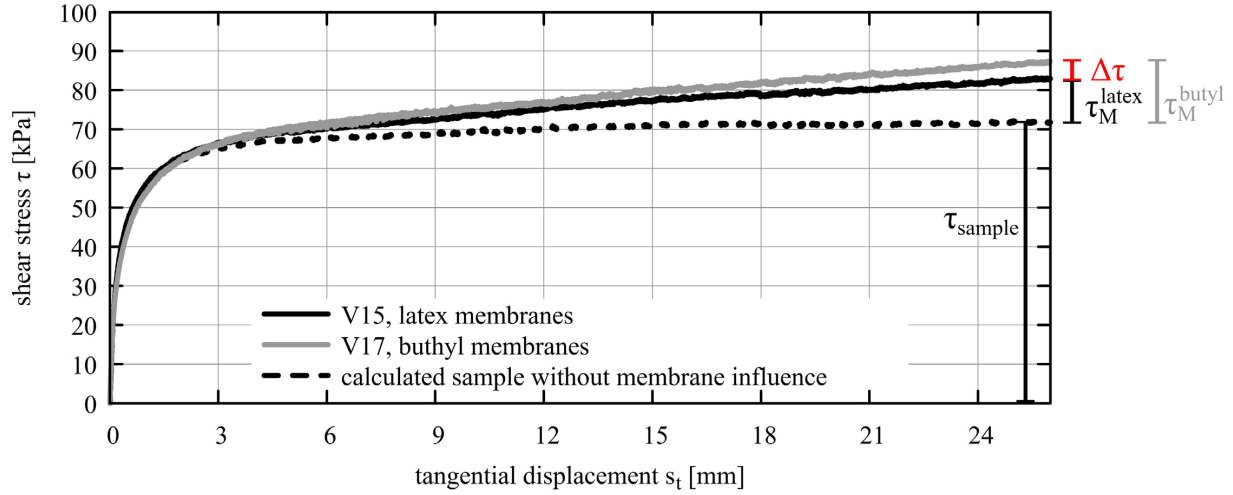


Figure 6. Test results of experiment V15 and V17 with different membrane types to develop a correction for the membrane stiffness.

$$\frac{\tau_M^{\text{butyl}}}{\tau_M^{\text{latex}}} = \frac{t_{\text{butyl}} E_{\text{butyl}} (r_o^3 + r_i^3) \theta}{(r_o^3 - r_i^3) h} \quad (15)$$

$$\frac{(r_o^3 - r_i^3) h}{t_{\text{latex}} E_{\text{latex}} (r_o^3 + r_i^3) \theta} = \frac{t_{\text{butyl}} E_{\text{butyl}}}{t_{\text{latex}} E_{\text{latex}}}$$

From the test results, only the difference of the total global measured shear stress can be calculated:

$$\tau_{\text{tot}}^{V17} - \tau_{\text{tot}}^{V15} = \Delta \tau \quad (16)$$

But with the assumption, that the measured shear stress contributed by the samples shear strength is the same for both tests, the calculated difference $\Delta \tau$ results only from the difference in the membrane stresses:

$$\tau_{\text{tot}}^{V17} - \tau_{\text{tot}}^{V15} = \Delta \tau = \tau_M^{\text{butyl}} - \tau_M^{\text{latex}} \quad (17)$$

There are still two unknown variables in Equation 17. To find their values, the equation is first expanded with a ratio:

$$\Delta \tau = \tau_M^{\text{butyl}} - \tau_M^{\text{latex}} = \frac{\tau_M^{\text{butyl}} \tau_M^{\text{latex}}}{\tau_M^{\text{latex}}} - \tau_M^{\text{latex}} \quad (18)$$

Now, it can be reduced to one unknown variable together with Equation 15:

$$\Delta \tau = \frac{t_{\text{butyl}} E_{\text{butyl}}}{t_{\text{latex}} E_{\text{latex}}} \tau_M^{\text{latex}} - \tau_M^{\text{latex}} \quad (19)$$

The additional measured shear stress induced by the membrane deformation in test V15 with the latex membrane can be calculated with:

$$\tau_M^{\text{latex}} = \frac{\Delta \tau}{\frac{t_{\text{butyl}} E_{\text{butyl}}}{t_{\text{latex}} E_{\text{latex}}} - 1} \quad (20)$$

The correction function is developed from the additional global shear stress induced from the latex membrane at the end of the test. The current global shear stress induced by the membrane is calculated as a proportional function of the rotation angle θ or the tangential shear displacement s_t . To address varying membrane thickness and stiffness, the correction can be adapted to the actual membrane thickness t_M and stiffness E_M :

$$\tau_M^{\text{latex}} = 0.389 \frac{\text{kPa}}{\text{mm}} \cdot \frac{t_M E_M}{0.55 \text{ mm} \cdot 1387 \text{ kPa}} s_t \text{ or}$$

$$\tau_M^{\text{latex}} = 0.272 \frac{\text{kPa}}{^\circ} \cdot \frac{t_M E_M}{0.55 \text{ mm} \cdot 1387 \text{ kPa}} \theta \quad (21)$$

This correction can be used for similar geometries and a rotation angle up to 40° and is applied to all tests presented here.

3.2 Influence of different stress states – tests with mode A (long samples)

With the membrane correction (Section 3.1) it is possible to determine the contact shear behavior at various stress states. Two representative tests in mode A (long sample) with contact plates with two different roughness properties are displayed in Figure 7. The evolution of the global shear

stress $\tau_{\theta z}$ of both tests is monotonic increasing and tends to an asymptotic value. V30 with a smooth plate (K11) reaches a lower maximum shear stress, then V02 with a rough plate (K0). The maximum shear stress of V30 with the smooth plate is reached at a larger displacement. This matches the findings in the literature for direct interface shear tests, that the contact shear strength increases with the roughness of the contact plate (e.g., Kishida & Uesugi, 1987; Martinez & Stutz, 2019) and will be discussed further in Section 3.5. In direct interface tests, the shear plane is forced to the horizontal plane by the test setup and the stress vector on the contact plane with normal stress σ_N and shear stress τ is analyzed. Whether this approach to analyze only the stress vector on the contact plane is suitable for the novel test method will be discussed along with the experimental results shown in Figure 8, as well as the implications for the description of the in situ behavior. Additionally, the fact that there is more fluctuation in the values of V30 than of V02 (Figure 7) will be discussed in Section 3.3.

Figure 8 displays the shear phase (torsion) of the test series A01:K11 and A02:K0. The graphs in Figure 8a and c belong to test series A01:K11 with a smooth contact plate, the graphs in Figure 8b and d belong to test series A02:K0 with a rough contact plate. The graphs a and b show the deviator stress q (light-colored) and the shear stress on the horizontal plane $\tau_{\theta z}$ (black lines) over the effective mean pressure p' .

During the shear phase, the mean effective pressure is constant because of the constant controlled normal stresses σ_z , σ_r and σ_θ . The deviator stress q and the horizontal shear stress $\tau_{\theta z}$ increases during the rotation of the bottom plate. The black crosses represent the critical deviator stress calculated with the Mohr-Coulomb model. In the Mohr-Coulomb model, the critical deviator stress q_{crit} depends not solely on the mean effective pressure p' but also on the Lode angle θ (see the graphs of the deviatoric plane in Figure 8c and d). The value

$\theta = 0^\circ$ for the Lode angle is defined as the triaxial compression (the two smallest principal stresses are equal). For the tests with an isotropic initial stress state ($p' = 100$ kPa) the Lode angle θ is constant during the shearing phase with $\theta = 30^\circ$. For the tests with smaller lateral stresses ($p' = 67$ kPa), the initial state is triaxial compression with $\theta = 0^\circ$ and during the shear phase the Lode angle θ increases. For the tests with larger lateral stresses ($p' = 133$ kPa) the Lode angle increases from an initial value of $\theta = 60^\circ$. From the comparison of Figure 8a and b it can be noticed, that for the contact plate with a smaller roughness (K11, left hand side) the maximum deviator stress q of the test with $p' = 67$ kPa (V26) reaches the calculated critical deviator stress q_{crit} , while the other two ones have a lower maximum deviator stress q , than the calculated q_{crit} . For the contact plate with a larger roughness (K0, right hand side) the deviator stress reaches the calculated q_{crit} for the tests with V32 ($p' = 67$ kPa) and V02 ($p' = 100$ kPa) while V31 with $p' = 133$ kPa has a lower maximum deviator stress q , than the calculated q_{crit} .

The stress ratio $M = \frac{q}{p'}$ represents the ratio τ / σ_N of the stress vector on the plane, where the τ / σ_N -ratio is the largest one. In a pure soil sample this would be the ratio leading to a failure if it exceeds the strength of the sample and the failure would happen in the corresponding plane. For the shear stress on the horizontal plane $\tau_{\theta z}$ displayed in Figure 8a and b the following observation can be made: The shear stress $\tau_{\theta z}$ for the tests with $p' = 67$ kPa – where q reaches q_{crit} – is markable lower than in the other tests. In all tests the normal stress on the contact plane is similar ($\sigma_z = 100$ kPa), hence the absolute shear stress $\tau_{\theta z}$ represents also the $\tau_{\theta z} / \sigma_z$ -ratio in the horizontal plane. This ratio is decisive for a failure in the (horizontal) contact plane. Such a failure would occur, if the contact shear strength in an interface would be exceeded or the failure would be forced to the contact plane (e.g. in direct shear tests).

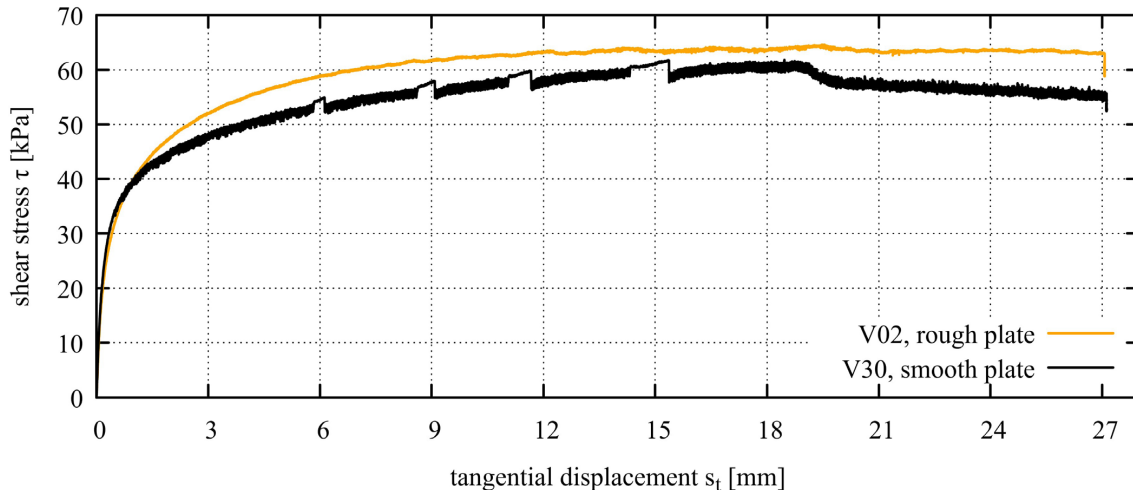


Figure 7. Comparison between a smooth (K11) and a rough (K0) top plate. Tests with isotropic initial stress states $\sigma'_z = \sigma'_r = \sigma'_\theta = p' = 100$ kPa.

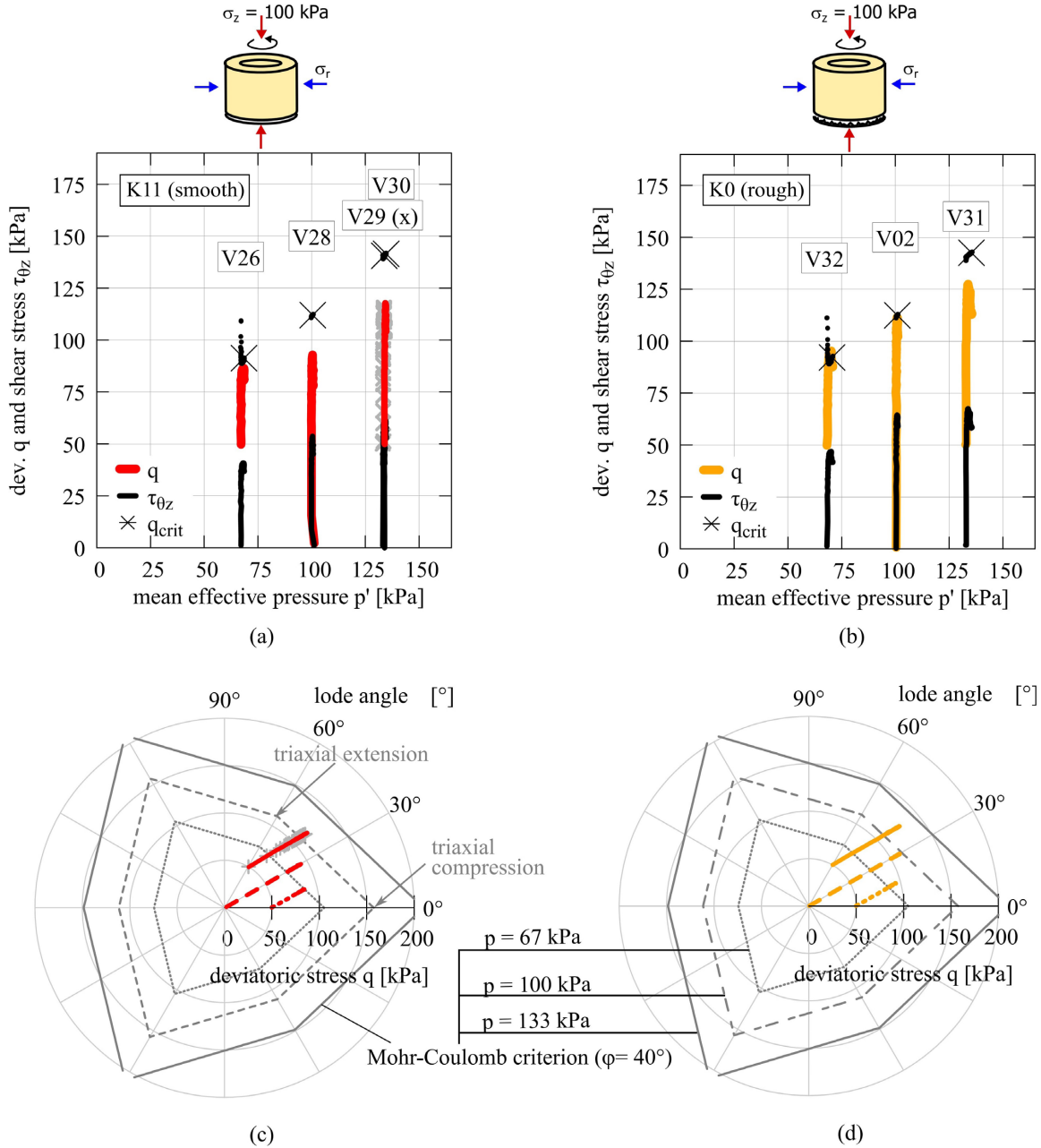


Figure 8. Tests in mode A (long sample). (a) and (c) test series A01:K11 (smooth plate), (b) and (d) test series A02:K0 (rough plate).

As a hypothesis, the failure mechanism in the interface tests in the modified hollow cylinder triaxial device is variable, so the failure will occur either in the soil or at the interface. In the soil, the failure will occur on the plane with the maximum ratio τ / σ_N (if it exceeds the soil strength). In the horizontal contact plane, the τ / σ_N -ratio is less, but the shear strength (=contact shear strength) is also less. If the shear stress on the contact plane reaches the contact shear strength first, the failure will happen in the contact plane.

For a smoother contact plate, the failure is more likely to happen in the horizontal contact plane. This can be observed

using the results of test series A01:K11 and A02:K0: For test series A01:K11 with a smooth contact plate, the test V26 matches the calculated deviator stress from the Mohr-Coulomb criterion (failure in soil is decisive), while the other tests deviate from it (failure on interface plane is decisive). For the test series with the rough endplate (A02:K0), only the test V31 does not match the calculated deviator stress (failure on interface plane is decisive). For the test with $p' = 100$ kPa, q matches q_{crit} , but the shear stress $\tau_{\theta z}$ is not less than in the test with $p' = 133$ kPa. It is likely, that in this test V31 both failure criteria were reached nearly at the same time.

Tests V29 (gray crosses) and V30 (line type as V26 and V28) are repetition tests to check the repeatability of the test results. The test results indicate a high reproducibility.

In a next step, different modeling methods for interface shear behavior can be tested and validated with the experimental results. From the experimental findings, two methods are proposed: 1) Use a relationship between the stress components on the contact plane ($\tau - \sigma_N$) for the interface and a neighboring (maybe extra small) element with the soil model to simulate a failure in the soil (q_{crit}). 2) Use an interface model including a dual yield function to model the failure in the contact plane and in an arbitrary plane in the soil at ones.

Both methods might capture the strength by the maximum horizontal shear stress or deviator stress. Another important aspect is dilation, which also affects the evolution of strength. The dilation in soil-structure interaction is influenced by the roughness of the contact surface and therefore might not be solely captured with a neighboring soil element.

3.3 Sliding and shearing at the contact

In the used test device, the torsional deformation of the sample is measured by the displacement of the top plate against the bottom plate. With this measurement, only the global deformation of the sample can be determined. In element tests, the assumption is taken, that the strain and stress in each position (small element of the sample) is equal and therefore, the stress and strain can be calculated from global measured values. Interface tests in general are no element tests because a concentrated deformation can occur on the interface and the strain near to the contact surface may be

greater than far away from it. Hence, the total displacement of the end caps is composed of the shear deformation in the soil (or a part of it) and the sliding between the contact plate and the soil (Niebler et al., 2025).

Figure 7 displays shear stress during tests V02 and V30 with contact plates with different roughness properties. It is noticeable that the scatter of the measured values has a different span for the smooth plate and the rough plate. Also, the span of the scatter is changing during the tests with the smooth end plate. A possible explanation is the slipping and shearing on the interface. For the rough end plate, no sliding occurs - there is a continuous frictional connection with force transmission between the contact plate and the soil. Therefore, the scatter in the shear stress is lower. For the smooth plate, there are alternating sections with higher and lower scatters. This might connect to an alternation between a sliding and a sticking regime at the interface. During phases with much scatter, there is a permanent change in the transferred force with sliding and sticking at the contact plate. The axial displacement and volume change measurements of V02 and V30 displayed in Figure 9 support this hypothesis.

During the phases of V30 with less scatter (connected to a sticking regime), the globally measured volumetric change and axial displacement show a steeper slope (more deformation is happening). This happens at a tangential displacement of approximately 6 mm, 9 mm, 11 mm, and 15 mm. A possible explanation is: During the sticking regime, more grains are mobilized and sheared and produce therefore a larger sum of dilative or contractive deformation which leads to a larger measured axial displacement or volume change.

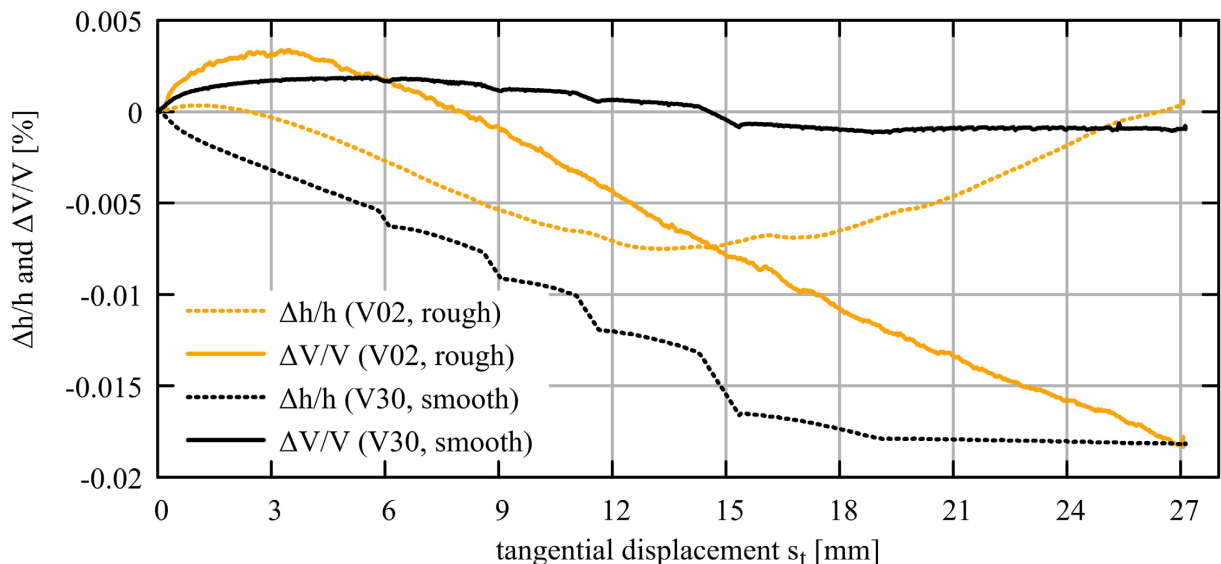


Figure 9. Normalized displacements of test V02 (rough contact plate, K0) and V30 (smooth contact plate, K11). The global measured and normalized axial displacement $\Delta h / h$ and volumetric change $\Delta V / V$ are displayed over the shear displacement (tangential displacement s_t).

3.4 Tests with mode B (short sample)

In this work, a second test mode (mode B) was used in the modified hollow cylinder triaxial device. The difference compared to test mode A with long samples ($h = 100\text{ mm}$) is the use of short samples with $h = 20\text{ mm}$. The test series B01:K3 with a less rough contact plate and test series B02:K0 with a rough contact plate are presented in Figure 10 and both conducted in mode B (short samples).

It is noticeable that the maximum deviator stress q reached in the tests does not match the calculated critical deviator stress q_{crit} with the Mohr-Coulomb model at the end of the test (marked with the black checkmarks in Figure 10). The deviator stress sometimes reaches not only smaller but even larger values than the calculated q_{crit} . This wouldn't be expected for soil element tests. The horizontal shear stress $\tau_{\theta z}$ (black lines) reaches similar values across the tests within each series. For tests V20, V15, V10 and V08, the

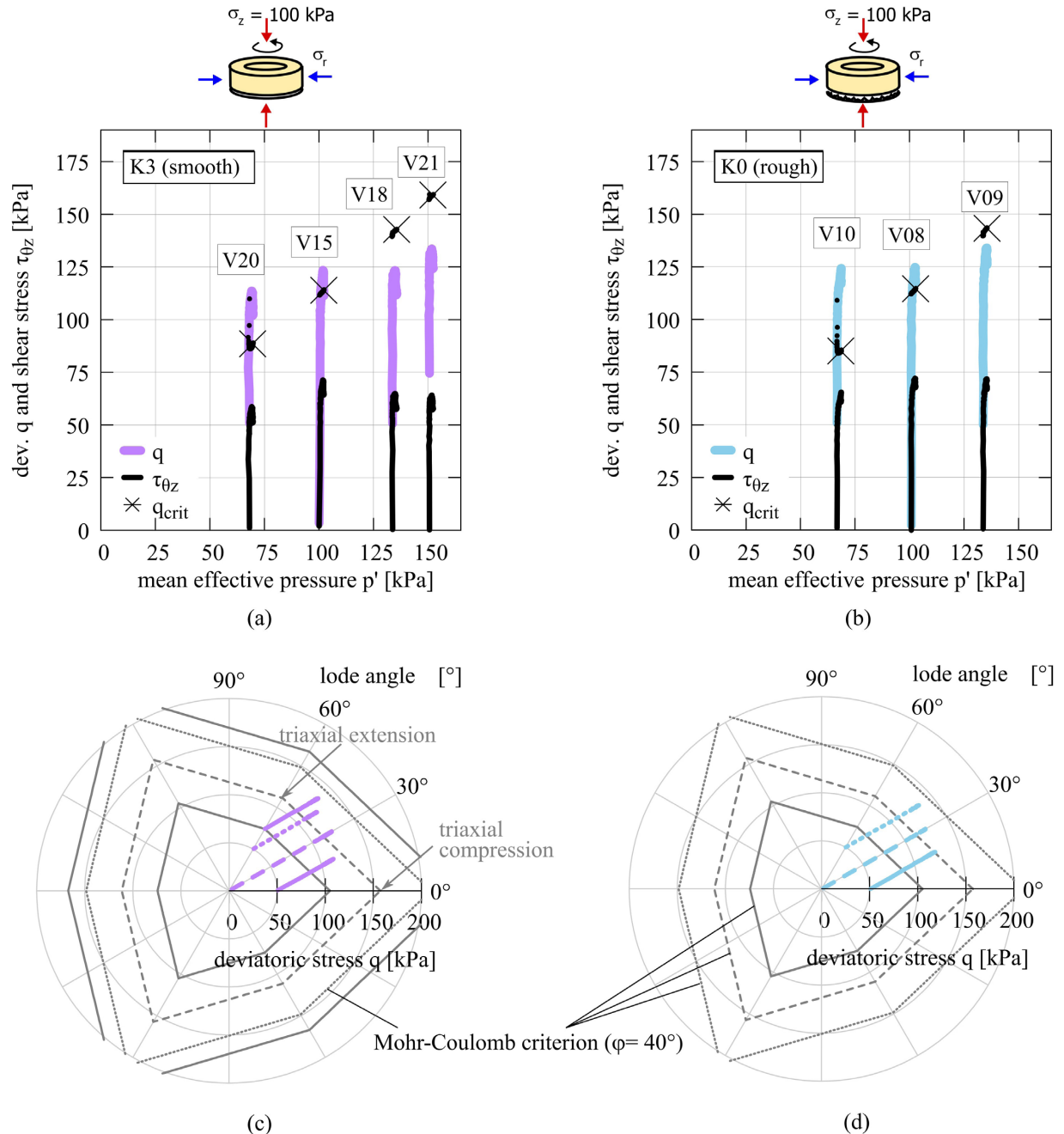


Figure 10. Tests in mode B (short sample). (a) and (c) test series B01:K3 with less rough contact plate K3, (b) and (d) test series B02:K0 with rough contact plate K0. Extended based on (Gehring et al., 2024).

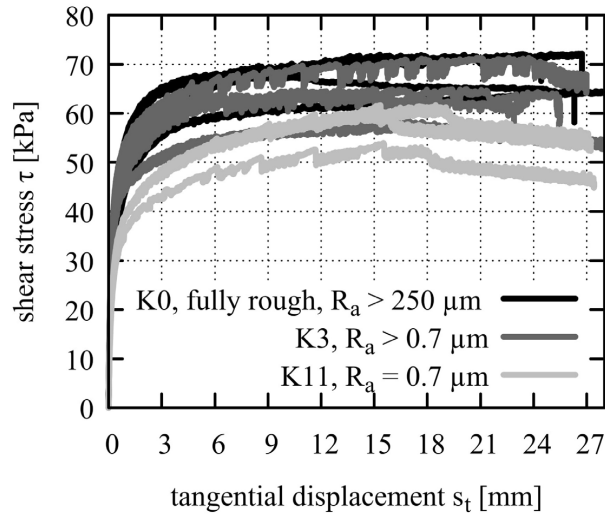


Figure 11. Shear stress for tests with horizontal failure plane and contact plates with different roughness properties.

deviator stress is larger than the calculated q_{crit} for failure in the soil. If these tests had been conducted in mode A, a failure would be expected in the soil on the plane with the maximum τ/σ -ratio and the deviator stress would be limited to the calculated q_{crit} (black crosses). This does not occur in these tests conducted in mode B. The different geometry appears to prevent this failure in the soil. This leads to the hypothesis that failure is restricted to the horizontal plane for tests in mode B. The maximum shear stress in the horizontal contact plane is decisive for the failure. It depends only on the normal force on the contact plane, which is equal in all tests in test series B01:K3 and B02:K0 and leads to similar values for the shear stress $\tau_{\theta z}$ across the tests.

3.5 Influence of roughness properties of the contact plate

Previous studies have shown that the maximum shear stress increases with an increasing surface roughness (see section 1). For tests in mode A with a long sample, the surface roughness is one of the factors governing the failure mode (see section 3.2). In the present section, tests with failure occurring in the horizontal plane on the contact surface are compared to investigate the influence of surface roughness on the maximum shear stress in the contact plane. Figure 11 displays the shear stress with increasing tangential displacement of the contact plate. The darker the line, the greater the surface roughness R_a of the used contact plate. The tests with a smoother contact plate tend to reach lower maximum shear stress values than those with a rougher contact plate. This is in accordance with the aforementioned findings from the literature. Still, the tests show significant deviations due to the different boundary conditions. Another point should be noted regarding the measured roughness values: The measured surface roughness in three spots is similar for the plate K3 and K11, but the surface

of K3 includes two larger filter stones and six screws (12% of the contact area) whereas K11 only contains two smaller filter stones (2% of the contact area). The additional screws contribute to the roughness of the surface and therefore, K3 behaves like a plate with a rougher surface than K11. This is reflected in the test results, as K3 shows higher maximum shear stress values than K11.

4. Conclusions

In this work, a novel test method for interface shear tests is presented. For this method and the first test results, the following conclusions can be made.

- With the new test method, four independent stress (or strain) components can be controlled in interface tests. Thereby, the influence of normal stress on the planes orthogonal to the interface can be investigated.
- A correction function for membrane stiffness was developed to correct the measured torque taking the deformation mode in the presented tests into account.
- For samples tested in mode A (long sample), as a hypothesis, the failure occurs either in the soil or on the contact plane. If it fails in the soil, the failure plane is the plane with the stress vector with the maximum ratio τ/σ_N of shear stress and normal stress. On the contact plane, the shear-stress-normal-stress ration is less, but also the strength is less on this plane: only the interface friction angle acts there.
- For samples tested in mode B (short sample), the failure in the soil is prevented by the geometry and therefore the failure plane is restricted to the horizontal contact plane.

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Declaration of interest

The authors have no conflicts of interest to declare. All co-authors have observed and affirmed the contents of the paper and there is no financial interest to report.

Authors' contributions

Sabine Gehring: conceptualization, data curation, formal analysis, investigation, visualization, writing – original draft. Hans Henning Stutz: conceptualization, resources, supervision, writing – review and editing.

Data availability

The data sets generated are available from the corresponding author upon reasonable request.

Declaration of use of generative artificial intelligence

This work was prepared with the assistance of generative artificial intelligence (Writefull and QuillBot) with the aim to enhance the linguistic quality of the text. The entire process of using this tool was supervised, reviewed and when necessary edited by the authors. The authors assume full responsibility for the content of the publication that involved the aid of GenAI.

List of symbols and abbreviations

b	coefficient for the ratio of the principal stresses
d_i	inner diameter of hollow cylinder sample
d_o	outer diameter of hollow cylinder sample
e_0	void ratio after sample preparation
$e_{\min}$	minimum void ratio, estimated in laboratory tests following DIN 18126:2022-10
$e_{\max}$	maximum void ratio, estimated in laboratory tests following DIN 18126:2022-10
h	sample height
p'	mean effective stress
p_i	inner cell pressure
p_o	outer cell pressure
q	deviator stress
q_{crit}	critical deviator stress (calculated with Mohr-Coulomb model from mean effective stress p and Lode angle θ)
r_i	inner radius of sample
r_o	outer radius of sample
s_t	tangential displacement, displacement of a point on the medium circumference of the bottom plate of the sample along the medium circumference, represents shear displacement
t_{butyl}	thickness of butyl membranes in test V17
t_{latex}	thickness of latex membranes in test V15
t_M	thickness of membranes
u_{so}	displacement due to shearing of soil (sticking of contact plate)
u_{str}	displacement due to sliding between soil and contact surface
CLA	center line average
E_{butyl}	stiffness of butyl membranes in test V17
E_{latex}	stiffness of latex membranes in test V15
E_M	stiffness of membranes
F_{ax}	axial force in the test device
I_d	relative density
M_{crit}	critical stress ratio
M_t	measured moment due to torsion in the test device

R_a	average roughness, average deviation from mean line of surface
ε_{vol}	global measured volumetric strain of the sample
θ	rotation angle of bottom plate
θ	lode angle (roscoe invariant)
ρ_s	grain density
$\sigma_1, \sigma_2, \sigma_3$	principal stresses
σ_{ij}	stress tensor
σ_N	normal stress component of stress vector on specific plane in sample
$\sigma_{\text{OP},1}, \sigma_{\text{OP},2}$	normal stress component of stress tensor in sample on orthogonal planes
σ_{pp}	normal stress component of stress tensor in sample on parallel plane
σ_r	radial stress
σ_z	vertical stress
σ_{θ}	tangential stress
τ	shear stress component of stress vector on specific plane in sample
τ_M	additional global measured shear stress due to the membrane stiffness
τ_M^{butyl}	additional global measured shear stress due to the stiffness of the butyl membrane in test V17
τ_M^{latex}	additional global measured shear stress due to the stiffness of the latex membrane in test V15
$\tau_{\text{tot}}^{\text{V15}}$	total global measured shear stress in test V15
$\tau_{\text{tot}}^{\text{V17}}$	total global measured shear stress in test V17
$\tau_{\theta z}$	shear stress in the horizontal plane of the sample
φ	friction angle
φ_r	contact friction angle
$\Delta \tau$	difference of global measured shear stress in V17 and V15

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Evaluation of slope failure through the assessment of meteorological conditions and earthworks management

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Article

Keywords

Rainfall triggered landslide
Infrastructure damage
Site restoration
Numerical analyses

Abstract

Slope failures due to erosion effect are a well-established cause of civil engineering structure disasters. Streams passing at the toe of a slope are a severe threat to geotechnical safety, especially in urbanized areas. To increase global safety, a retaining wall is usually proposed to limit the risk of failure. However, such works could also bring the high danger of a landslide during the construction works if not managed properly. The paper aims at investigating the collapse cause of residential buildings located at the edge of the slope at northwest part of Vietnam, where engineering works were performed to prevent failure due to soil erosion at the toe of the slope. During the retaining wall construction works, the applied excavation method caused significant vibrations and additional dynamic loads that could have affected the bearing capacity of the building foundations. Another cause of failure is associated with heavy precipitation recorded at the site just before the excavation works down the slope. The paper consists of numerical analyses allowing simulation of possible scenarios, including the effect of changing groundwater conditions using the meteorological data collected from the nearby monitoring weather station. The computations were performed using geotechnical software to reflect the in-situ conditions, and the entire process of earthworks performed at the site was carefully analyzed. The numerical analyses involved computations performed for a reinforced slope where a retaining wall was proposed to assure geotechnical safety of the remaining infrastructure after the failure.

1. Introduction

Slope stability is a critical safety concern in urban development planning, particularly in areas where construction activities alter the natural landscape. Landslides can occur due to both natural geological and climatic processes, as well as human-induced factors, such as poor site management and engineering practices. While natural processes—including extreme weather events, increased precipitation, flooding, earthquakes, and volcanic eruptions—are inherently difficult to control, implementing proper mitigation measures, such as groundwater and surface runoff management, can significantly reduce the risk of slope failure. In contrast, landslides triggered by human activities can often be prevented through comprehensive geotechnical assessments, well-planned construction practices, and effective site management (Kim et al., 2014).

Key human-related causes include inadequate drainage systems, improper excavation techniques, unsuitable earthworks, and poorly executed construction works. Understanding these risks and implementing appropriate mitigation strategies are essential for ensuring the long-term stability of slopes and preventing damage to infrastructure.

The present study investigates the factors that contributed to the collapse of buildings located near the edge of a natural slope. The failure is primarily linked to extreme precipitation events recorded at the site, combined with the effects of retaining wall construction. The retaining structure was intended to enhance the geotechnical stability of the slope while preventing toe erosion caused by a nearby stream, which had previously inflicted seasonal damage on the supporting bank. However, despite its intended function, the wall's construction may have contributed to slope instability, ultimately leading to structural failure.

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Retaining walls are essential geotechnical structures designed to resist lateral earth pressures and control excessive slope deformation. Their effectiveness largely depends on the construction material and method used. Different types of retaining walls include gravity walls, masonry walls, gabion walls, diaphragm walls, and sheet pile walls (Lanzieri & Avesani, 2025; Look, 2022; Salgado, 2022; Raposo et al., 2017). More recently, Mechanically Stabilized Earth (MSE) walls, which utilize geotextile reinforcement to separate and strengthen soil layers (Kawalec et al., 2019), have become increasingly popular in engineering practice. In addition to their role in stabilizing slopes, retaining walls also serve as protective barriers against soil erosion along riverbanks and waterways, where undercutting at the slope's base can pose a serious threat to structural integrity. When implementing slope stability measures, particularly in areas with existing infrastructure, careful planning is essential to minimize the risks associated with reinforcement works. One of the most critical factors influencing slope stability is groundwater management. Continuous monitoring of groundwater levels is crucial, as fluctuations in water content can significantly impact soil behavior and slope stability (Wang et al., 2014). Properly designed and well-maintained drainage systems play a vital role in improving working conditions at sites undergoing reinforcement earthworks.

The method of reinforcement installation is especially crucial in high-risk areas, where slope failure could endanger surrounding infrastructure. Previous research has extensively examined rainfall-induced landslides (Benz & Blum, 2019; Lacroix et al., 2020; Lira et al., 2020; Maturidi et al., 2021) and failures caused by inadequate execution of earthworks during reinforcement installation. Engineering studies indicate that one of the most common triggers of slope instability is the increase in groundwater pressure, which reduces effective stresses along potential slip surfaces (Augusto Filho & Fernandes, 2019). This effect is particularly pronounced in developed areas, where changing groundwater conditions can alter soil properties, leading to failures that compromise both the ultimate and serviceability limit states of structures. Moreover, bearing in mind climate change predictions presented in Figure 1 after the Vietnamese governmental report (Thuc et al., 2016), estimating 80% increased torrential rainfalls by the end of 21st century the particular measures should be taken accordingly.

Rainfall plays a significant role in landslide initiation by increasing porewater pressure and reducing the shear strength of soil materials (Orense, 2004). As rainwater infiltrates the soil, it saturates upper layers, potentially triggering shallow slope failures. Prolonged precipitation, however, affects deeper layers of the slope profile, leading to much more severe collapses (Alsubal et al., 2019; Han et al., 2014; Patuti et al., 2017). One of the key factors influencing soil behavior under varying moisture conditions is soil suction, which develops in the unsaturated zone above the groundwater table. Changes in soil suction can have a substantial impact

on the mechanical and hydraulic response of the soil, further exacerbating slope instability.

This paper aims to thoroughly investigate the root causes of slope failure in a site located in northwest Vietnam. A combination of natural and human-induced factors contributed to the failure, leading to extensive structural damage and the forced evacuation of local residents. To accurately reconstruct the sequence of events leading to the collapse, the study relies on comprehensive site investigation data, meteorological records, and documentation of earthworks executed during the retaining wall construction. By analyzing these factors, the authors seek to provide a detailed assessment of the mechanisms that contributed to the slope failure. One of the key objectives of this study is to propose effective restoration solutions for the site. The research focuses on re-engineering the failed slope by designing appropriate reinforcement strategies that ensure long-term geotechnical stability while minimizing risks to existing infrastructure. By considering past failures and evaluating alternative stabilization techniques, the study aims to offer insights that can be applied to similar high-risk construction projects in the future. Slope stability is a fundamental concern in urban development, particularly in areas where construction activities interact with complex geological and hydrological

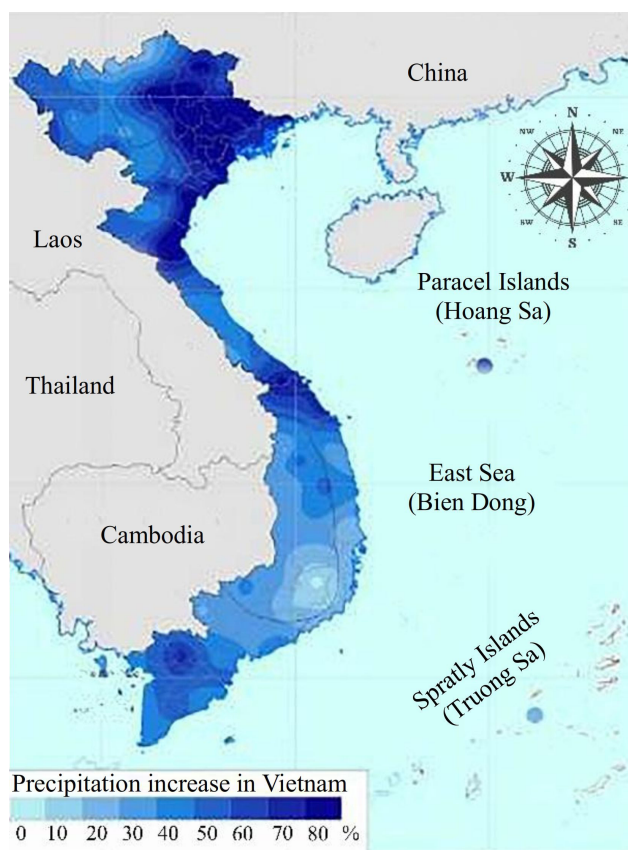


Figure 1. Average change in torrential precipitation (%) predictions for end of 21st century for Vietnam (modified after Thuc et al., 2016).

conditions (Osiński et al., 2020). While natural factors such as extreme precipitation can contribute to landslides, human activities—especially improper drainage and reinforcement techniques—can exacerbate instability and lead to structural failures. The findings of this study highlight the importance of integrating geotechnical assessments, effective drainage solutions, and appropriate construction methods to mitigate slope failure risks. By understanding the interplay between natural and anthropogenic factors, engineering practices can be improved to ensure safer and more sustainable urban development in landslide-prone areas.

2. Study site description

2.1 Geomorphology and climate conditions

The case study site is located in a mountainous district located in the center of Dien Bien province, in north-west part of Vietnam. The entire area is surrounded by high mountains reaching 450 m above the ground level. The district is divided into two landform zones where the terrain elevation ranges from 350–1000 m AMSL, and the zones where slopes reach up to 1500 m AMSL, with the slopes facing northwest-southeast. The study site location is presented on Figure 2.

The terrain is morphologically complex with a number of valleys and rivers crossing the entire region, where during the wet season, the meandering streams cut into the mountain's toes. What is important for the case study is the

presentation of the local climate condition that has a major effect on the working conditions of the infrastructure and influences natural hazards like flooding and landslides. The climate conditions are typical of a tropical monsoon climate. The winter season is considered rather cold, with the lowest temperature at 2 °C, and the rainfall volume does not exceed 60 mm per month. The summer season is considered hot and rainy, with the highest temperatures reaching 40 °C and the monthly highest volume of rainfall up to 500 mm recorded in July. The average temperature during the year is between 18 °C and 25 °C. The average annual rainfall ranges from 1,600 to 2,400 mm. The rainy, hot and humid season starts in April and lasts for 6 months. The dry season starts from October to April, with low temperature and low rainfall volume (Luu et al., 2022). The average air humidity is from 83 to 85% with the lowest values recorded for springtime (March to May). According to data from the Vietnam Institute of Meteorology, Hydrology and Climate Change, the average annual rainfall, in the period from 1961 to 2018, was 1562 mm. In that same period, the average annual temperature was 22.17 °C, with an increase by 0.75 °C between 1961–1990 and 1991–2018. Dien Bien is expected to continue experiencing temperature increases (Tran-Anh et al., 2022). According to unpublished baseline data of the Agricultural Cooperative Strengthening (ACS) project run by humanitarian organization CARE in Vietnam, several adverse weather and climate events can be damaging to infrastructure development. These adverse conditions include floods, flash floods, landslides, early onset of the rainy season and extended rainy seasons.



Figure 2. Location of study area.

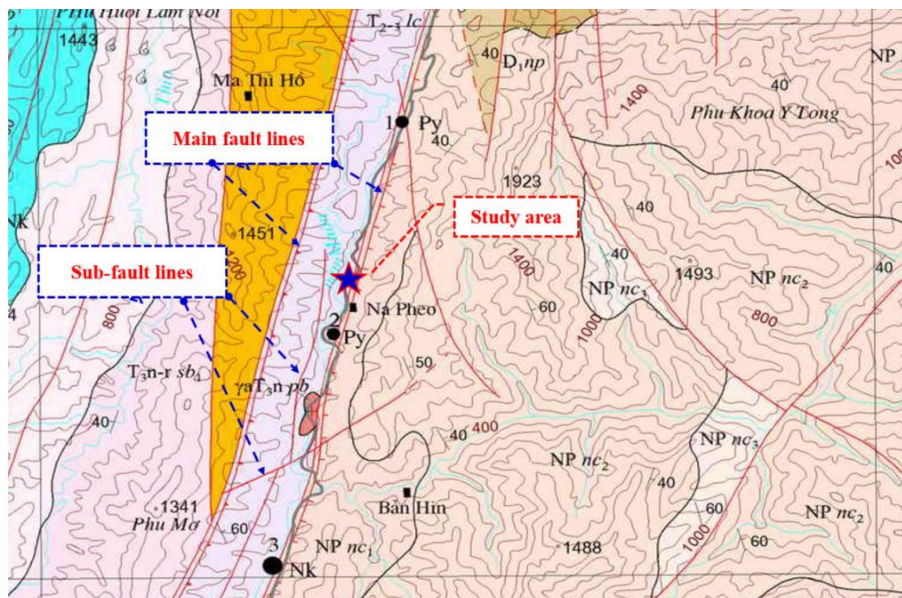


Figure 3. Geological map of the study area.

It is also important to highlight the fact that the area is seismically active. The most recent earthquake that took place at the site was recorded in 2018, of 4.3 magnitude (M). According to the geological map of Vietnam with a scale of 1:200000 (Figure 3) (DGMV, 2005), the study area is located on the main fault of the Dien Bien-Lai Chau fault zone.

The Điện Biên–Lai Châu fault zone is a sinistral strike-slip conjugate fault system of significant geological importance to the region. It consists of a main fault running through the center, accompanied by several subsidiary faults on either side. The main fault plane dips westward at an angle of 60° – 70° , and is nearly vertical in the southern segment. This tectonic system results in highly jointed rock formations within the study area, which may expose the proposed structure to seismic risk. The fault cuts across formations dating from the Proterozoic, Paleozoic, and Mesozoic eras. In the territory of Vietnam, the fault zone is about 160 kilometers long (Nguyen, 2001). This is one of the most active fault zones in Vietnam where seismic activities have been recorded (Phuong & Truyen, 2015). A number of seismic events took place there along the main fault and the adjacent areas i.e., Dien Bien earthquake of M 6.75 in 1935, Na Pheo earthquake in 1980 of M 4.6; Lai Chau earthquake of M 4.2 in 2001 and Dien Bien earthquake of M 5.3 in 2001. Phuong and Truyen (2015) rated that the region of Dien Bien-Lai Chau could have experienced the maximum values of Peak Ground Acceleration (PGA) at 180 - 272 gals for 475 - 9975 years. According to the geological map (DGMV, 2005), the study area is located in Lai Chau formation (T2-3lc) which is characterized by fine-grained terrigenous sediments mainly of deep-water facies, distributed in a narrow band along the Lai Chau – Dien Bien fault. The site investigation revealed that the study area comprised light grey, fine-grained, thin to medium bedded (0.1 to 0.5m) sandstone



Figure 4. Bedrock at the stream bank just beneath the reported failure.

interbedded with dark-grey, fine-grained thin bedded siltstone and argillaceous. According to the geotechnical investigation report, the bedrock is mainly composed of shale interbedded with sandstone which is in accordance with the descriptions in the geological map. The site survey (Figure 4) showed that the shales are strongly folded, and the surface condition was fractured due to a highly weathered surface with cracks and angular deposits.

The rock formations at the study site have two predominant joint sets, namely the bedding joints and near vertical set. Joints are mostly weathered and smooth with a spacing range from a few millimeters to 10 centimeters. Cracks less than 1 mm are usually filled with clay. The shale contains fissures due to shearing. Remarkably, the shale represents softening behavior due to water infiltration. Sandstone is whitish grey, fine grained and thinly bedded. The sandstone presents high uniaxial compressive strength. The bed rocks are randomly intersected by thin deposits of quartz. The presence of thin bedding joints and the tectonically induced discontinuities cause the rock mass to be highly fractured. The sandstone beddings are persistent, and the joint condition is rated to be fair with a moderately weathered and altered surface. Accordingly, the geological strength indices are estimated using Marinós (2019) classification as 15 and 25 for slightly weathered shale and slightly weathered sandstone, respectively.

2.2 Geotechnical site investigation and additional key data

For the purpose of detailed analyses of the conditions at the study site, the geotechnical investigation was performed to establish the geological and geotechnical soil profiles and the parameters necessary for further computations of geotechnical safety. The analyses of the geotechnical structures were based on the site investigation performed for two cross-sections involving 5 geological boreholes. The maximum depth of drilling was 30 m. The total depth for all the drillings was 100 m. The location of geological boreholes is presented in Figure 5. During the site investigation, 11 core samples

were collected and transported to the laboratory, where physical and mechanical parameters were tested. During the field works Standard Penetration Tests (SPT) were also performed to determine the shear strength parameters. The investigation allowed distinguishing 4 major geotechnical layers. Starting from the ground level, these are:

- Layer 1 (Backfill): clay and gravel, inconsistent state from hard to soft of low SPT index. The thickness of the backfill layer varies from 1.8–4.7 m.
- Layer 2 (Strongly weathered shale): shale weathered into clay and gravel, thinly layered hard plastic state, of low SPT index. The thickness of the soil layer varies from 1.3 m to 5.8 m.
- Layer 3 (Lightly weathered shale): thin, fragile rock material. Thickness varies from 4.0 m to 9.2 m.
- Layer 4 (Sandstone): fine-grained sandstone, of high stiffness with interbedding of quartz particles. The thickness varies from 3.0 m to 3.4 m.

The experimental results of physicommechanical parameters from soil samples collected during site investigation are presented in Table 1.

To provide a better understanding of the geotechnical structure the slope profile is presented in Figure 6.

Since the rainfall events in the study were concerned with having a major effect on the failure, the precipitation data were also analyzed. To allow further understanding of the failure cause, the rainfall data was collected from the nearby monitoring station (Na Sang station, 10 km from the study area) and is presented in Figure 7. Important for the study is the accumulative rainfall recorded for October 2021, when 2000 mm of accumulative precipitation was reached.

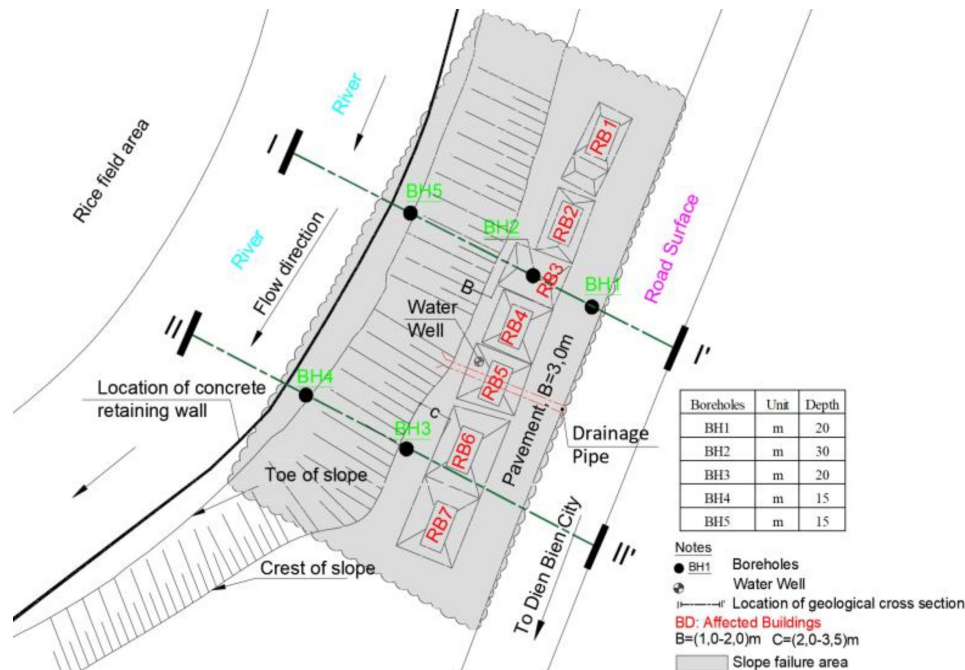


Figure 5. Site investigation cross-sections and boreholes' locations.

Based on gathered data regarding geological, and geotechnical investigation and rainfall volume, the computations of geotechnical safety were performed to determine the failure mechanism and to reflect the sequence of the events.

Table 1. Physical and mechanical parameters of the investigated soil layers.

Parameter	Soil type			
	Fill	Shale*	Shale**	Sandstone**
w (%)	23.9	28.2	-	-
γ (kN/cm ³)	19.6	19.0	22.0	25.0
e	0.69	0.8	-	-
I_p (%)	16.2	15.7	-	-
c' (MPa)	0.0167	0.174	0.09	0.18
φ' (°)	17	20	32	46

*highly weathered. **moderately weathered.

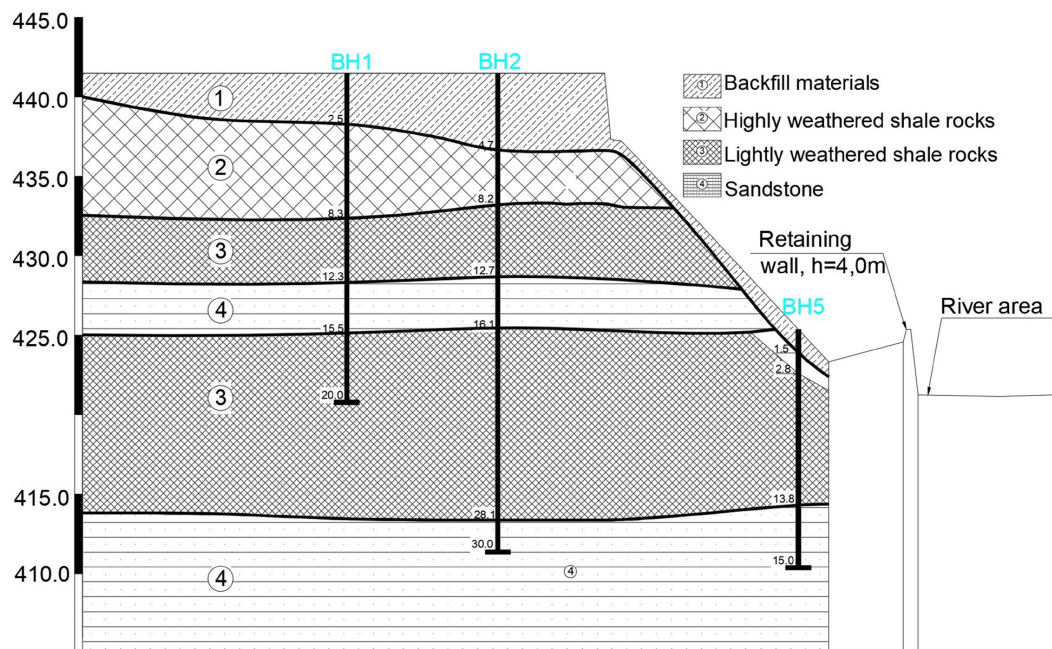


Figure 6. Geological cross-section I-I'.

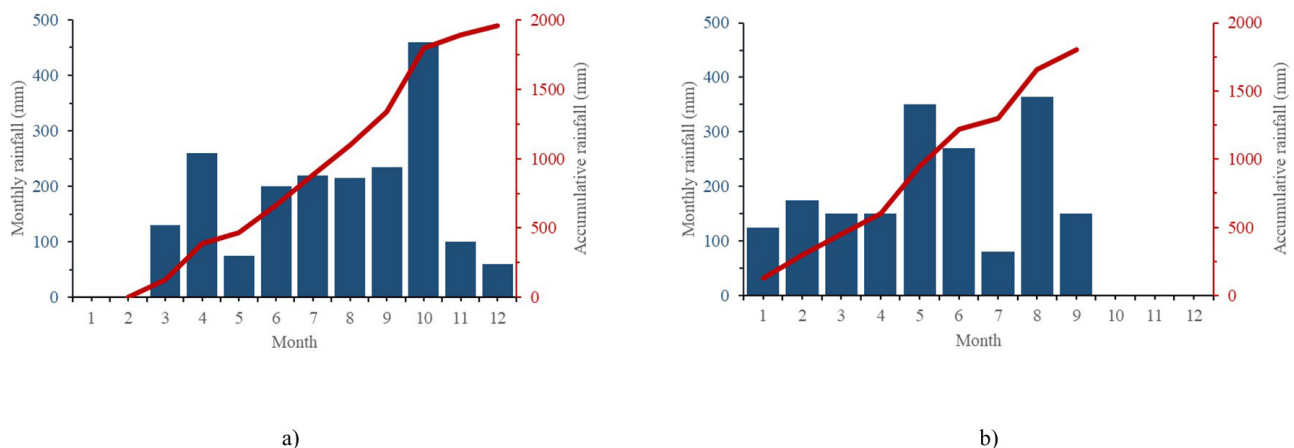


Figure 7. Rainfall data record for particular months a) in 2021 and b) in 2022 in Dien Bien district.

3. Results and discussion

3.1 Analyses of the events sequence

The case study concerns a natural slope with extensive infrastructure developed at the top edge. The infrastructure development started with the construction of a local road near the edge and the design of the drainage system, redirecting the runoff down the 25 m high slope. Following the road construction number of storey buildings were constructed in the section between the road and the slope's edge. At the toe of the slope, there is a natural stream passing with the flow changing depending on the rainfall volume. In recent years, the local community's major concern has been focused on the erosion effect due to the stream continuously cutting the toe of the slope. Thus, a retaining wall was constructed to control the water erosion and improve the slope's stability. After the works were accomplished, significant rainfall events took place. The finger pipe drainage was installed at the back of the wall. A few weeks later, the residents reported the first building cracking, which continued to develop until the next severe rainfall event that was followed by the slope failure and a collapse of buildings located at the study site. The damages to the infrastructure are presented in Figure 8.

The authors of the present paper aimed to investigate events and the processes contributing to the failure occurrence. Based on the data collected from the site investigation report, meteorological report and the information gathered during the site visits, the analytical study was performed to reflect the in situ conditions. The authors started their analysis by reconstructing the sequence of events leading to the slope failure. This involved a thorough examination of all relevant site conditions, including correlating recorded rainfall data with the timeline of the retaining wall construction, observing the development of cracks in residential buildings, and identifying

the moment of the final slope failure. To provide a clearer understanding of the interplay between these factors, Figure 9 presents a chronological sequence of events, illustrating the possible causes that ultimately led to the collapse and the severe damage sustained by nearby structures.

In late 2021, following growing concerns about the deteriorating condition of the slope, local authorities decided to implement preventive stabilization measures. The primary intervention involved the construction of a concrete gravity retaining wall at the toe of the slope, intended to mitigate potential failure and reinforce the stability of the area. The excavation phase, which took place between October 7 and October 20, 2021, was reported to have caused significant ground vibrations. These vibrations were particularly noticeable to local residents living in the buildings situated at the crest of the slope. Complaints were raised regarding the intensity of the shaking, which raised concerns about its potential impact on the structural integrity of the buildings.

Despite these reports, the construction proceeded as planned. Shortly after the completion of the retaining wall, an extreme rainfall event occurred on October 22, 2021, with recorded precipitation levels reaching 30 mm in a single day. It is important to emphasize that the total accumulated rainfall for October 2021 was approximately 2000 mm, highlighting the exceptionally wet conditions in the region during that period. One week later, the first signs of structural distress appeared, as cracks were observed in the walls of residential buildings near the slope edge. Over the next two months, additional damage to the buildings became evident. The situation escalated dramatically on January 16, 2022, when a heavy rainfall event of nearly 45 mm in a single day occurred. Following this event, the slope experienced a catastrophic failure, resulting in the partial collapse of buildings and severe infrastructural damage. During the earthworks phase, the dynamic loads generated by excavation activities and construction-induced vibrations disturbed and loosened the soil structure at the building foundations.



Figure 8. Extensive damage to the residential buildings located on the study slope.

This disturbance likely compromised the shear strength of the soil layers, making it more susceptible to external environmental factors. Following this phase, heavy rainfall decreased the stability by infiltrating the loosened soil profile. The increase in pore water pressure within the soil layers led to a reduction in shear strength, thereby lowering the overall stability of the slope. Additionally, the inadequate drainage system failed to efficiently redirect surface and groundwater flow, causing excessive water retention within the slope mass. This progressive saturation contributed to the build-up of pore water pressure in the soil, further weakening its mechanical properties and increasing the likelihood of slope failure. Figure 10 presents a chronological sequence of events, providing a comprehensive overview of the factors contributing to the slope failure. By illustrating the circumstances including construction activities, recorded rainfall events, and the progressive development of structural

damage the graphical interpretation helps to identify the potential interactions between these elements.

Understanding this sequence is crucial in determining the underlying causes of the slope's ultimate failure, which led to significant structural damage to residential buildings and posed serious safety risks to the affected community.

3.2 Restoration and slope stability improvements

Based on the geotechnical investigation data and recorded rainfall volume, computational analyses of geotechnical safety were conducted to determine the failure mechanism and reconstruct the sequence of events leading to the slope collapse. These analyses were crucial in understanding the contributing factors to instability and assessing the extent of structural damage. Following this assessment, restoration works were designed, and additional geotechnical safety

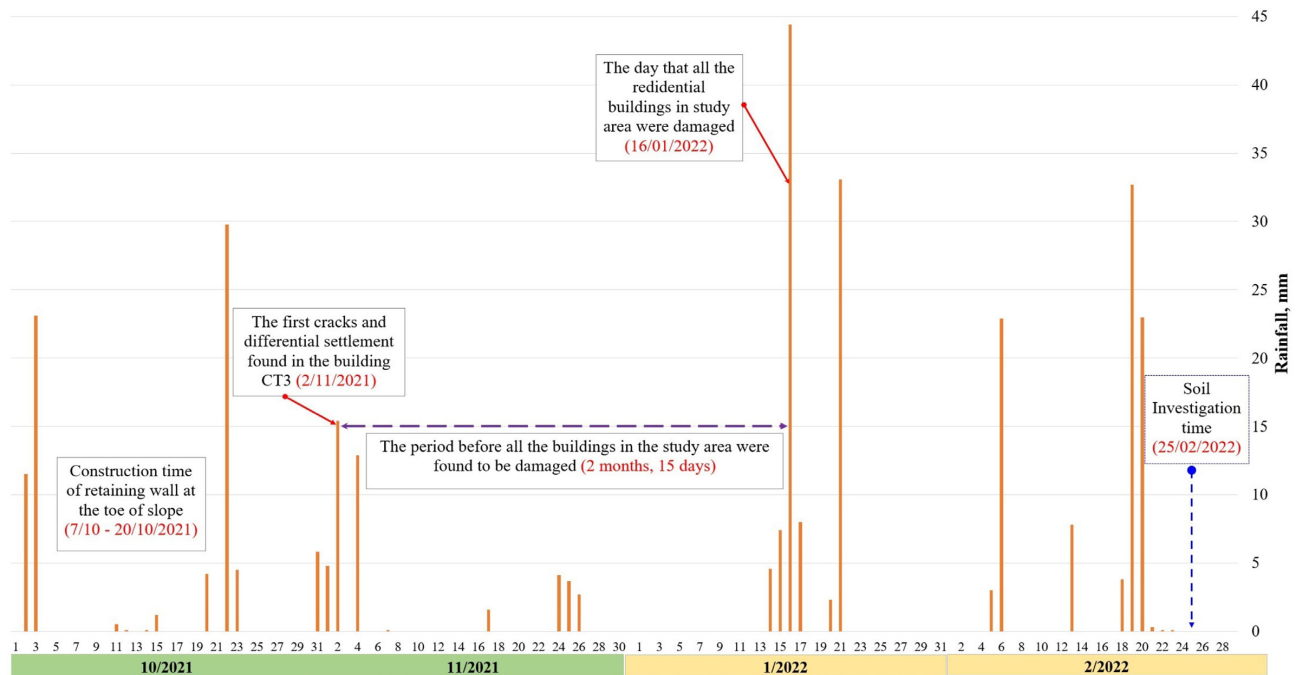


Figure 9. The sequence of events leading to a final failure with referencing rainfall data.

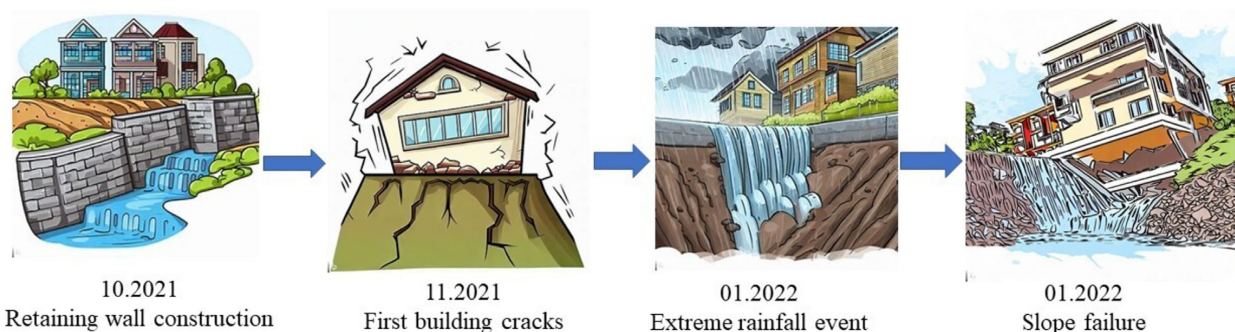


Figure 10. Graphical presentation of the sequence of events leading to a structural failure of residential buildings (AI generated).

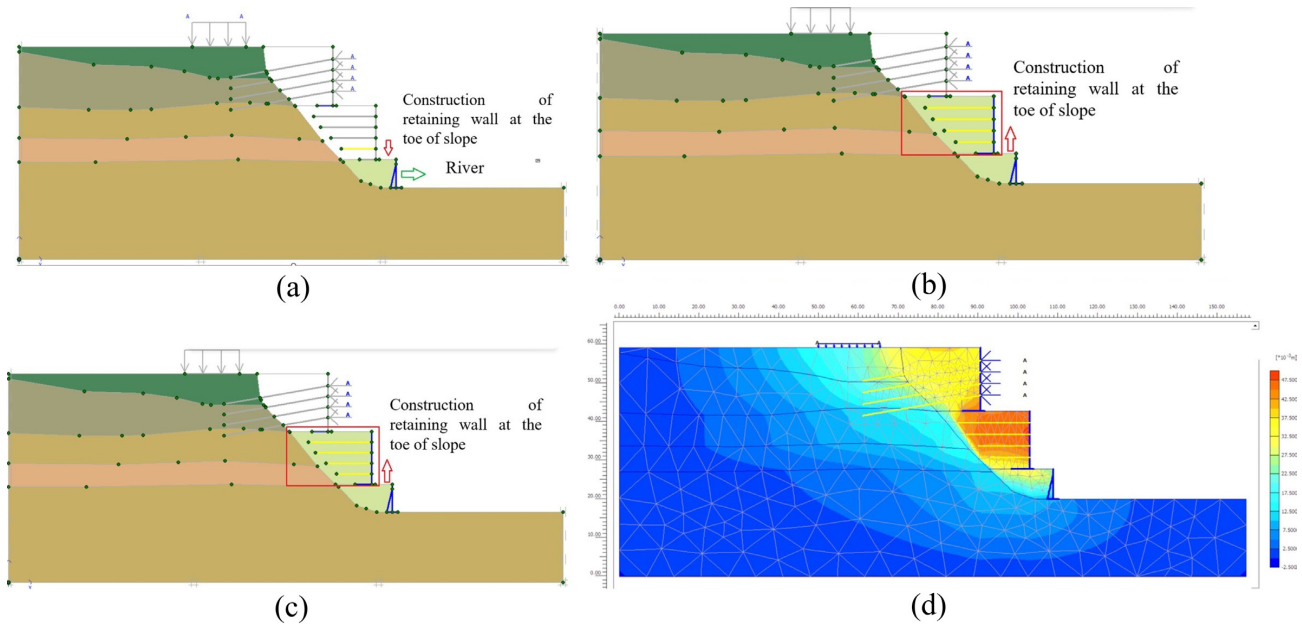


Figure 11. Sequence of slope stability improvement a) 1st stage-backfill with gabions, b) 2nd stage MSE, c) 3rd stage soil nails, d) displacements computations.

computations were performed to ensure the long-term stability of the slope and surrounding infrastructure. The slope stability analyses were carried out using the Finite Element Method (FEM), using commercial software Plaxis (Brinkgreve et al., 2016), incorporating geotechnical parameters obtained directly from site investigations. The Mohr-Coulomb soil model was applied to simulate soil behavior under different loading conditions, while the Factor of Safety (FoS) was computed using the Shear Strength Reduction (SSR) method to evaluate slope stability under various scenarios. To account for the hydrogeological influences on slope stability, multiple simulation scenarios were analyzed, considering the changing groundwater levels due to rainfall infiltration. These simulations provided insights into how excessive water infiltration reduced soil shear strength, increasing the likelihood of failure. To ensure long-term safety and prevent further instability, the entire slope was re-engineered using a combination of reinforcement techniques. The proposed stabilization system included: Mechanically Stabilized Earth (MSE) Wall – Implemented to provide additional lateral support and enhance slope resilience; Gabion Retaining Wall – Selected due to its flexibility, permeability, and ability to absorb dynamic loads, minimizing further deformation and reducing the impact of future precipitation events. This integrated reinforcement system was specifically designed to minimize excavation and earthwork disturbances to protect the remaining infrastructure and prevent further damage. The optimum stabilization solution was determined through numerical simulations, where deformation patterns and safety factors were analyzed. The results of these computations, presented in Figure 11, illustrate the effectiveness of the

proposed stabilization measures in improving the overall geotechnical safety of the site. The obtained factor of safety was $1.34 > \text{FoS} = 1.25$, and the maximum horizontal displacement did not exceed $4.25 \text{ cm} < \delta = H/200 = 7.5 \text{ cm}$, where H is the total height of the wall.

4. Conclusions

The primary objective of this study was to investigate the on-site conditions and reconstruct the sequence of events that may have led to structural damage in residential buildings and ultimately caused the slope failure. A comprehensive analysis was conducted using site investigation data, meteorological monitoring reports, drainage assessments, and an evaluation of earthworks carried out during the construction of the retaining wall. This multidisciplinary approach enabled a thorough understanding of the key contributing factors to the slope's failure. Based on the collected data, a reclamation and stabilization plan were developed to mitigate further risks and restore geotechnical safety. The factor of safety (FoS) and total deformations of both the failed and newly engineered slopes were computed to validate the effectiveness of the proposed solutions. Advanced numerical modeling techniques were employed to assess different stabilization scenarios, ensuring that the remedial measures would prevent future failures while minimizing additional risks to the surrounding infrastructure. A critical takeaway from this case study is that earthworks intended to enhance safety can, under certain circumstances, have the opposite effect—particularly when executed in regions prone to extreme weather events. In this instance, the construction works, combined with heavy rainfall and inadequate drainage management, contributed to soil instability

rather than preventing it. This underscores the importance of integrating climate-responsive geotechnical solutions, adopting real-time monitoring systems, and implementing proactive risk management strategies in similar high-risk environments. The findings suggest that the combination of intense construction activities and extreme meteorological conditions created a scenario where the slope became highly vulnerable to failure. While the retaining wall was intended to reinforce the area, the methods of excavation and drainage management likely played a significant role in destabilizing the slope rather than securing it. This case highlights the critical importance of incorporating advanced drainage solutions, real-time monitoring, and soil stability assessments in engineering interventions designed to mitigate landslides in high-risk areas. The findings from this study provide valuable insights into the interaction between construction-induced stress changes and hydrological effects on slope stability. Future research should focus on implementing early warning systems, improved geotechnical modeling, and resilient infrastructure designs to prevent similar failures in regions prone to heavy rainfall and complex topographical conditions.

Declaration of interest

The authors have no conflicts of interest to declare. All co-authors have observed and affirmed the contents of the paper and there is no financial interest to report.

Authors' contributions

Nguyen Quang Tuan, Duc Bui Van and Piotr Osinski: conceptualization, data curation, visualization, writing – original draft. Nguyen Van Manh, Ngoc Anh Do and Eugeniusz Koda: conceptualization, data curation, methodology, supervision, validation, writing – original draft. Piotr Osinski, Duc Bui Van and Eugeniusz Koda: formal analysis, funding acquisition, investigation, methodology, project administration, resources, software. Piotr Osinski, Ngoc Anh Do, Nguyen Van Manh: supervision, validation, writing – review & editing.

Data availability

All data produced or examined in the course of the current study are included in this article.

Declaration of use of generative artificial intelligence

This work was prepared with the assistance of generative artificial intelligence (GenAI) DALL-E with the aim of creating the graphics in Figure 10. The entire process of using this tool was supervised, reviewed and when necessary edited by the authors. The authors assume full responsibility for the content of the publication that involved the aid of GenAI.

List of symbols

c'	Effective cohesion
e	Void ratio
w	Moisture content
ACS	Agricultural Cooperative Strengthening
AMSL	Above Mean Sea Level
FEM	Finite Element Method
FoS	Factor of Safety
H	Total height of the wall
I_p	Plasticity index
M	Magnitude
MSE	Mechanically Stabilized Earth
PGA	Pick Ground Acceleration
SPT	Standard Penetration Test
SSR	Shear Strength Reduction
γ	Unit weight
δ	Horizontal displacements
ϕ'	Effective internal friction angle

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Design of the Vienna metro shaft: probabilistic analysis and settlement forecasting of the soil-structure system via Monte Carlo simulation

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Article

Keywords

Probability approach
Reliability
Monte Carlo simulation
Reliability assessment
Regression analysis

Abstract

Reliability and safety assessments require optimizing decision-making throughout the lifecycle of systems such as the “soil mass – underground structure” system. Changes in the engineering-geological environment cause various geohazards that must be identified and evaluated during the project development stage, taking into account both natural and anthropogenic factors related to subway construction and operation in dense urban environments. This article presents an analysis of shaft structures for the proposed Vienna metro line using numerical and probabilistic modeling, with particular emphasis on the quality of input data, their probabilistic variability, and their influence on the stress-strain state of the structures. Finite element modeling was performed using the PLAXIS software package, employing advanced elastoplastic Hardening Soil (HS) and Hardening Soil Small (HSS) ground models. By repeatedly varying initial soil parameters surrounding reinforced concrete tunnel structures, variation coefficients for key loads and deformations in both the structures and soil mass were obtained, which can inform the design of similar structures in the future. A methodology and algorithm for assessing the reliability of the “soil mass – underground structure” system were proposed, based on the Monte Carlo statistical method. The evaluation criterion is the probability of exceeding threshold values of additional vertical settlement after tunnel excavation in unfinished sections of the Vienna metro. This approach enables the prediction of additional deformations affecting the foundations of existing buildings impacted by tunneling. Finally, regression analysis was conducted to assess the influence of varying factors on stress-strain parameters derived from FEM calculations, using both linear and multivariate models.

1. Introduction

The metro is a key element of the transport infrastructure in megacities, playing a crucial role in ensuring effective mobility and the sustainability of the urban environment. In conditions of high population density and limited space for building traditional transport networks, the metro becomes an important mechanism for addressing the issues of road congestion, reducing traffic jams, and improving the environmental situation. The role of the metro in stimulating economic development and enhancing the quality of life of citizens is extremely important. The metro contributes to improving the transport accessibility of various districts of the city, which in turn affects the optimization of human flows and fosters the harmonious development of the urban environment. An efficient transport network opens up opportunities for new residential, commercial, and business projects, ensuring a more even distribution of economic activity across the city.

2. Instances of significant accidents occurring during subway construction

The examples presented below underscore the critical necessity of rigorous engineering analysis and meticulous planning in the execution of construction projects, particularly in historically significant and densely populated areas, in order to minimize the risks posed to surrounding buildings and urban infrastructure.

2.1 Incidents during the construction of the Washington metro in the 1970s

In the 1970s, the construction of the Washington Metro was accompanied by several incidents leading to damage to buildings situated within the influence zone of the construction activities. One of the most significant cases occurred in 1973, when a ground subsidence at the intersection

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of Connecticut and M Streets NW prompted the evacuation of nearby buildings. However, no structural damage to the buildings themselves was observed in this instance. A more serious event transpired in 1975 in the 100 block of D Street SW, where two buildings, each over 100 years old, were at risk of collapse due to the effects of Metro construction. To mitigate the potential disaster, steel cables were employed to stabilize the structures and prevent further damage. The Washington Post newspaper wrote the following about this in 1975: “Metro has been plagued by cracks in walls and building owners’ complaints (Washington Post, 1975). The Lincoln Room of the National Portrait Gallery suffered severe cracks during construction about four years ago and special shoring was necessary. Two years ago, there was a cave-in at Connecticut and M Streets NW in which nearby buildings were evacuated, although no structural damage occurred. The worst damage occurred in the 100 block of D Street SW, where two buildings more than 100 years old nearly collapsed and Metro had to use steel cables to hold them together.”

2.2 The accident at the Hangzhou metro construction site in 2008

A comprehensive study by Qian & Lin (2016), focused on the systematization and management of risks arising throughout the entire lifecycle of underground engineering structures, provides a detailed analysis of the frequency and significance of accidents that occurred in China between 2001 and 2013 during the construction of subway systems. Specifically, on November 15, 2008, a major collapse occurred at the construction site of Line 1 of the Hangzhou Metro, resulting in a 75-m stretch of Fengqing Avenue sinking to a depth of 15 m, with 11 vehicles falling into the resulting pit. The collapse of the ground outside the foundation pit caused the loss of stability of the foundation walls, leading to the failure of the supporting system, and a large amount of contaminated water flooded into the pit. Around the construction site, four hazardous buildings had to be demolished, and residents within a 500-m radius were evacuated and relocated. Work on 35 subway stations under construction across the city was halted. According to Gong & Zhang (2012), the accident resulted in 21 fatalities and 24 injuries, making it one of the most serious catastrophes in the history of metro construction in China.

2.3 The accident at Pinheiros station (São Paulo) in 2007

The accident at Pinheiros Station (São Paulo), which occurred on January 12, 2007, stands as the most catastrophic incident in the history of underground construction in Brazil (Tunnels and Tunnelling, 2025). It resulted in substantial material damage to both the station and the surrounding infrastructure, along with partial destruction of local facilities. Tragically, the incident claimed the lives of seven

individuals, as reported by Nieble et al. (2010). An expert commission appointed to investigate the event, as described by Barros et al. (2008), concluded that the primary causes of the accident were critical design flaws and significant errors in the construction process.

In each of the cases examined, the accidents were caused by the impact of metro construction activities on the surrounding environment and existing buildings. These activities affected both residential properties and elements of urban infrastructure, highlighting the high level of risk associated with metro construction in densely developed urban areas. All the incidents posed a threat to human life and health and required significant efforts to mitigate the consequences, underscoring the need for comprehensive safety measures.

An analysis of these events emphasizes the importance of meticulous oversight of construction projects in densely populated and rapidly developing cities. While the accidents in Washington and São Paulo were primarily related to design and geotechnical deficiencies, the incident in Hangzhou serves as an example of a catastrophe resulting from errors in the design and construction processes, with more extensive and devastating consequences. In Washington, evacuation and stabilization efforts affected only a limited number of buildings. In Hangzhou, the accident necessitated the demolition of several structures and the evacuation of residents within a 500-m radius of the disaster zone, leading to a more extensive intervention. In São Paulo, efforts were focused on the restoration of the damaged station and adjacent infrastructure.

3. The application of probabilistic approaches to reliability assessment in the context of objects interacting with the geological environment

The examples of major accidents in subway construction vividly demonstrate the critical importance of accounting for a multitude of risk factors in the design and construction of underground structures. Consequently, to enhance safety and reliability, a probabilistic approach is increasingly being employed, which enables a more accurate consideration of uncertainties and potential risks. The use of probabilistic analysis methods in design, including in the construction of underground facilities, has become a vital tool for minimizing errors and preventing accidents. This approach allows for a more effective assessment of potential risks and the adaptation of design solutions in the context of complex geological and construction conditions.

The presented examples of major accidents during subway construction vividly illustrate the critical importance of accounting for complex interactions among multiple risk factors under conditions of uncertainty. This necessity highlights the value of adopting fundamental approaches

to structural reliability, which are thoroughly addressed in authoritative sources such as EN 1990 Eurocode and ISO 2394 (ISO, 2015), alongside theoretical advancements exemplified by the Probabilistic Model Code (JCSS, 2001) developed under the auspices of JCSS.

These key documents provide a systematic framework for understanding and applying the principles of structural reliability. EN 1990 Eurocode and ISO 2394, in particular, outline fundamental concepts and methodologies, while the “Probabilistic Model Code” emphasizes the use of probabilistic methods for evaluating structural reliability, offering detailed models for critical variables and probabilistic assessment techniques.

To determine the probability of failure of a structure (system), a limit state function is used, which describes the conditions under which the structure or its elements lose their functionality. This function is developed based on the established methodology for solving a specific deterministic problem within the framework of limit state theory. When solving reliability theory problems, it is particularly difficult to determine the risk of reaching a limit state based on solving the corresponding statistical dynamics problem. Currently, the following methods are used to solve such problems: numerical integration methods, Monte Carlo simulations, and First and Second Order Reliability Methods (FORM/SORM), among others. The Monte Carlo method (Monte Carlo Simulation) is a numerical technique for solving mathematical problems, based on the modeling of random variables and conducting a large number of statistical trials using random variables in accordance with known distribution laws. This method is quite universal and is used to assess the reliability and safety of building structures and foundations when solving reliability theory problems based on modeling random variables and constructing statistical estimates, as well as for developing possible (probable) processes or outcomes using various random number generation methods, which are then applied to a real model.

The application of the probabilistic-statistical approach in reliability calculations for complex technical systems requires a functional description and comparison of resistance and loading processes, resulting in the determination of an integrated operability function. In this context, loads and impacts are considered through the nature of random processes. In other words, probabilistic-statistical reliability calculations of systems are based not on specific (deterministic) values of loads, but on discrete or continuous probability distribution functions of random variables.

The Monte Carlo method has been extensively utilized in geotechnical design, with notable applications demonstrated in works by Baecher & Christian (2003), Chalermyanont & Benson (2004), Griffiths & Fenton (2007), Pereira & Caldeira (2011), among others. The comprehensive assessment of the reliability of both structural and geotechnical systems, as well as individual structures, necessitates the incorporation of the probabilistic nature of external loads and impacts,

alongside the inherent variability of material properties and soil conditions. This is of paramount importance, particularly for structures situated in aggressive environments, such as underground structures exposed to moist soil and groundwater. The Monte Carlo method for statistical analysis has proven to be a robust and highly effective tool in the evaluation of structural element and foundation reliability and safety. Its application in geotechnical design is extensive, and it adeptly addresses uncertainties within input parameters, yielding more precise and reliable results. Thus, it has become an indispensable component of contemporary engineering analysis.

Reliability assessment methods have found increasingly broad application in tunnel engineering, facilitating the development of safe and economically justified design solutions. Foundational principles regarding safety factors and reliability in geotechnical engineering were established in the seminal work of Duncan (2000), which laid the groundwork for subsequent research in quantitative risk evaluation. A significant contribution to the understanding of the soil-structure interaction system was made by Wang et al. (2014), who analyzed the processes of soil swelling and shrinkage affecting tunnel settlement. Wang et al. (2023) provided a comprehensive review of contemporary approaches to reliability analysis of tunnel structures, encompassing the evaluation of support system behavior and potential directions for methodological advancement in engineering practice. The study by Sun & Dias (2022) emphasized the critical importance of accounting for geotechnical uncertainty, quantitatively assessing seismic-induced tunnel deformations under variable soil properties through stochastic modeling. Modern principles of reliability-based design and assessments of the effectiveness of design formats were addressed by Lin & Yuan (2023), while Zhang & Lin (2021) presented multi-objective optimization approaches aimed at mitigating tunnel-induced damages.

Despite considerable progress in numerical-probabilistic analysis and the formalization of engineering solutions, there remains a compelling need to develop integrated methodologies for reliability assessment of the coupled “soil mass – underground structure” system. This need is particularly pronounced in high-density urban environments, where geological heterogeneity combines with anthropogenic factors to substantially influence the stress-strain state of subterranean constructions.

The description, modeling, calculation, and analysis of the results of these processes is a complex task, the solution of which is impossible without the use of numerical methods and the construction of correct calculation schemes. For structures operating under significant ground contact conditions, the choice of the soil model must be substantiated to a sufficiently high degree. In turn, obtaining adequate results is impossible without conducting thorough geotechnical surveys and appropriate soil testing. This paper describes the experience of using probabilistic-statistical tools (Monte Carlo simulation) in determining and analyzing the stress-strain state (SSS) of

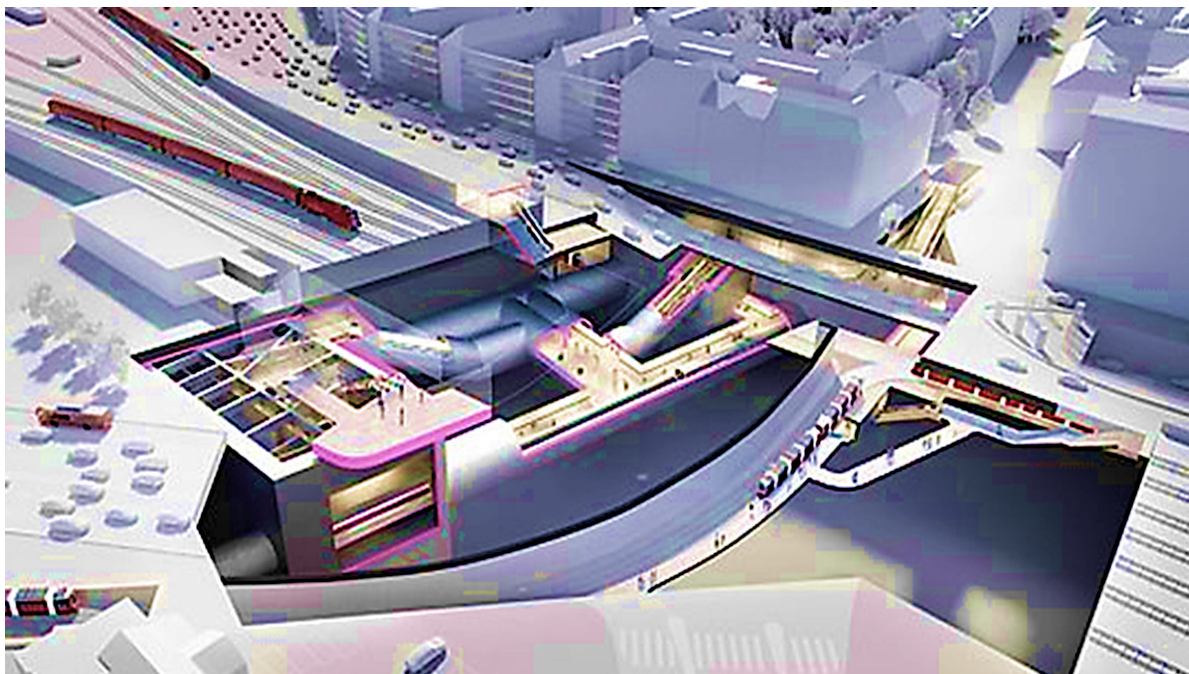


Figure 1. Visualization of Matzleinsdorfer Platz station (© Wiener Linien GmbH & KG. Used with permission).

the geotechnical system at the underground construction site in Vienna – Schacht Triester Straße (U2/18 Matzleinsdorfer Platz, Figure 1, 2).

4. Engineering-geological conditions in the area of the Vienna metro under construction

4.1 The role of metro systems in urban mobility and sustainable development: The case of the Vienna metro

The Vienna metro constitutes a fundamental element of the city's transportation infrastructure, serving as a key international transit hub and an essential component of urban mobility. As the capital of Austria, Vienna is a principal hub for tourism, education, and commerce, drawing millions of visitors and hosting a substantial student population. The metro system facilitates the efficient integration of the city's districts, significantly contributing to economic growth and enhancing societal well being.

Over the past 47 years, the Vienna metro has undergone five distinct phases of expansion. The current phase involves the construction of the U5 line and the modernization and extension of the U2 line. This project is characterized by its ambitious scope and technical complexity, as it is being developed within a highly urbanized area that presents challenging engineering and geological conditions (Vienna, 2025).



Figure 2. Reinforced concrete structures of the metro line under construction (photo by Julian Sigmund (TU Wien), 2023).

Construction methodologies are selected based on the geotechnical characteristics along the proposed alignment. In sections with favorable subsurface conditions, the open-cut excavation method is employed. However, in more geologically complex areas, the closed method is utilized, incorporating tunnel boring machines for efficient and safe excavation. Furthermore, the New Austrian Tunnelling Method (NATM) is implemented to optimize tunnelling processes, ensuring structural integrity and mitigating risks associated with underground construction in diverse geotechnical environments.

The territory of Vienna is situated on the eastern edge of the Alps, with the foothills of the Alps extending to the west and south, while the Pannonian Lowlands are located to the east and north. The absolute elevation of the city ranges from 151 m above sea level in Lobau to 542 m at Hermannskogel. To the north-west, west, and south-west of the city extends the Vienna Woods (Wienerwald), with its elevations.

The geomorphological structure of Vienna's territory is represented by a complex of Quaternary abrasion-erosion terraces that gradually descend toward the Danube River (Figure 3). These terraces are intersected by the valleys of streams and rivers, such as the Wien, Ottakringer, and Alserbach. The Vienna terraces consist not only of pure gravel but also contain sand, silt, and clay, often occurring as thin lenses that extend laterally for several hundred meters. The alluvial deposits of the Danube within the city area are significantly intermingled and fairly evenly distributed; however, to the

south of the confluence of the Vienna and Danube rivers, there is a marked increase in sandy inclusions. This is particularly noticeable on the Stadt and Arsenal terraces, and to a lesser extent on the Wienerberg and Laaerberg terraces. The accumulation of sandy lenses results from the Vienna River transporting sandy erosional material from the western part of the Vienna Woods.

The geological structure of the city's territory is primarily composed of sedimentary deposits of Quaternary and Neogene age. The metro infrastructure is situated within the sedimentary layers of the Miocene epoch. The lower soil strata at the construction site consist of Neogene Miocene deposits, which are overlaid by Quaternary Danube alluvial deposits from the Pleistocene. Based on these characteristics, the following lithological units were distinguished, as shown in Krepper & Kronberger (2022), Figure 4:

- Danube quartz gravel (QG3);
- Miocene deposits (MZ).

The Miocene layer (MZ) is further subdivided into the following engineering-geological units:

- MZa (silt with a higher clay content);
- MZb (silt with a lower clay content);
- MZc (silty fine and medium sand);
- MZd (sand and gravel).

In connection with the expansion of line U2 and the construction of the new U5 line, a thorough investigation of the physicochemical properties of the soil beneath the buildings was carried out prior to the commencement of

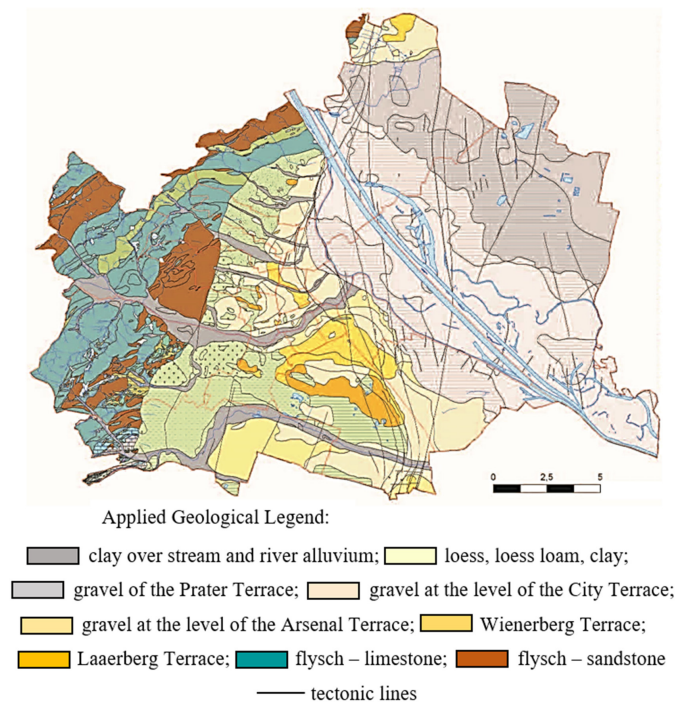


Figure 3. Applied Geological Map of Vienna (modified from Brix, 1972).

Legend reduced for clarity.

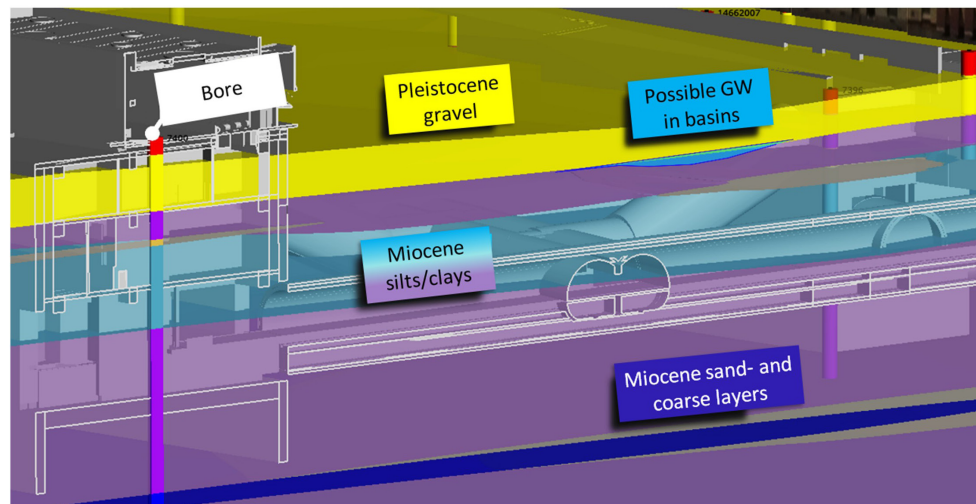


Figure 4. A subterranean model of the station Matzleinsdorfer Platz (Krepper & Kronberger, 2022).

construction works (U-Bahn Wien, 2025). This also involved the evaluation of their foundations, including the assessment of depth, material composition, and technical condition. In addition to the inspection of basements, an examination of the apartments in all buildings along the future metro route was conducted, during which their condition was documented to identify pre-existing damages as well as those occurring during the construction process (Mein Bezirk, 2017).

Throughout the course of the U2xU5 project, all the technical challenges posed by the Vienna soil in its various forms have been revealed: dense development with historical buildings on the surface, groundwater layers, and recurrent water ingress into tunnels and shafts (Wiener Linien, 2025). For instance, during the construction of the shaft for the future metro station on Reinprechtsdorfer Straße, additional measures were required, such as the installation of temporary foundations to counteract the forces exerted by the soil and provide further protection for the shaft. Moreover, the tunnel construction process turned out to be significantly slower than initially anticipated: the behavior of the surrounding soil during the excavation deviated from expectations, resulting, for example, in ground movements at the surface. To address these issues, further measures were necessary, which led to delays in the project schedule.

The construction of the metro line in the vicinity of Rathaus Square encountered considerable difficulties due to the presence of high groundwater levels. In order to maintain dry conditions within the tunnels during the construction phase, Wiener Linien implemented a large-scale groundwater drainage system extending from Rathaus to the Danube Canal, prior to the commencement of drilling activities in 2022.

Despite this preventive measure, unforeseen interactions with groundwater layers occurred during the construction process, including, for example, the inflow of 100,000 L of water from an aquifer into the tunnel. As a result, construction

activities had to be temporarily suspended, additional pumping equipment was installed, and new wells were constructed to facilitate water extraction. The excavation in the water-saturated bedrock of the Danube led to additional soil deformations, which necessitated the implementation of supplementary safety measures and contributed to the slowing down of construction processes.

5. Modeling and calculations

5.1 Soil model

When analyzing the complex technical system of “soil mass – underground structure”, it is necessary to address an integrated problem that includes evaluating the stiffness and strength of the foundation, the spatial stiffness and strength of the structure itself, its elements and joints, as well as their interaction and stress redistribution. This requires treating the foundation and the structure (building) as a unified spatial stochastic system and developing methods for its numerical analysis.

To automate the assessment of the stress-strain state in such complex systems, it is advisable to use the finite element method (FEM) in conjunction with probabilistic approaches. This approach enables the monitoring of reliability not only for individual system components but also for the entire structure as a whole. In this context, research in the field of numerical methods applied to probabilistic formulations for the “soil mass – underground structure” system remains a current and critical task in modern engineering practice.

In determining the key forces and deformations within the “soil mass – underground structure” system, the physical and mechanical properties of the soil play a pivotal role. These properties can significantly vary with

depth and horizontal extent, even within a single soil layer. The variation in these properties is due to the diversity in the composition and structure of soils, their polydispersity, multiphase nature, polyminerality, and the characteristics of particle bonding, as well as the impact of geological and anthropogenic processes. Additionally, these parameters are influenced by factors such as stress magnitude, time, and engineering-geological processes that alter soil properties.

The correctness of the computational model of the “soil mass – underground structure” system, considered in this study, undoubtedly depends on the choice of an appropriate soil model. For numerical modeling, the Hardening Soil (HS) and Hardening Soil Small (HSS) models were selected, which are advanced elastoplastic models, implemented in the PLAXIS software suite.

The relationships for selected parameters of this model are presented according to Schanz et al. (2019). The parameter E_{50} is the confining stress-dependent stiffness modulus for primary loading which can be described in Equation 1.

$$E_{50} = E_{50}^{ref} \left(\frac{c \cdot \cos \phi' - \sigma_3 \cdot \sin \phi'}{c \cdot \cos \phi' - p^{ref} \cdot \sin \phi'} \right)^m \quad (1)$$

For unloading and reloading, another stress-dependent stiffness modulus is defined as:

$$E_{ur} = E_{ur}^{ref} \left(\frac{c \cdot \cos \phi' - \sigma_3 \cdot \sin \phi'}{c \cdot \cos \phi' - p^{ref} \cdot \sin \phi'} \right)^m \quad (2)$$

For oedometer conditions of stress and strain, the oedometer stiffness modulus E_{oed} for primary loading is:

$$E_{oed} = E_{oed}^{ref} \left(\frac{c \cdot \cos \phi' - \sigma_3 \cdot \sin \phi'}{c \cdot \cos \phi' - p^{ref} \cdot \sin \phi'} \right)^m \quad (3)$$

Where p^{ref} is the reference pressure, E_{50}^{ref} is a secant stiffness modulus defined for a reference stress; m is the parameter that defines p^{ref} and minor principal stress, σ_3 , E_{oed}^{ref} is the oedometer tangent stiffness modulus defined for a reference stress. E_{ur}^{ref} is the Young's modulus for unloading and reloading defined for a reference stress, ϕ' – angle of internal friction.

Both E_{50}^{ref} and E_{oed}^{ref} are necessary for the formulation of hardening model. The E_{50}^{ref} controls the magnitude of the plastic strains associated with E_{50}^{ref} the shear yield surface. Similarly, E_{oed}^{ref} is used to control the magnitude of plastic strains originating the yield cap.

The coefficients of variation for the varying characteristics were taken in accordance with the JCSS Probabilistic Model Code, Section 3.7: Soil Properties, as well as with the consideration of the research by Phoon & Kulhawy (1999), Griffiths & Fenton (2007).

5.2 Calculation scheme

The calculation model of the “soil mass – underground structure” system for the metro section was developed and analyzed using the PLAXIS program (Figure 5). The dimensions of the soil massif are 110×200 m.

The specified distributions for loads and materials are mainly taken from the Probabilistic Model Code, draft Eurocode 7 prEN 1997-1:2022 (E) (CEN, 2003), and from some other sources of scientific literature, including Griffiths & Fenton (2007), Misra & Roberts (2007). Geometric dimensions of building structures and their physical and mechanical characteristics are accepted in accordance with the design data.

Following a comprehensive series of calculations, the full set of parameters characterizing the stress-strain state of the “soil mass – underground structure” system within the

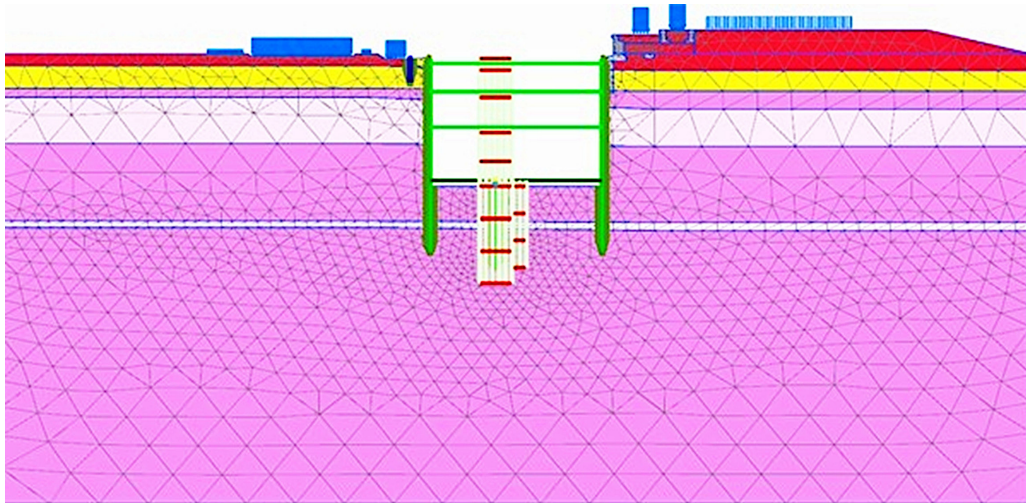


Figure 5. Calculation scheme.

metro construction site was determined. Figure 6 presents one of the possible distributions of vertical deformations in the analyzed system.

In the course of the calculations, empirical distributions of the parameters were modeled based on the initial probabilistic and statistical characteristics, which constituted the input data for the formulation and solution of this problem.

5.3 Calculation algorithm

The conducted numerical experiment determines all the parameters of the stress-strain state of the system using Finite Element method and Monte Carlo method in the following order. Values of some parameters of the soils used in the calculation (γ , ϕ' , c , E_{50}^{ref} , E_{oed}^{ref} , E_{ur}^{ref}) and laws of distribution of values are shown in Table 1. The SSS random values of the system are fixed.

Since the calculation model includes not only the self-weight of the soil but also additional loads (from buildings, tram tracks, vehicles, and sidewalks with pedestrians) (see Figures 6), these loads were also subject to variation. The magnitudes of the loads and their coefficients of variation

were determined in accordance with regulatory documents and European standards.

In the design and construction of underground structures in densely urbanized areas, one of the key requirements is to ensure the preservation of surrounding buildings. This requirement is more stringent than the conventional settlement limit (SLS), as the permissible deformations for existing buildings are always smaller than those for newly constructed structures. Therefore, when designing underground structures, it is essential to adopt structural solutions that ensure not only the reliability of the structure itself but also the safety of adjacent buildings. In this context, a quantitative risk criterion for existing buildings may involve limiting the additional settlements of urban buildings to an acceptable level: the magnitude of additional deformations must not exceed the allowable threshold. To assess the probability of exceeding these deformations, it is advisable to apply the statistical testing method (Monte Carlo method) based on calculations performed using the PLAXIS software.

At the second stage, the rules and probabilistic parameters of two random variables are defined: 1) the maximum permissible values of additional vertical deformations

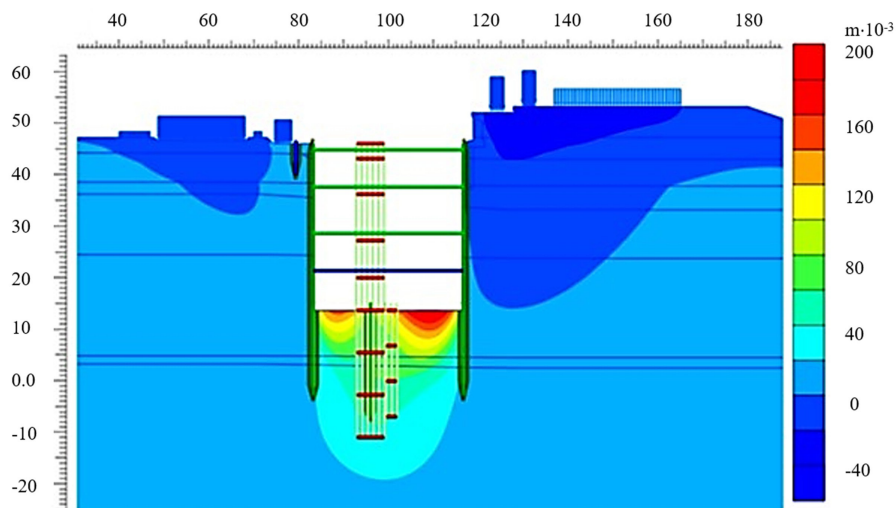


Figure 6. Distribution of vertical deformations as a result of one of the calculation variants.

Table 1. Parameters of soils accepted for calculation.

Parameters	Designation	Distribution	COV	Mean (1 layer)	Mean (2 layer)	Mean (3 layer)	Mean (4 layer)	Mean (5 layer)
Weight density (kN/m ³)	γ	normal	0.05	19	21	20	20	20
Angle of internal friction (°)	ϕ'	normal	0.1	25	35	25	25	25
Cohesion (kPa)	c	normal	0.1	1	1	30	30	30
Secant stiffness modulus (MPa)	E_{50}^{ref}	log-normal	0.2	10	60	20	20	20
Oedometer modulus (MPa)	E_{oed}^{ref}	log-normal	0.2	10	60	25	25	25
Secant stiffness modulus (MPa)	E_{ur}^{ref}	log-normal	0.2	40	240	100	100	100

S_{add} . 2) the calculated values of additional vertical deformations s , which are determined through Finite Element Method (FEM) calculations.

The following distribution laws are adopted for the input parameters:

- The distribution of the random variable $P_{S_{add}} = P_{S_{add}}(S_{add})$, representing the maximum additional vertical deformations, is assumed to follow a normal distribution, with a mean value $m_{S_{add}} = 0.03$ and a standard deviation $\sigma_{S_{add}} = 0.003$ m.
- The distribution of the random variable $P_s = P_s(s)$, representing the calculated deformations, is also assumed to follow a normal distribution, with parameters m_s and σ_s , which are determined from the results of FEM calculations.
- Using the Monte Carlo statistical testing method, 10^5 trials were conducted with the Mathcad software suite. For each trial, the condition $s \leq S_{add}$ was checked. After performing all N trials, the probability of exceeding the additional deformations during the design service life was calculated as the ratio of the number of trials where $S_{add} - s < 0$ to the total number of trials. An analogous calculation is also carried out for the 50-year service period.
- According to EN0, the probability of reaching the limit state (probability of structural failure) P_f can be calculated using the formula: $P_f = \Phi(-\beta)$, where Φ is the cumulative distribution function of the standardized normal distribution, and β is the reliability index.
- The failure probability was found to be $P_f = 0.002$ for a 1-year period and $P_f = 0.0055$ for a 50-year period. The reliability indices were $\beta_l = 3$ and $\beta_{50} = 2.71$, which are acceptable values for the SLS state.

The value of S_{add} set at 3 cm, was determined based on a comprehensive analysis of monitoring data for buildings

and structures, as well as theoretical and applied research. This choice is grounded not only in empirical data but also in the practical consideration of allowable deformations that ensure the safety of existing buildings.

With high probability, it can be assumed that the surrounding construction primarily falls under consequence class CC2, for which the minimum value of the reliability index β is 2.9 for a 1-year period and 1.5 for a 50-year period (according to the EN 1990:2002). The three reliability classes – RC1, RC2, and RC3 – can be associated with the three consequence classes – CC1, CC2, and CC3. According to EN1990:2002 (CEN, 2003), the designations RC1, RC2, and RC3 represent reliability classes, which can be determined using the reliability index β . Table 2 also provides the recommended minimum values for the reliability index associated with each reliability class (in accordance with EN 1990:2002).

The calculation results are summarized in Table 3. Figures 7 and 8 depict, respectively, the probability density functions (PDFs) and cumulative distribution functions (CDFs) for three principal random variables employed in the conducted analysis. These variables comprise the calculated additional settlement (s , represented by the light gray curve), the additional settlement accumulated over a 50-year period (s_T , dark gray curve), and the maximum allowable additional settlement (S_{add} , black curve). The presented functions facilitate a quantitative assessment of the range of possible values of the investigated random variables, as well as the probability of exceeding the established serviceability limit within the designated design service life of the structure.

5.4 Analysis and results

Based on the results of finite element method (FEM) calculations with varying initial data, all characteristics of the stress-strain state of the “soil mass – underground structure”

Table 2. Reliability classes and recommended minimum values for reliability index β (CEN, 2003).

Reliability Class	Minimum values for β					
	Ultimate limit states		Fatigue		Serviceability	
	1 year reference period	50 years reference period	1 year reference period	50 years reference period	1 year reference period	50 years reference period
RC1	5.2	4.3				
RC2	4.7	3.8		1.5 to 3.8	2.9	1.5
RC1	4.2	3.3				

Table 3. Results of probabilistic calculations of exceeding for additional ultimate deformation of the base.

Name of the values	Reference period	
	1 year	50-year
Probability of exceeding the additional limit deformation of the base	2×10^{-3}	5.5×10^{-3}
Recommended EN minimum values (SLS) (normal level)	3.2×10^{-3}	8.1×10^{-2}
Reliability index β	3	2.71
Recommended minimum values (SLS) (normal level)	2.9	1.5

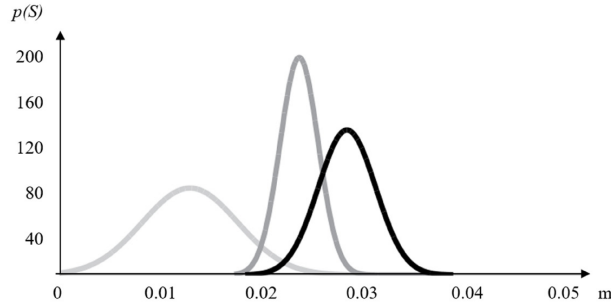


Figure 7. Probability density functions of s , s_T and S_{add} .

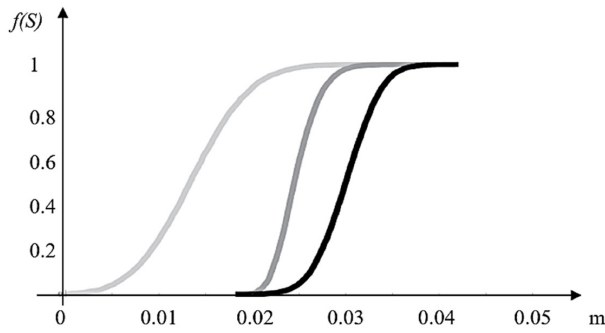


Figure 8. Cumulative distribution functions of s , s_T and S_{add} .

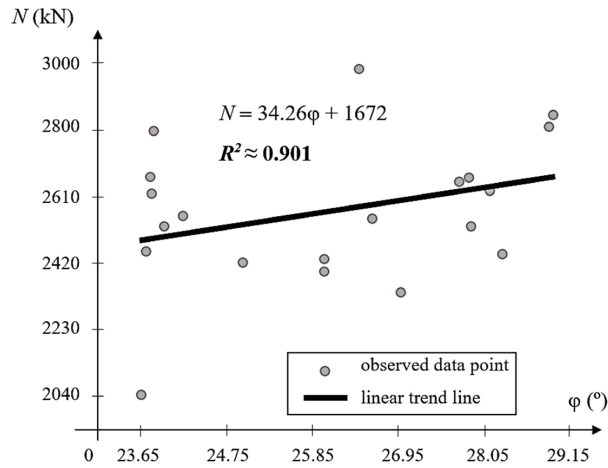


Figure 9. 90.1% of the variability in the longitudinal N force values can be explained by a linear regression model.

system were derived, along with the coefficients of variation for these characteristics, which are as follows:

- for longitudinal forces N , $COV = 0.08$ (8%);
- for bending moments M , $COV = 0.09$ (9%);
- for shear forces Q , $COV = 0.11$ (11%);
- for vertical displacement s , $COV = 0.237$ ($\approx 24\%$);
- for horizontal displacement, $COV = 0.2$ (20%).

Following the verification of the obtained values for all the aforementioned parameters using the agreement criterion at a significance level of 0.05, it was established that these parameters exhibit a normal distribution.

Probabilistic analyses conducted using the Mathcad Prime software package revealed that the probability of system failure, based on the criterion of additional vertical deformations, is $P_f = 0.002$, corresponding to a safety index $\beta = 3.0$, which is considered satisfactory and complies with the guidelines specified in the “Probabilistic Model Code.”

5.5 Regression analysis

As part of this study, a regression analysis was conducted to evaluate the influence of the varying factors – γ , ϕ , c , E_{50}^{ref} , E_{oed}^{ref} , E_{ur}^{ref} – on the primary parameters of the stress-strain state, which were derived from the calculations: M , N , Q , and the vertical and horizontal deformations of the soil mass. Prior to the commencement of the numerical calculations using the PLAXIS software, the values of the varying factors were averaged over the thickness of the soil layer involved in the collaborative interaction of the underground structure with the foundation. This was necessary due to the fact that the numerical model encompasses five distinct soil layers, each characterized by its unique properties. It should be noted that, when compared to the initial results, such averaging exerted a minimal effect on the magnitudes of forces and deformations, and can be employed, for instance, in preliminary (approximative) calculations.

Regression analysis constitutes a set of statistical methods used to evaluate the relationships between variables, and is applied to assess the degree of interdependence between variables as well as to model future dependencies. The regression analysis incorporates several models, the most prevalent of which are: linear, multilinear (multiple linear), and nonlinear models. In this study, the first two models were utilized, and the regression analysis was conducted based on the least squares method.

After an extensive evaluation of various alternatives, an intriguing pattern was identified: the values of the longitudinal force N in the shaft wall can be satisfactorily represented by a linear regression model, as 90.1% of the variability in the longitudinal force values is accounted for by this model, as shown in Figure 9. In this two-factor experimental setup, the independent variables were the unit weight of the soil γ and the angle of internal friction of the soil ϕ , while the dependent variable was the longitudinal force N . All computations were performed using the Mathcad software environment. Subsequently, an analysis was conducted to determine the relationship between the unit weight of the soil and the longitudinal force values, revealing that 86% of the variability in the longitudinal force can also be described by the linear regression model (Figure 10).

Furthermore, an experiment was conducted to establish the correlation between vertical deformation values and the

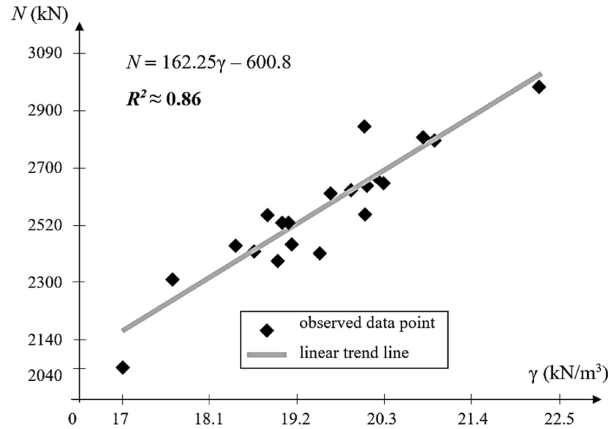


Figure 10. 86% of the variability in the longitudinal N force values can be explained by a linear regression model.

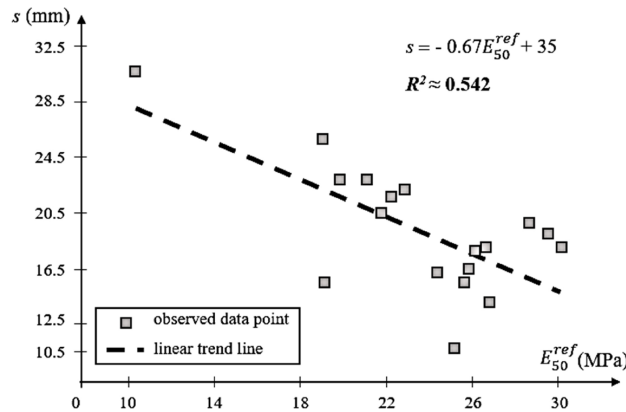


Figure 11. 54.2% of the variability in vertical deformation can be explained by a linear regression model.

unit weight of the soil γ as well as the modulus E_{50}^{ref} . It was found that the variability in vertical deformation values is also linearly dependent on these two independent variables; however, the linear model accounts for only 54.2% of the variability in the deformation values, as shown in Figure 11.

6. Conclusions

The modeling of empirical distributions for the parameters of the “underground structure – soil” system facilitated the construction of a more refined statistical model, which encompasses the full spectrum of parameters governing the stress-strain state of this system. The extensive variation analyses performed utilizing the PLAXIS software suite yielded coefficients of variation for the primary forces and deformations, which constitute a critical component in the reliability evaluation of structural integrity. These coefficients are instrumental not only in the design and strength assessment but also in the selection of cross-sectional

profiles and reinforcement for reinforced concrete structures of comparable typologies.

The obtained range of displacement values for buildings during the construction process, alongside the reliability assessment conducted via Monte Carlo simulations, enabled an in-depth evaluation of the impact of additional settlement on surrounding structures. A reliability assessment algorithm for the “underground structure – soil” system was applied, based on the criterion of additional settlement of buildings, which was operationalized for the construction site located at Schachtes Triester Straße (Baulos U2/18 Matzleinsdorfer Platz). The system’s failure probability was calculated to be 0.002 (for a period of 1 year) and 0.0055 (for a period of 50 years), values that are considered entirely acceptable within the framework of the Serviceability Limit State (SLS) condition. The reliability indices $\beta_1 = 3.0$ and $\beta_{50} = 2.71$ further substantiate the system’s robustness under the specified loading conditions and operational parameters. Furthermore, the results derived from this analysis are applicable in forecasting potential settlements during metro construction in any geotechnical context, thereby enabling a proactive approach to assessing the impact of construction activities on pre-existing structures and mitigating the risk of induced damage.

A comprehensive regression analysis was undertaken to investigate the influence of varying soil characteristics on the principal parameters of the stress-strain state of the system, revealing a substantial impact of these characteristics on the computed results. Correlational dependencies among several parameters were established, facilitating a more precise characterization of the behavioral model of both the soil and the structures during their operational lifecycle. This insight is of paramount significance for further enhancing the precision of analytical models and advancing the design of infrastructure situated within regions characterized by complex geotechnical conditions.

Declaration of interest

The authors have no conflicts of interest to declare. All co-authors have observed and affirmed the contents of the paper and there is no financial interest to report.

Authors’ contributions

Oksana Kichaieva: conceptualization, data curation, methodology, investigation, visualization, writing – original draft. Dietmar Adam: conceptualization, supervision, validation, formal analysis, project administration, resources, software, writing – review and editing.

Data availability

The data sets generated analyzed in the course of the current study are available from the corresponding author

upon request. All data produced or examined in the course of the current study are included in this article.

Declaration of use of generative artificial intelligence

Generative artificial intelligence (GenAI), specifically OpenAI's ChatGPT, was used solely to assist with minor language editing and grammar correction. The use of this tool was limited to improving linguistic clarity and did not contribute to the scientific content or interpretation of the work. All edits were reviewed and approved by the authors, who take full responsibility for the final content.

List of symbols and abbreviations

c	Cohesion
m	Parameter, that defines the reference pressure and the minor principal stress, σ_3
m_s	Mean value of s
$m_{S_{add}}$	Mean value of S_{add}
P_f	Probability of failure
p^{ref}	Reference pressure
s	Vertical displacement
$CC1, CC2, CC3$	Consequence classes according to EN 1990:2002 (CEN, 2003)
$CDFs$	Cumulative distribution functions
COV	Coefficient of variation
E_{s0}	Secant modulus
E_{s0}^{ref}	Secant stiffness modulus for a reference stress
E_{ur}	Stiffness modulus for unloading/reloading
E_{ur}^{ref}	Secant stiffness modulus for unloading/reloading at a reference stress
E_{oed}	Oedometric modulus
E_{oed}^{ref}	Oedometric modulus for unloading/reloading at a reference stress
EN	European Norm
FEM	Finite Element Method
HS	Hardening Soil
HSS	Hardening Soil Small
ISO	International Organization for Standardization
JCSS	Joint Committee of Structural Safety
M	Bending moment
MZ	Miocene deposits, Miocene layer
MZa	Silt with a higher clay content
MZb	Silt with a lower clay content
MZc	Silty fine and medium sand
MZd	Sand and gravel
N	Longitudinal force
NATM	New Austrian Tunnelling Method
P_s	Distribution of random variable s (calculated deformations)
$P_{S_{add}}$	Distribution of random variable S_{add}

$PDFs$	Probability density functions
Q	Shear force
QG3	Danube quartz gravel
$RC1, RC2, RC3$	Reliability classes according to EN 1990:2002 (CEN, 2003)
S_{add}	Additional vertical deformation
S_T	Additional settlement accumulated over 50-year period
SLS	Serviceability limit state
SSS	Stress-strain state
β	Reliability index
β_1	Reliability index for 1-year period
β_{50}	Reliability index for 50-year period
γ	Weight density
σ_3	Minor principal stress
σ_s	Standard deviation of s
$\sigma_{S_{add}}$	Standard deviation of S_{add}
ϕ'	Angle of shearing resistance in terms of effective stress (angle of internal friction)

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Interpretation of large diameter preliminary pile tests in Ampthill Clay, West Walton and Oxford Clay formations for Thame Valley Viaduct

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Article

Keywords

Pile
Test pile
Back analysis
Plaxis
Osterberg cell

Abstract

Two instrumented preliminary test piles have been installed and tested in Aylesbury, UK, using Osterberg Cell bi-directional load test method, for the purpose of verifying the construction methodology and load performance of the bored piled foundations designed for the 880m long HS2 Thame Valley Viaduct. The local ground conditions comprise stiff clay and mudstone of Ampthill Clay, West Walton and Oxford Clay Formations which constitute part of the Ancholme Group. The significant length of the asset allowed a comprehensive ground investigation package of site and laboratory testing to be undertaken in developing a refined detailed design in accordance with the recommendations of Eurocode 7. The field test results of the two test piles have been interpreted and back-analysed using 3D finite element analyses undertaken to simulate the bi-directional loading of the Osterberg Cell testing method and compared against the ultimate pile capacity estimated during detailed design of the viaduct. Subsequent simulation of the pile loaded from pile head level using the calibrated finite element model provided the required load-settlement characteristic necessary to verify the performance of the pile under serviceability limit state.

1. Introduction

High Speed 2 (HS2) is a high-speed line under construction in England, connecting London to the West Midlands (Phase 1) through approximately 230 kilometres (km) of railway. HS2 Contract C23 (formally Central Area 2 and Central Area 3) covers 80km of railway between North Portal of Chiltern Tunnel (southern end) and the South Portal of Itchington Wood Green Tunnel (northern end). It covers four major geological groups and thirteen geological formations; from the White Chalk Group at the southern end to Ancholme Group, Great Oolite Group and Lias Group to the north. Five sets of preliminary test piles were installed and tested in the five geological groups to verify pile design assumptions made for piled foundations along the route. Table 1 summarises the eleven (11) preliminary test piles for the C23 HS2 project between September 2020 and August 2021.

Figure 1 shows approximate locations of the preliminary test piles for the HS2 project.

Two preliminary test piles (PTP10A and PTP11) were installed and tested using bi-directional Osterberg Load Cell (O-cell) at the 880m long Thame Valley Viaduct (TVV), as part of the Aylesbury Area package of the HS2 project. These test piles were found in the local ground conditions

comprising Ampthill Clay (AMC), West Walton Formation (WWB) and Oxford Clay (OXC). The Arcadis, Setec, COWI Design Joint Venture (ASC) has been appointed by Eiffage, Kier, Ferrovial, and BAM Nuttall Joint Venture to provide detailed design services for the civil engineering assets within HS2. Bauer Keller joint venture (BKJV) was responsible for the installation of the piles while Fugro Loadtest was responsible for testing the piles.

Due to access and programme constraints, the test site was located approximately 200m north of the viaduct's proposed north abutment.

O-cell has transformed pile load test since its invention in the mid-1980s by Dr Jorj O. Osterberg (Hayes, 2012). This method of testing pile has several advantages over conventional pile test methods, including the elimination of the need for a reaction frame or kentledge. O-cell testing method was selected for the HS2 project primarily due to its capability to safely apply very high test loads to full-size test piles. This is achieved by embedding hydraulic jacks within the pile shaft with the reaction to the applied load provided by the ground itself, without the need to handle the heavy reaction system in the form of structural steel frame or concrete kentledge blocked to provide the resistance to the applied load. Furthermore, in the event of any unforeseen failure of

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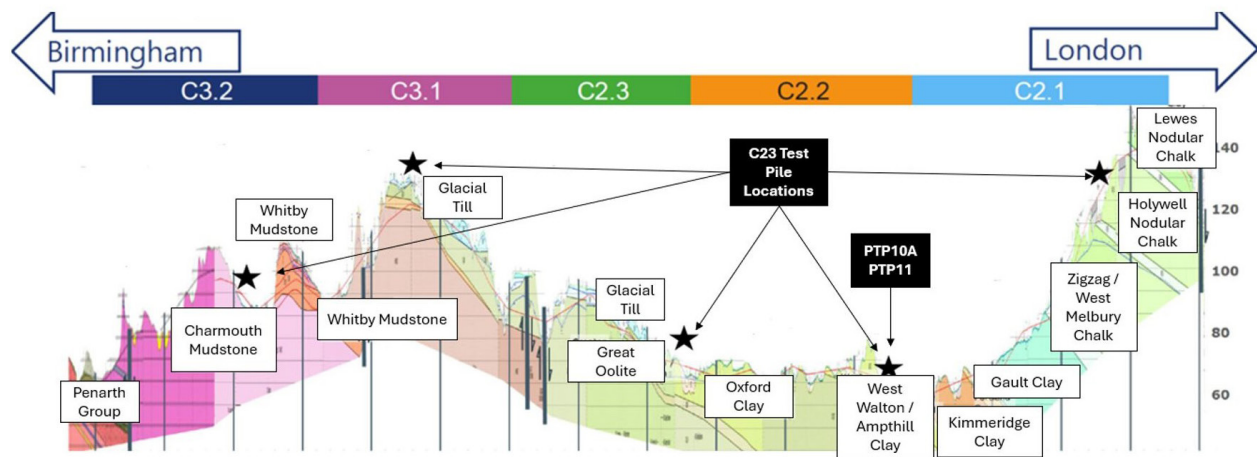


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Table 1. Preliminary test piles for the C23 HS2 project.

Test pile	Geological Formation	Date tested	Pile length (m)	Diameter (mm)	Maximum test load (MN) ¹
PTP1	Chalk	18/10/21	39.7	1500	21.0
PTP2		25/10/21	35.7	2000	17.7
PTP3	Oxford Clay	08/12/20	40.1	1200	7.7
PTP4		14/12/20	40.5	1200	7.7
PTP5	Charmouth	28/09/20	20.8	1500	5.3
PTP6	Mudstone	05/10/20	20.9	1500	6.4
PTP8		09/10/20	26.0	900	6.2/9.32 ²
PTP9		19/10/20	26.0	900	6.5/9.72 ²
PTP10A	Ampthill Clay,	17/03/22	41.1	1500	11.5
PTP11	West Walton and Oxford Clay	09/03/22	39.1	1800	24.2

¹Maximum load applied by O-cell during test (total load is two times O-cell load). ²Maximum load applied to top/bottom O-cell.

**Figure 1.** Approximate locations of the preliminary test piles of HS2 project.

the reaction system, the O-cell test method would only need to cater for more manageable burst hydraulic hose while such event, when occur in conventional pile test methods, could lead to a catastrophic collapse of the steel reaction frame or kentledge assembly. Compared to conventional top loaded test pile using kentledge or reaction frame system, any test load beyond 30MN would be very expensive to perform, if not impossible, whereas O-cell test method could achieve a maximum test load well beyond 300MN. As the need to limit the test load in construction industry prior to the use of O-cell, conventional test piles would be performed on smaller size piles and the test data is then used directly for larger size working piles. The use of O-cell to test full size piles would eliminate the uncertainty of scaling the test data from small diameter test pile to a larger working pile. This includes the effects of construction of full-size pile when using O-cell to test the pile.

Testing pile using O-cell is not without its disadvantages. As the O-cell test applies load bidirectionally with sacrificial load cell(s) cast into the pile shaft unless the test pile is founded in rock or similar ground with very high end bearing capacity,

determining the most appropriate position of the load cell(s) in the pile is critical to maximise the load applied. Any failure of the pile upwards or downwards before achieving the intended load due to incorrect positioning of the cell means premature end of the test. This is possibly the main disadvantage of using such testing method. Another disadvantage is the more complex installation process involving tremie concrete placement over the full length of the pile with the O-cell positioned at some distance above the pile toe. As the load is applied directionally, it is impossible to accurately determine the portion of the mobilized shaft and base resistance to an equivalent load when applied directly from the pile head under normal pile loading situation. This prevents an accurate determination of the magnitude of load holding for maintained load test commonly specified for static load test under the Institution of Civil Engineers' specification for load test (ICE, 2017). As a direct consequence of such limitation, the interpretation of the load-settlement performance of the test pile is also more complex without resolving to full calibration using more advance numerical modelling.

There is very sparse case history information available on pile performance in the local ground conditions of the TVV site. These tests were deemed necessary to fill the knowledge gap of the designers of the HS2 project and this paper presents the interpretation and back-analysis of the test data with the objective of verifying the design approach adopted for the design of the viaduct piled foundations and the performance of the pile under serviceability limit state. It also provides a comparison of the methods of deriving the test pile's load-settlement performance required for pile tested using O-cell.

2. The site

2.1 Geological setting

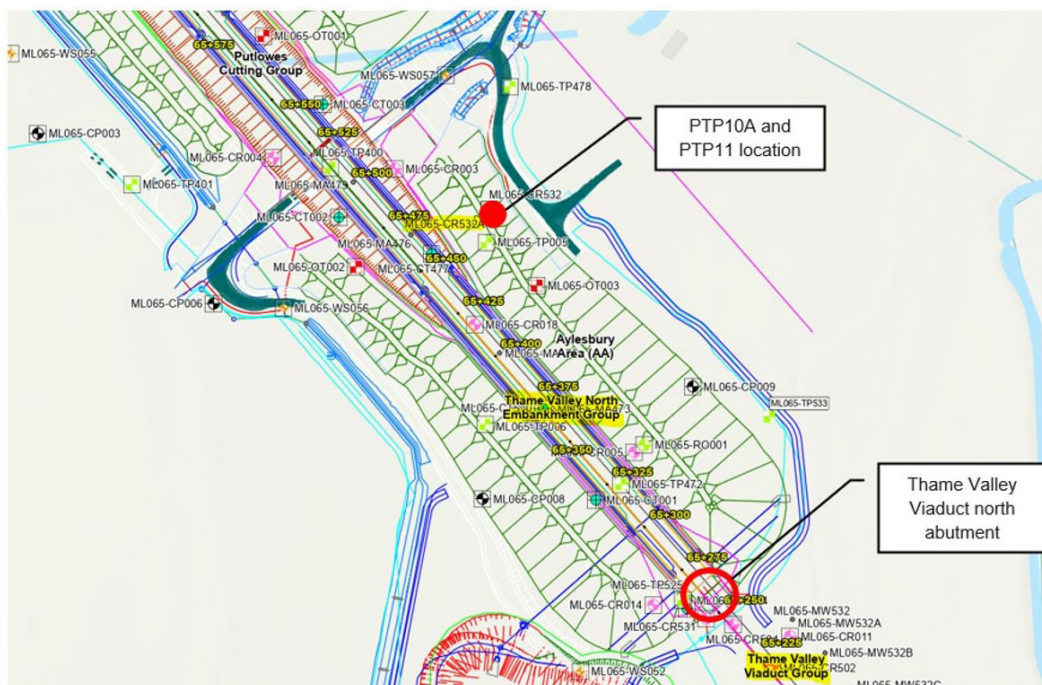
Aylesbury Area is situated in the East Midlands Shelf which contains an Anchohme Group succession of clay and mudstone formation which were deposited in the Upper Jurassic period. The ground conditions at the test site are consistent with the ground conditions along the viaduct and comprise less than 1m of Head Deposits (HEAD) overlying AMC, WWB and OXC to depth greater than 60m. AMC and WWB are generally described as stiff to very stiff Clays or calcareous Clays, with small localised pockets of siltstone and limestone, while OXC is generally described as extremely weak Mudstone.

2.2 Site investigation and soil parameters

The ground conditions at the test site have been characterised based on ground investigation works within a 200m radius, which include 6 No. Cone Penetration Tests (CPT), 11 No. boreholes, 6 No. trial pits and respective in-situ and laboratory tests (see Figure 2). The borehole descriptions at the test site and along with the viaduct are not always consistent which could imply a very significant variation in the thicknesses of AMC and WWB, even in boreholes located adjacent to each other. This possibly highlights the difficulty in identifying the marker horizons that distinguish AMC, WWB and OXC, therefore resulting in very different stratigraphy being presented in adjacent boreholes. Despite the discrepancies noted above, the analysis of the Atterberg limits suggests a relatively flat stratigraphy throughout the northern section of the viaduct and the pile test site. Other markers such as calcareous siltstone, thinly interbedded limestone and limestone bands present in AMC and WWB formations also support an assumption of horizontal stratigraphy or otherwise a slight dip towards the south, which would be expected following the general trend of the geology in Aylesbury Area.

Figure 3 presents undrained shear strength and Atterberg limits versus elevation while considering the site ground investigation data. Borehole ML065-CR532A has been drilled at the test site.

The undrained shear strength profile has been determined based on in-situ and laboratory tests.



- Where estimated from CPT, $c_u = (q_c - \sigma_v) / N_{kt}$ with N_{kt} assumed as 17.
- Where estimated from Standard Penetration Tests (SPT), $c_u = f_1 N_{60}$ as proposed in (Clayton, 1995) where $f_1 = 4.5$, based on plasticity index for most of the layers except WWB. For WWB f_1 has been taken as 6 for better fit with the triaxial (TRX) test data.
- Where estimated from Uniaxial Compressive Strength tests (UCS), undrained shear strength was taken as $c_u = UCS / 2$.
- The design profile of the test pile has been developed based on the characteristic undrained shear strength profile shown as continuous black line in Figure 3.

Table 2 summarises the stratigraphy and the key geotechnical parameters considered at the test site. AMC has been split into AMC I and AMC II to reflect a change in the plasticity index below 58m AOD (m Above Ordnance

Datum), with a slight adjustment in the undrained shear strength profile. The equivalent undrained shear strength of the OXC has been determined using the UCS values from laboratory tests of an extremely weak mudstone.

3. Viaduct and pile design

3.1 Description of the viaduct and the foundations

The viaduct carries HS2 over the Thame Valley flood plain, a relatively flat area of open fields surrounding the Thame River which runs approximately North East to South West across the proposed route.

Thame Valley Viaduct is a 36 span, 880m long continuous structure consisting of 4 No. 20 m long spans and 32 No. 25 m long spans. The piers are connected to 12x2.5x1.8 m deep pile caps which are in turn supported by a single

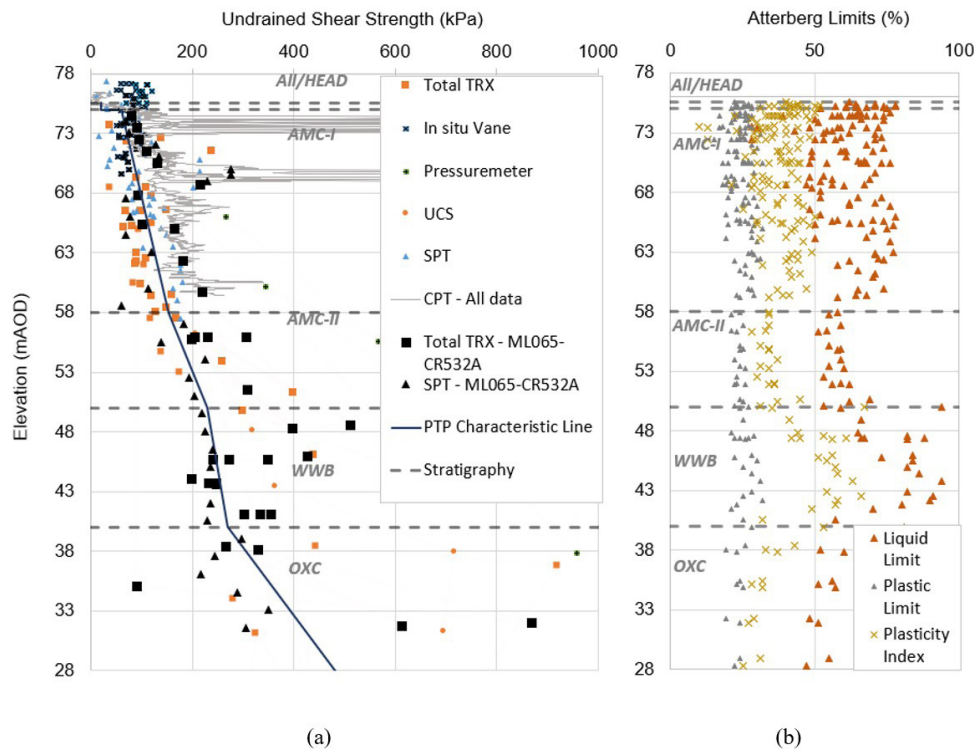


Figure 3. Undrained shear strength (a) and Atterberg limits (b) in the vicinity of the test site.

Table 2. Stratigraphy and Geotechnical Parameters assumed at the test site.

Geological unit	Top of layer (m AOD)	Thickness (m)	c_u (kPa)	UCS (MPa)
HEAD	75.6	0.6	20	-
AMC I	75	17	$60+5.5z$	-
AMC II	58	8	$154+9.6z$	-
WWB	50	10	$230+4z$	-
OXC	40	Not proven	-	$0.54+0.035z$

row of three 1500 mm diameter piles (same diameter as PTP10A) with a fixed pier providing longitudinal restraint. The fixed pier is connected to a 14x14x2 m deep pile cap, supported by nine 1800 mm diameter piles (same diameter as PTP11) in a 3x3 layout. The abutments are supported on six 1500 mm piles in a 3x2 layout. After completion of the test pile program, all working piles have been constructed as typical bored cast in-situ bored piles in the dry without the use of any support fluid.

3.2 Pile design

Pile design for the viaduct foundations has been carried out following the established standard practice in the UK. This corresponds to estimating unit shaft resistance in HEAD, AMC and WWB using the α design method, i.e. $f_s = \alpha c_u$ (Tomlinson & Woodward, 2008), where α is the empirical coefficient assumed as 0.5 and c_u is the undrained shear strength presented in Figure 3. A limit to $f_s = \alpha c_u$ of 125 kPa was applied to shaft resistance in clay. For the length of piles through OXC, the unit shaft resistance has been estimated using the approach proposed by Horvath & Kenney (1979), i.e. $f_s = 0.225\sqrt{UCS}$. A limit to f_s of 300 kPa has been applied to the design.

The unit base resistance in the OXC has been estimated as $f_b = 4.5 UCS$, (Rowe & Armitage, 1984) where UCS is taken at the pile toe level. Table 3 presents the estimated ultimate resistance of the two test piles.

4. Test piles

4.1 Description of tests

As shown in Figure 4, the instrumentation in the two piles included 4 pairs of extensometers and 9 sets of strain gauges. The displacements of the top plate, toe and pile head and expansion of the O-cell were also measured electronically using Linear Voltage Displacement Transducers (LVDT).

The 1500 mm test pile, PTP10A, was installed with length of 41.06 m, incorporating the O-cell assembly at 36.1 m below the pile head. The maximum sustained gross load applied to the pile was 10.75 MN in each direction. At the maximum sustained load, the displacements above and below the O-cell assembly were 6.01 mm and 7.97 mm, respectively. At the next load increment with a gross applied load of 11.70 MN in each direction, the upper pile section above the O-cell was unable to sustain the applied load, see Figure 5.

Table 3. Estimated Ultimate Resistance of the test piles.

Test Pile ID (diam.)	Pile length (m)	Shaft capacity (MN)	Base capacity (MN)	Total capacity (MN)
PTP10A (1.5m)	41.06	17.8	5.7	23.5
PTP11 (1.8m)	39.05	19.2	7.4	26.6

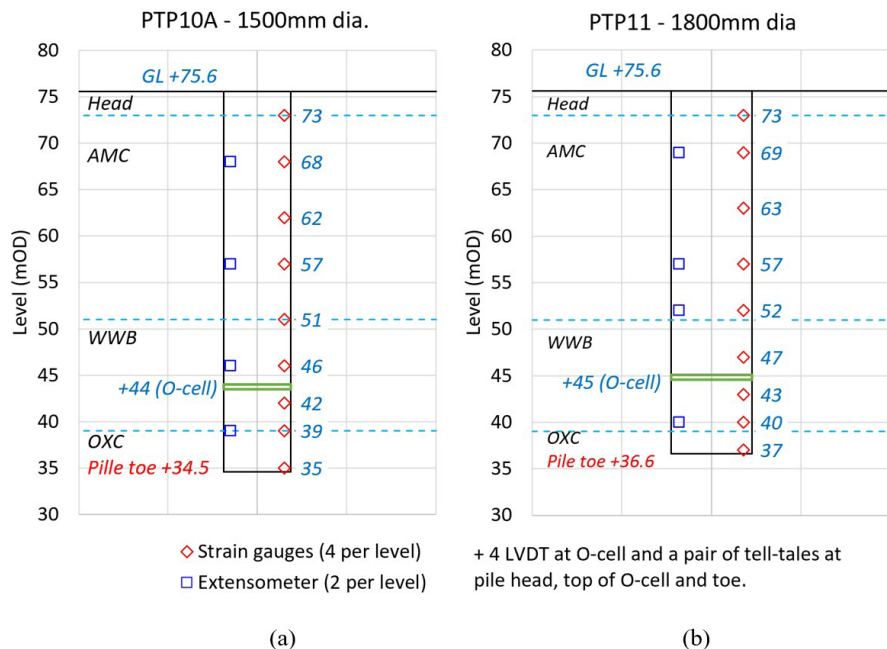


Figure 4. Schematic representation of the instrumentation in the test piles PTP10A (a) and PTP11 (b).

The 1800 mm test pile PTP11 was installed with length of 39.05 m, incorporating the O-cell assembly at 30.6 m below the pile head. The maximum sustained gross load applied to the pile was 13.32 MN in each direction. At this maximum sustained load, the displacements above and below the O-cell assembly were 4.97 mm and 10.13 mm, respectively. Again, the upper pile section was unable to sustain the applied load at the next load increment, see Figure 5.

4.2 Interpretation of the results

The load distribution along the pile shaft is shown in Figure 6 and has been calculated by multiplying the average strain at each level of strain gauges by the calculated axial

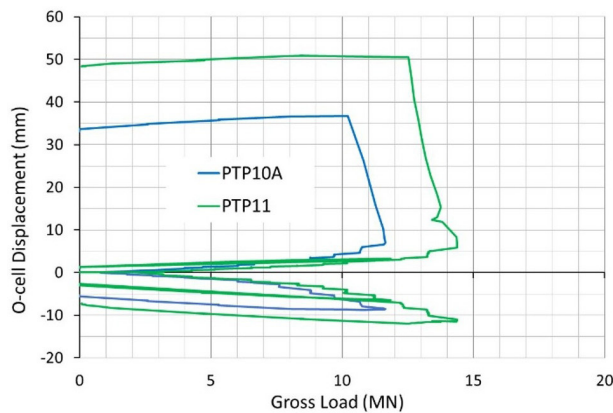


Figure 5. Upper and lower shaft movements.

stiffness ($EA=71000$ MN for PTP10A and $EA=97800$ MN for PTP 11). The strain gauges immediately above and below the O-cell (SG3 and SG4 in PTP10A, and SG3 in PTP11) recorded results that were inconsistent with typical load distribution patterns observed in test piles. These readings were excluded from the interpretation, as their close proximity to the O-cell may have caused anomalous strain measurements - likely due to high stress concentrations or reduced local concrete stiffness resulting from placement irregularities during tremie concreting around the O-cell assembly.

The interpreted unit shaft friction between levels of strain gauges is summarised in Table 4.

The equivalent top load-displacement has been analysed using two methods, the CEMSET Analysis method (Fleming, 1992) and the conventional Equivalent Top Load (ETL) method (England, 2008). The CEMSET method was used to determine the behaviour of the pile above and below the O-cell by considering the upward and downward movement analysed separately. Figure 7 presents the individual CEMSET curves that produce the best fit load-displacement profiles with the measured test data, using the input parameters in Table 5. Given that both piles failed upwards, the base capacity has not been fully mobilised, therefore this base capacity has been estimated using the approach presented in Section 3.2 but slightly adjusted to fit the data for the downward movement curves of the two test piles.

Figure 8 shows the equivalent top-loading curves for the two test piles determined by combining the upward and downward load-displacement using the CEMSET method (dashed lines). It also shows and the equivalent top-loading

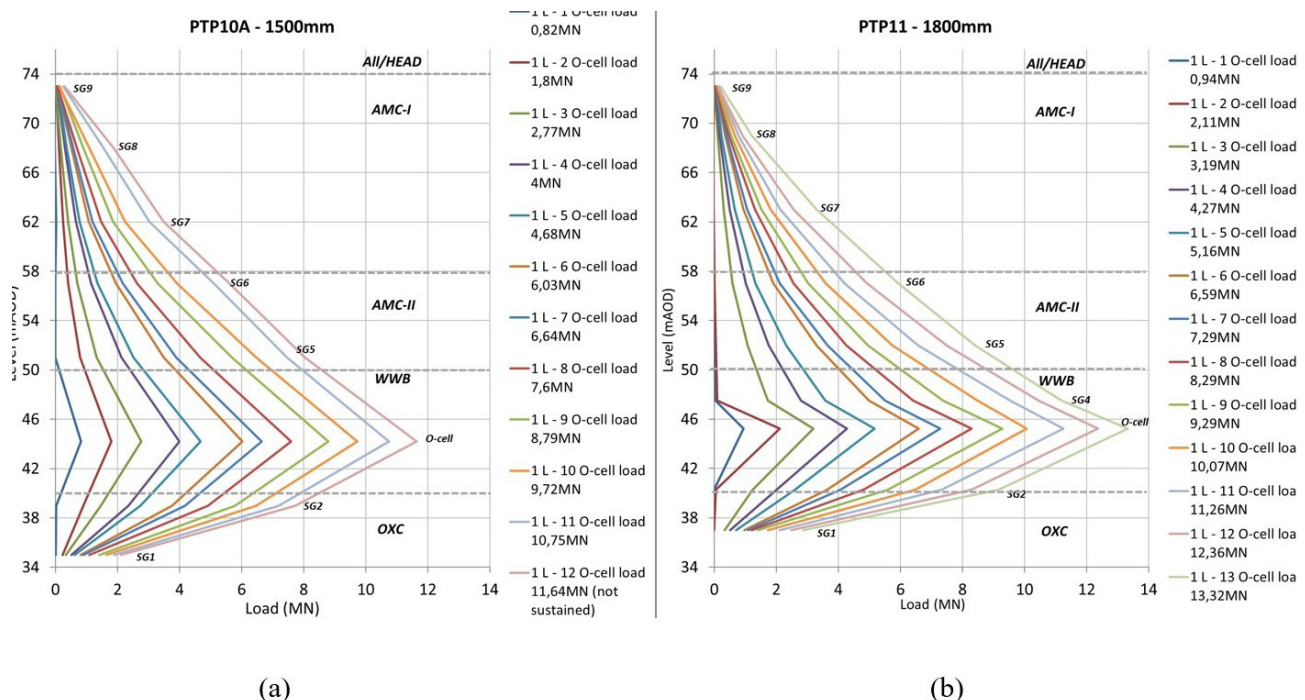


Figure 6. Axial load calculated along the test piles PTP10A (a) and PTP11 (b).

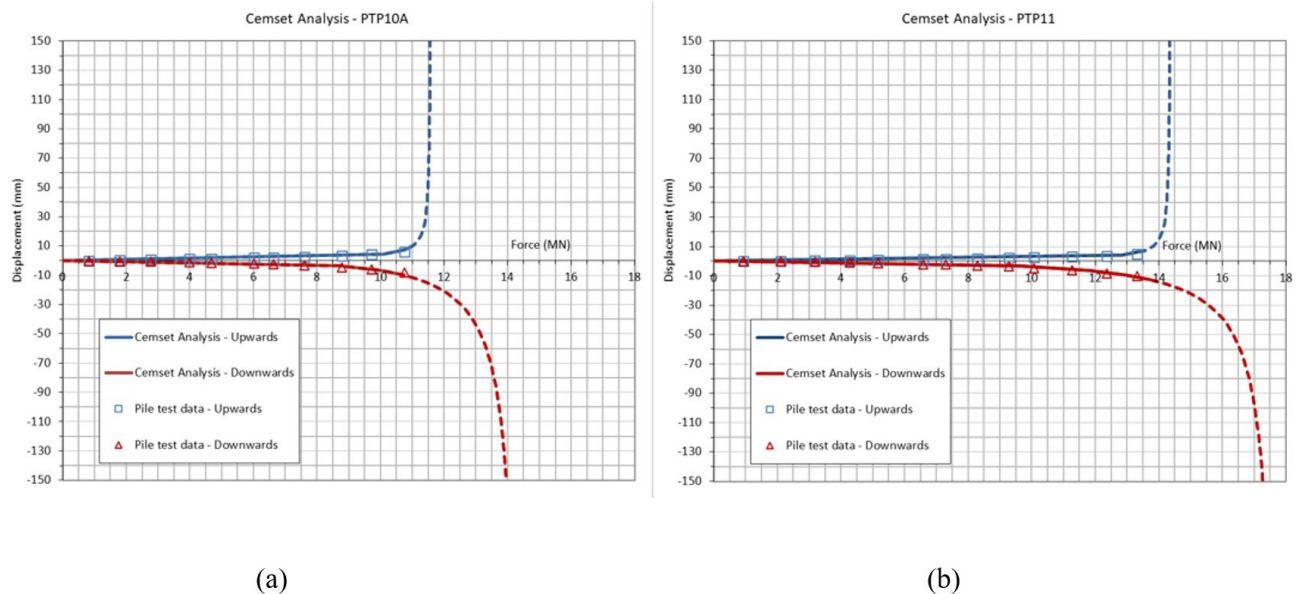
Table 4. Interpreted unit shaft friction.

Levels (m AOD)	Strata	Unit shaft friction (kPa)	
		PTP10A	PTP11
75.6 - 73	HEAD	20	13
73 - 68.5	AMC	56	45
68 - 62.5		52	61
62 - 57		89	78
57 - 51.5		83	88
51 - 46	WWB	103	128
45 - 40		145	149
39 - 35	OXC	281	338

Table 5. Summary of input parameters for CEMSET analysis above and below the O-cell.

Input Parameter	PTP10A		PTP11	
	Above O-cell	Below O-cell	Above O-cell	Below O-cell
Q_s^1 (MN)	11.6	9.47	14.38	11.32
M_s (-)	0.0003	0.0004	0.0002	0.0006
Q_b^2 (MN)	-	5.0	-	6.5
E_b (MPa)	-	129	-	188
L_f (m)	31.6	9.46	30.6	9.46
K_e (-)	0.3	0.5	0.3	0.5
E_c (GPa)	40	40	40	40

¹Ultimate shaft capacity. ²Ultimate base capacity.

**Figure 7.** CEMSET Analysis used to produce the best fit load-displacement profiles to the measured test pile data of PTP10A (a) and PTP11 (b).

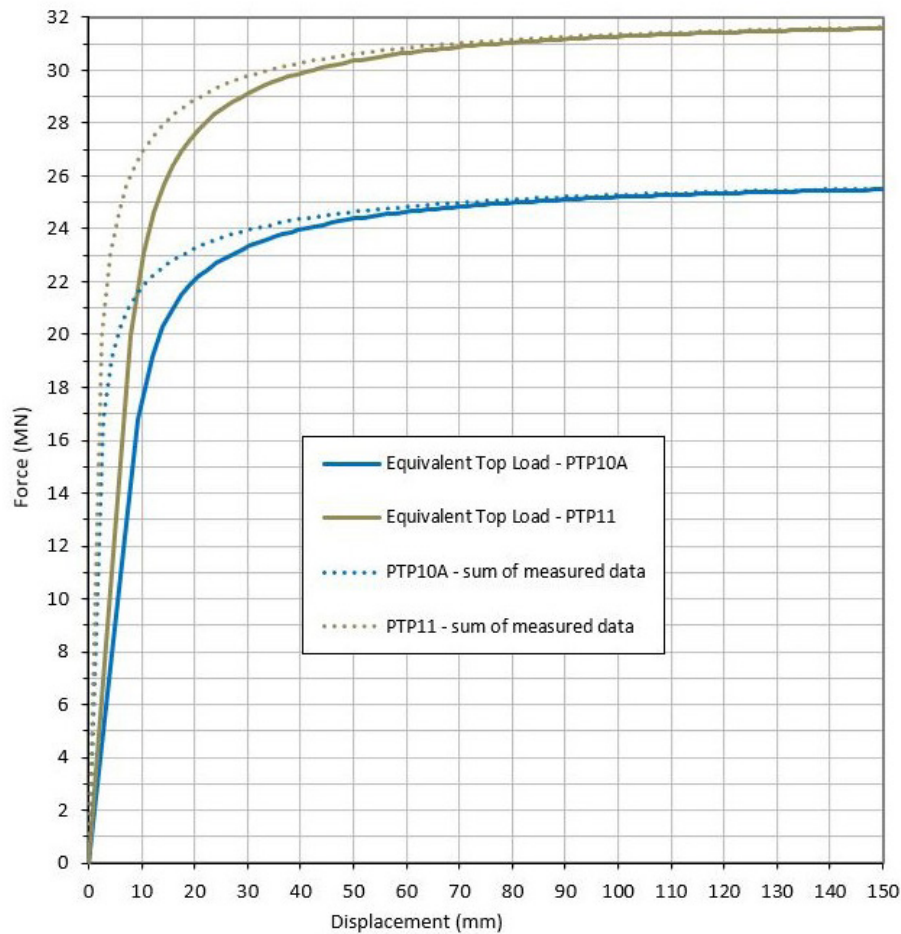


Figure 8. Equivalent top load displacement curves determined using CEMSET Analysis.

Table 6. Summary of estimated pile capacities.

Pile	Ultimate Capacity (MN)	
	Estimated in Section 3.2	CEMSET Analysis
PTP10A	23.5 ($Q_s = 17.8$; $Q_b = 5.7$)	26 ($Q_s = 21.0$; $Q_b = 5.0$)
PTP11	26.6 ($Q_s = 19.2$; $Q_b = 7.4$)	32 ($Q_s = 25.5$; $Q_b = 6.5$)

Q_s = Ultimate shaft capacity; Q_b = Ultimate base capacity.

curves corrected for the additional elastic shortening using the method proposed by England, 2009 (solid line).

Alternatively, the equivalent top-loading displacement curve has been approximated with the conventional ETL method. For this method, arbitrary displacement points were picked (1 mm, 2 mm, 3 mm, 4 mm, 5 mm, etc.) and the corresponding O-cell loads in the upwards and downwards curves have been measured at the selected displacement points. To take advantage of the fact that the test produced downward movement data, an extrapolation of the upward movement has been made for PTP10A and PTP11 using a suitable hyperbolic curve fitting the test data that corresponds to a high correlation coefficient ($R^2 > 0.99$). Figure 9 shows the ETL method (also corrected for the elastic shortening for

top loaded piles) compared with the CEMSET method. The figure shows a good agreement between the two methods up until the point when extrapolation is introduced, where ETL method predicts a much stiffer response suggesting that the extrapolation using the ETL method may be unconservative in predicting the ultimate capacity of the pile. Table 6 shows the comparison of ultimate capacities between the design approach presented in Section 3.2 and those calculated from the CEMSET analysis.

4.3 Comparison with viaduct design

Figure 10 shows a good agreement between the unit shaft friction estimated using the design profile and the values

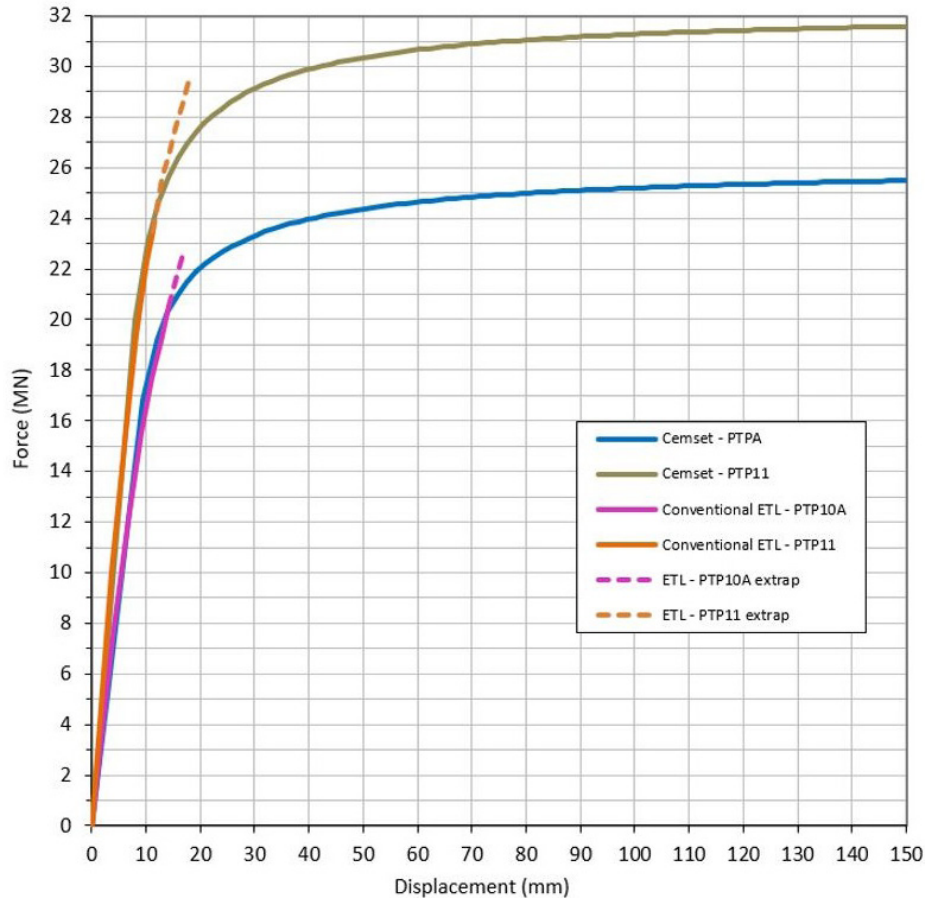


Figure 9. Comparison of equivalent top load displacement curves determined using CEMSET and ETL methods.

interpreted from the test pile data in most strata except the OXC where there is significant underprediction of the shaft resistance.

Particularly throughout AMC-I, the estimation of unit shaft friction based on design profile is slightly lower than interpretation of the test data, as expected given the slightly cautious undrained shear strength profile for the site-specific borehole data. There is no such trend for the AMC-II stratum except it is fair to assume the design values are in good agreement with the measured ones. The interpretation of the test data suggests constant unit skin friction between levels 52 and 62 m AOD, which is unlikely to be a correct representation since gradual increase in soil strength is expected. In WWB, the estimation based on design profile is slightly lower than interpretation of the test data for the bottom half of this unit, because a limit to $f_s = \alpha c_u$ of 125 kPa has been applied to shaft resistance in clay, otherwise a good agreement should be expected. It could be argued that such limit could be higher, for instance a limit to $f_s = \alpha c_u$ of 150 kPa could be recommended, however the current limit is only applicable to the unit shaft friction over a short length and therefore does

not affect the overall pile capacity. In OXC it becomes clear that the assumed unit shaft friction based on design profile underestimated actual shaft resistance. This is possibly due to the assumption of a conservative characteristic strength in the Mudstone layer that resulted from the wide scatter in the available ground investigation data. While this is not demonstrated by Figure 10, it is also possible that the base capacity in OXC is being underestimated if the characteristic base parameters are conservative. However, noting that both piles only penetrated a short length into OXC and that more data should be considered to confidently infer any conclusions, it is considered prudent to keep the current approach in the viaduct pile design.

4.4 Analysis using 3D finite element analysis

The preliminary test piles were back-analysed using Plaxis 3D finite element (FE) program to mimic the measured performance of the test results under the bi-directional O-cell test before the introduction of load at pile head to derive the equivalent top loaded pile response. Although an axisymmetric 2D model would suffice, the decision to adopt

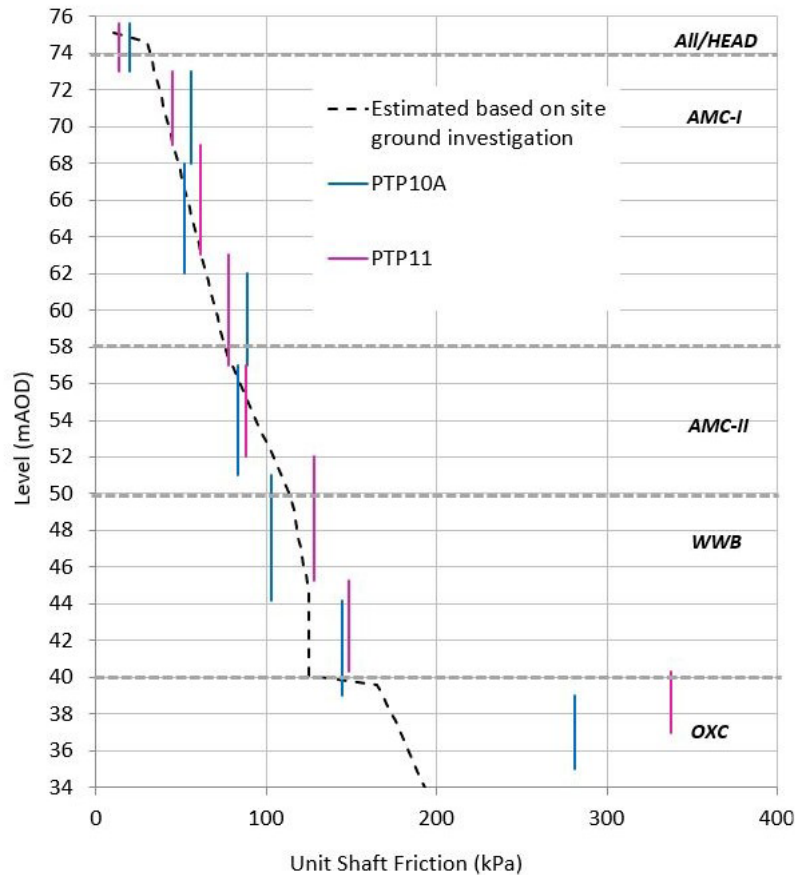


Figure 10. Comparison of unit shaft friction determined by preliminary test piles and design.

a full 3D model allows the assessment of lateral response of the pile be it as a single pile or as a pile group for the design of the viaduct foundations.

4.4.1 Methodology

In the FE model, the test pile was modelled by splitting the shaft into several sections to mimic the positions of the strain gauges installed to measure the load transfers during the test. Each of these shaft sections is then assigned a shear strength parameter to reflect the measured unit skin friction values, see Figure 11. Under the test load, the measured displacements of the O-cell's top and bottom plates, pile head and pile toe were compared and calibrated before the pile is loaded from the top.

It is worth noting that when assigning individual shear strength using interface element, only Plaxis command codes were used to change the input shear strength parameters rapidly for multiple assessments to calibrate the load-displacement of the test pile. The use of automated Python script would speed up this calibration process, but this is not used at this stage.

Apart from the shear strength parameters assigned to the test pile, soil properties of the ground and stiffness of the pile concrete are the other two variables which could

affect the behaviour of the pile under the applied load. For the calibration undertaken, the following are the soil model and properties used in the FE model:

- concrete stiffness: 40-45 GPa;
- stiffness of the soils: $E_u/c_u = 500-750$.

Linear elastic perfectly plastic Mohr-Coulomb (MC) soil model has been used to represent the soils. Although parameters for more complex or advanced soil models are currently not available in this type of soils, an attempt is also made to calibrate one of the test pile using Small Strain Hardening Soil model (HSS) in Plaxis with a view to understand the significance of varying the type of soil model on the result of the back analysis. The parameters of the HSS have been based on the back analyses work undertaken for retaining walls in London Clay (Yeow, 2022) where the derived HSS parameters were found to be similar to the retaining wall back analyses with a E_u/c_u ratio of approximately 1000 (Yeow, 2022). Although it is well understood the vertical stiffness of sedimentary deposits such as London Clay is usually lower than lateral stiffness (Gasparre, 2005), the same set of retaining walls back analysed parameters are first used to determine the behaviour of the 1500 mm diameter test pile before a more rigorous calibration is undertaken.

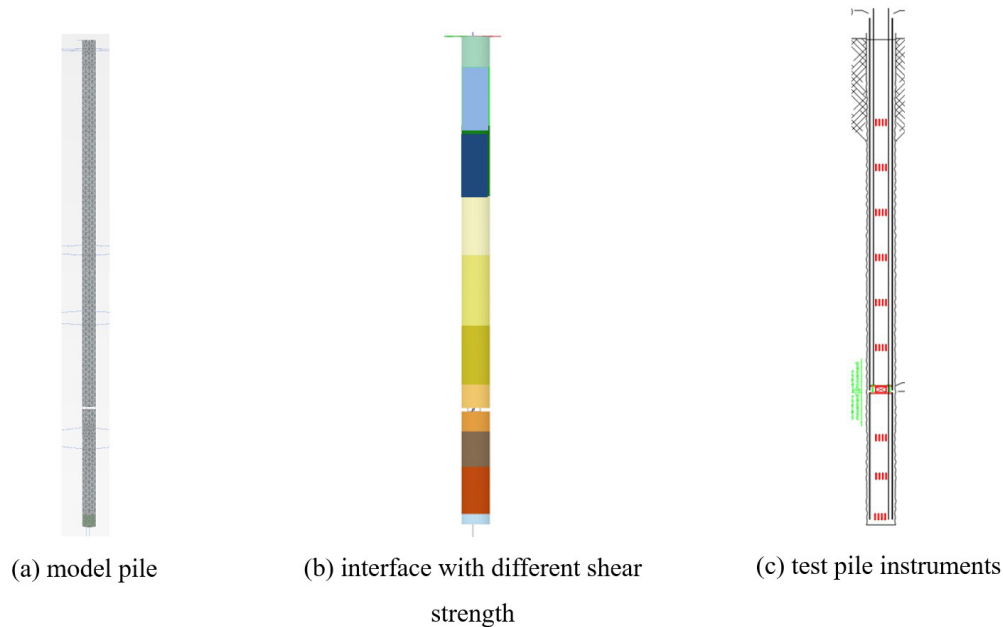


Figure 11. Pile shaft with different shear strength values.

4.4.2 Test pile calibration using Mohr Coulomb soil model

The process of calibration started with the first set of parameters based on the pile design approach indicated in Section 3.2 to derive the unit shaft friction of the pile. Due to the low unit shaft friction in the OXC, the lower section of the pile settled excessively beyond a test load of 8MN for PTP10A. Similar observation for PTP11 albeit with slightly higher applied O-cell load.

With the derived unit shaft resistance from the test piles, the load displacement profiles for both the test piles are shown in Figure 12.

When load is applied from pile head, the equivalent load-displacement profiles are shown in Figure 13, including the range of response computed using MC soil model with different soil stiffness values. When comparing this to the CEMSET back-analysis, the difference in load-settlement performance of the piles at serviceability limit state is negligible although the ultimate capacity of the larger 1.8 m diameter test pile is approximately 5% lower. This could be due to more pronounced pile base softening effects on the larger test pile.

4.4.3 Further assessments using HSS soil model

In order to use the HSS model, the first task is to ensure the inherent undrained shear strength of the soil is comparable. Figure 14 shows comparison of the equivalent undrained shear strength of the soil derived using the retaining wall's HSS model with the characteristic profile presented in Figure 3.

With higher stiffness values based on the retaining wall parameters, the computed O-cell equivalent load displacement behaviour is presented in Figure 15. With this set of parameters, the upwards displacement of the test pile is showing stiffer response while the downwards displacement matches the measurement well.

It is also a rule of thumb that E_u/c_u ratio of 500 (Katsigiannis, 2017) is used for displacement assessment of London Clay under vertical load. When the stiffness values in the HSS model is reduced to 50% of the values used for retaining walls, the following load displacement behaviour is computed and shown in Figure 16. With this set of stiffness parameters, both the computed upwards and downwards displacements show reasonable match with the measured values.

From the above comparisons using both the MC and HSS soil models, it is clear that as long as the strength parameters are correctly assigned, the ultimate capacity of the pile could be reasonably predicted. In the UK, piles are designed with an overall factor of safety of approximately 2.8. For PTP10A, the working load would be approximately 8.5 MN. The differences in stiffness values would affect the upwards displacement of the O-cell more than the downwards displacement. Such differences in stiffness would not affect the serviceability limit performance of the pile which is generally related to load-displacement up to 1.5 times the working load, or less than 13 MN for PTP10A, before the displacement exceeds the non-linear region or elasto-plastic behaviour of the pile. In this case the elasto-plastic behaviour initiates when the equivalent top load of greater than 16 MN.

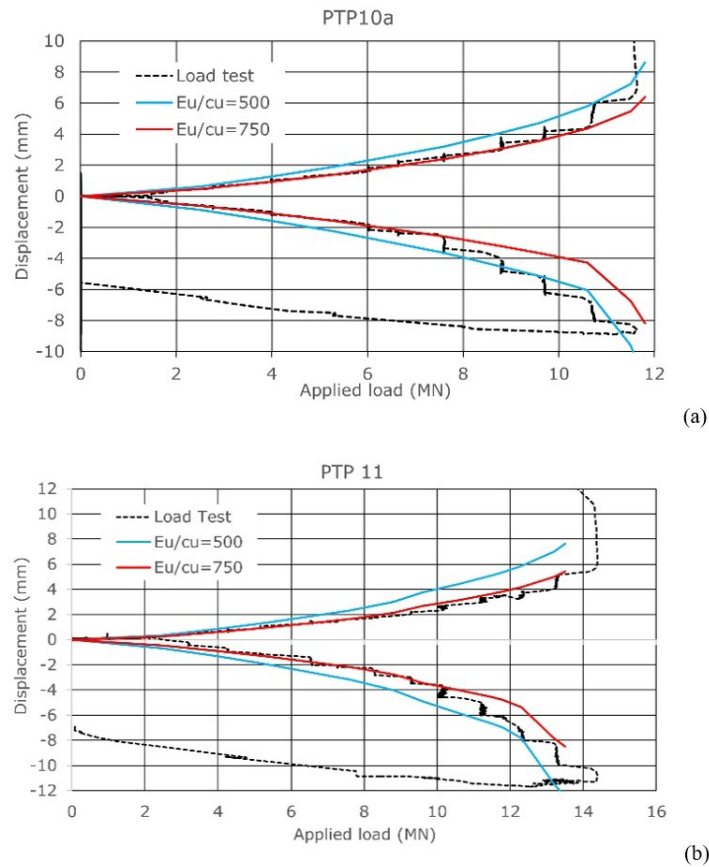


Figure 12. Load-displacement profiles for the test piles PTP10A (a) and PTP11 (b) back-analysed using Plaxis 3D model.

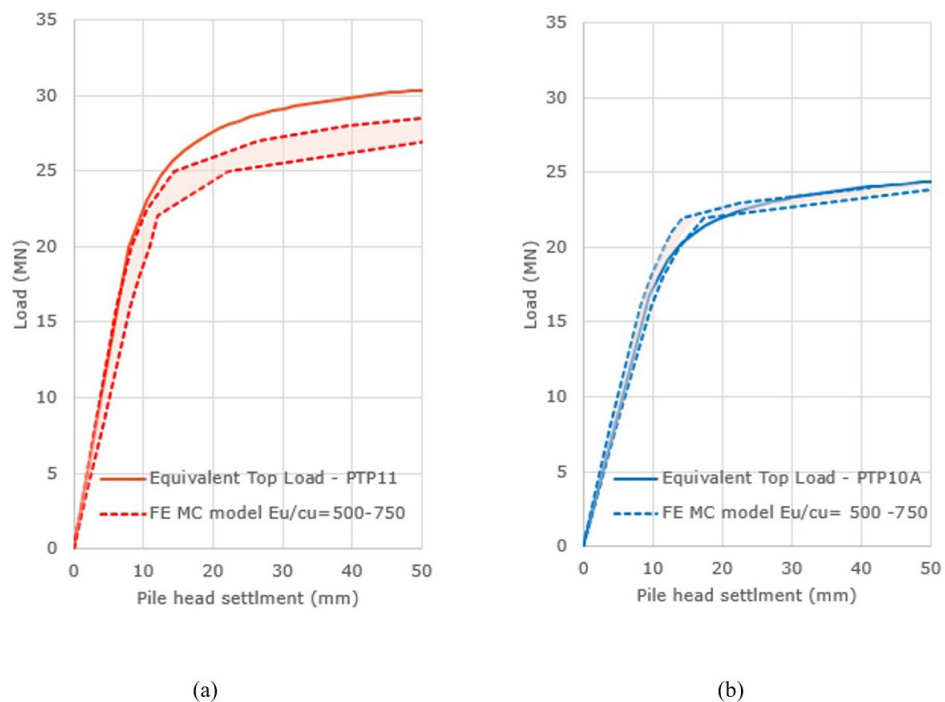


Figure 13. Range of load-displacement profiles for the test piles PTP10A (a) and PTP11 (b) with different E_u/c_u ratios.

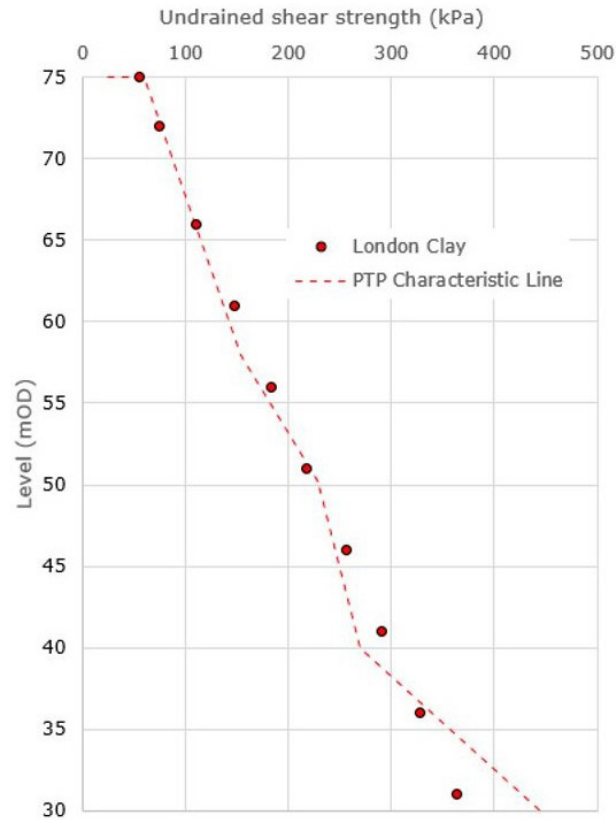
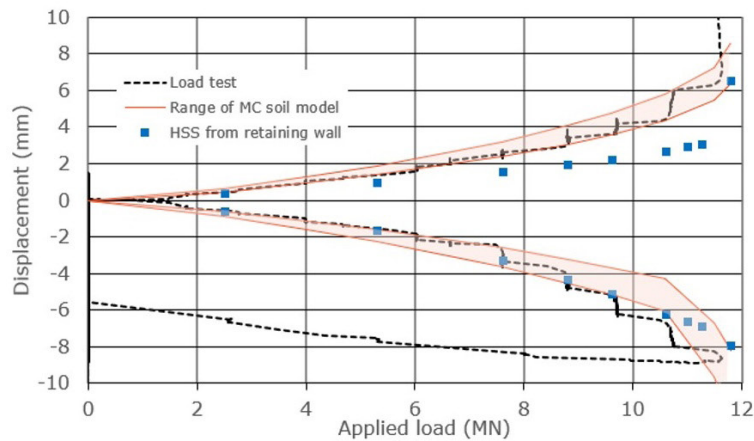
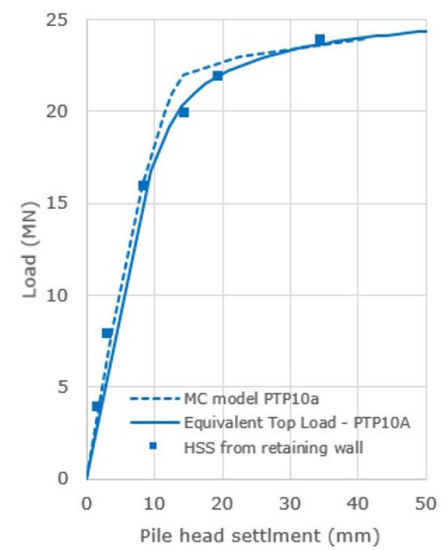


Figure 14. Equivalent undrained shear strength of the HSS model compared to PTP characteristic profile.



(a)



(b)

Figure 15. Computed load displacement of PTP10A using retaining wall HSS model. (a) applied load versus upwards and downwards displacement; (b) Pile head settlement versus load.

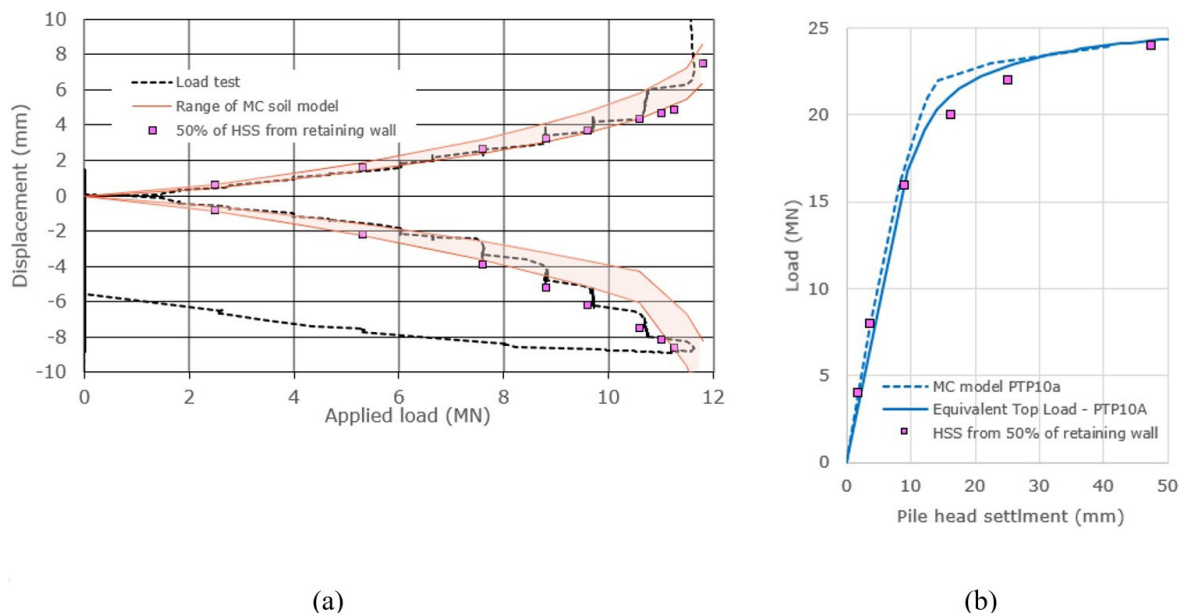


Figure 16. Computed load displacement of PTP10A using 50% of stiffness of retaining wall HSS model. (a) applied load versus upwards and downwards displacement; (b) pile head settlement versus load.

5. Conclusions

The ultimate pile capacities estimated using the approach assumed by the designer outlined in Section 3.2 were calculated as 23.5 MN and 26.6 MN for test piles PTP10A and PTP11, respectively. In this approach the unit shaft resistance in HEAD, AMC and WWB was calculated as $f_s = 0.5c_u$, while through OXC, calculated as $f_s = 0.225\sqrt{UCS}$. The unit base resistance has been estimated as $f_b = 4.5UCS$, although base capacity has not been fully mobilised. This methodology was validated through the interpretation of pile load test results, confirming it as the recommended approach for the succession of clay and mudstone formations within the Ancholme Group.

When this approach is compared with the back-analysed unit shaft frictions from the test pile data, there was a good agreement in the clay layers but it underestimated the shaft resistance in the Mudstone layers. It is possible that the assumption of characteristic strength of the Mudstone layer is conservative. More ground investigation data and pile test in these strata would be required to verify a more appropriate strength parameters in order to incorporate higher shaft resistance values.

Due to the short length of pile in the OXC, where the most discrepancy in the shaft resistance value was found, the design approach presented for pile design in AMC/WWB/OXC is considered appropriate and slightly conservative for pile lengths between 39 and 41 m.

The back-analysis of the two preliminary test piles in Plaxis 3D finite element models using linear elastic perfectly plastic Mohr-Coulomb and Small Strain Hardening Soil models

has confirmed the load-settlement characteristic necessary to verify the performance of the pile under serviceability limit state, which is generally related to load-displacement up to 1.5 times the working load. The assessment has also shown that differences in stiffness would not affect the serviceability limit performance of the piles and that, as long as the strength parameters are correctly assigned, the ultimate capacity of the piles could be reasonably predicted.

The back-analysis presented in this paper demonstrates that standard UK pile design practices are adequate for the ground conditions encountered (Ancholm Group). This outcome reflects a successful collaboration between design teams and contractors to outline a scope of test piles, enabling an efficient and verified design solution for the 880m-long Thame Valley Viaduct.

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Declaration of interest

The authors have no conflicts of interest to declare. All co-authors have observed and affirmed the contents of the paper and there is no financial interest to report.

Authors' contributions

Sergio Vitorino: conceptualization, data curation, formal analysis, investigation, methodology, project administration, resources, software, supervision, validation, visualization, writing – original draft. writing – review & editing. Hoe-Chian Yeow: conceptualization, data curation, formal analysis, investigation, methodology, resources, software, supervision, validation, visualization, writing – original draft. writing – review & editing.

Data availability

The datasets generated analyzed in the course of the current study are available from the corresponding author upon request and pending the necessary approval.

Declaration of use of generative artificial intelligence

This work was not prepared with any assistance of generative artificial intelligence (GenAI).

List of symbols and abbreviations

c_u	undrained shear strength
f_l	CPT and SPT correlation constant
f_s	unit shaft friction in kPa
m AOD	metre Above Ordnance Datum
q_c	cone resistance
z	depth below top of stratum
E_b	soil modulus as base
E_c	concrete Young's Modulus
K_e	CEMSET parameter: centroid of friction transfer
L_f	CEMSET parameter: length of pile
M_s	CEMSET parameter: shaft flexibility factor
N_{kt}	cone correlation constant
Q_b	base resistance
Q_s	shaft resistance
UCS	unconfined compressive strength

α	empirical correlation constant used for pile design in cohesive soils
σ_v	vertical stress

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CASE STUDIES

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New electric powered drill rig to meet sustainable geotechnics

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Case Study

Keywords

Measuring while drilling
Electrical drill rig
Sustainable development

Abstract

Drilling boreholes for geotechnical purposes such as sampling, in-situ tests and permeability testing are usually performed with the aid of drilling machines. These heavy duty machines generally use internal combustion motors to provide energy to hydraulic pumps feeding the different actuators needed to push, rotate and hammer the drill bit into the ground. This results in a relatively noisy and polluted environment for the workers and the surrounding area. With the popularization of electric vehicles, it's time drilling machines also make the switch to electric power. This paper presents one such prototype machine developed by Fondasol company, with the initially proposed technical specifications and the realized equipment. Autonomy for up to one week has been observed on real sites, depending on the project's drilling objective. The use of electric motors allows for a new type of measuring while drilling, as the measurement of electric current and voltage is much more reliable than hydraulic pressure and flow rates. With this electric-based design, additional external sensors are no longer necessary. A first insight into this new type of measurements and its physical meaning is also given and discussed within this paper.

1. Introduction

Historically used in the oil and gas industry but gaining popularity in the field of geotechnical engineering in the last decades, the method of measuring while drilling (MWD) involves monitoring and recording drilling parameters during the drilling process (Rodgers et al., 2020). In this method, a drill bit destroys the terrain as it advances downward, pushed and spun by hydraulic motors while a fluid cools down the equipment and removes loose debris from the hole, with the eventual use of a hydraulic hammer to break down tougher material (Reiffsteck, 2010). Typically, the parameters monitored are the penetration rate, downthrust, torque, rotation speed and fluid injection pressure; while other data can also be collected such as fluid injection rate, drill string vibration, both the frequency and energy of the percussion blows and the energy reflected back after each blow (Moussouteguy, 2002).

The drilling may have multiple objectives: creating holes for sampling, in-situ tests like pressuremeter tests, installing piezometers and inclinometers, verifying the efficacy of soil treatments or simply gaining knowledge about the subsoil. It has the advantage of being a rapid essay giving

continuous information, much like the cone penetration test, while allowing an identification of the soil (Fortunati & Pellegrino, 1998; Cardu et al., 2013).

The drill rigs used in creating these holes may vary in size and overall power but share similar architecture: they all have a mast on which the drilling head moves vertically. The length of the mast and the characteristics of the drilling head may differ according to the project's objectives (Reiffsteck, 2010).

In addition to the translation motor, the drilling head is equipped with a rotation motor and eventually a hammer that transmits rotating forces to the drill bit underground through the drill string. Both elements are usually hydraulic. The mast has the functions of guiding and driving the drilling head vertically, pushing the drill bit into the soil. This movement is also achieved by a hydraulic system (Reiffsteck, 2010). Other special mechanisms may be attached to either the mast or the drilling head to increase the versatility of the machine by allowing it to perform SPT investigations or to collect undisturbed samples. Figure 1 shows a simplified scheme of a drill rig.

There are other hydraulic systems in the rig as well, such as the stabilizing cylinders, the mud pump, the winch,

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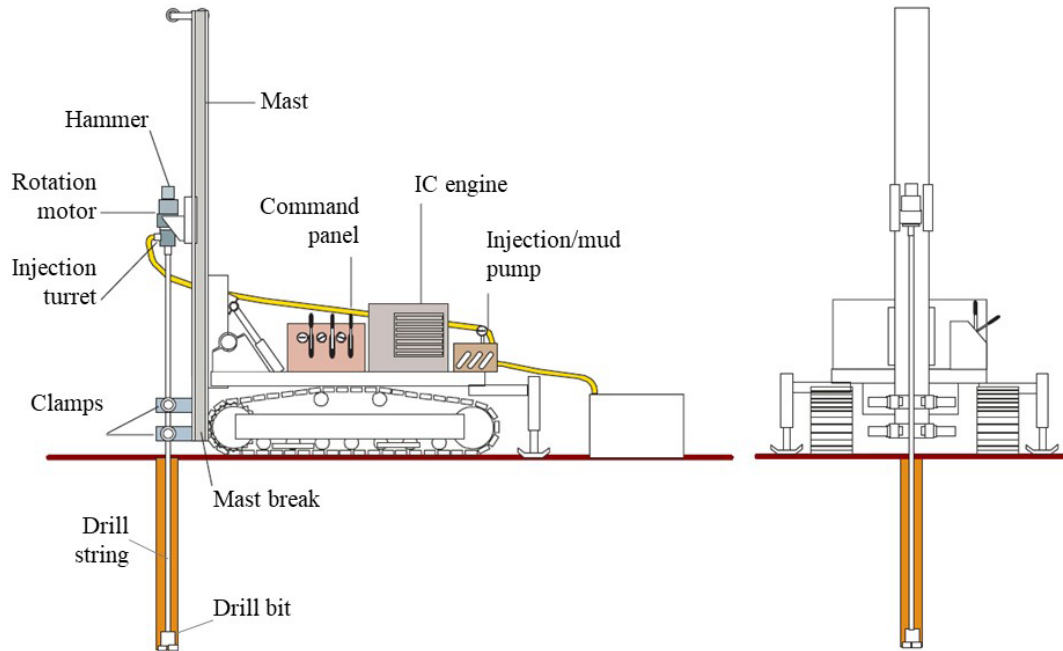


Figure 1. Diagram of a hydraulic drill rig.

the clamps and the cylinder that folds down the mast to put the machine in displacement mode. All these systems, as well as the pump for drilling fluid, are typically powered by a hydraulic pump connected to an internal combustion (IC) engine (Nishi et al., 1998; Reiffsteck, 2010).

According to Reiffsteck (2010), one of the risks associated with this method is the accidental release of hydrocarbons (oils or fuel) into the soil from the various hydraulic systems or the engine, which could contaminate the water table. The combustion motor also produces polluting gases and a lot of noise which can be detrimental to the workers' health and the surrounding area, even more so when used in closed environments.

To remedy these issues and improve on the method's energy efficiency, this paper presents the design and construction of one such machine. The use of electric motors also allows a different way of measuring drilling parameters, which is also discussed in the article.

2. Objectives of the project

The new CLEA (connected, light, electric and autonomous) drill rig proposes to change soil investigations by using electric motors instead of mechanic/hydraulic actuators fed by an internal combustion engine. The new machine retains the same architecture and usability as the currently used rigs, with a foldable mast and mounted on a crawler while being mechanically simpler as there is no clutch or gearbox.

This allows more efficient drilling, as electric motors are more energy efficient than hydraulic systems, while producing less noise and air pollution. The deployment of one such machine is estimated to avoid the emission of

8.2 $t_{eq}CO_2$ / year, which would greatly reduce the carbon footprint of geotechnical investigation companies. The drilling parameters can also be measured more reliably, as it is easier to continuously monitor electrical currents and voltages than fluid pressure or flow rate. A digital model and the realized prototype are shown in Figure 2.

3. Drill rig specifications

The proof of concept can achieve vertical speed of 900 m/h or 1500 mm/min with a maximum force of 21 kN. This movement is powered by an electric motor. The electric rotation motor allows a rotation speed of 350 rpm and a torque of 1500 Nm. These specifications, shown in Table 1, were chosen to approximate those of a conventional 32 kW hydraulic drill rig: vertical speed of 900 m/h, down force of 20 kN, rotation speed of 300 rpm and 1500 Nm of torque.

The second prototype was built to have the same specifications as the proof of concept. The translation and rotation motors were kept the same while the battery was

Table 1. Drill rig specifications.

	Conventional machine	Electric prototype
Translation speed (m/h)	900	900
Rotation speed (rpm)	300	350
Torque (Nm)	1500	1500
Vertical force (kN)	200	210
Mast length (m)	2,5	2,2

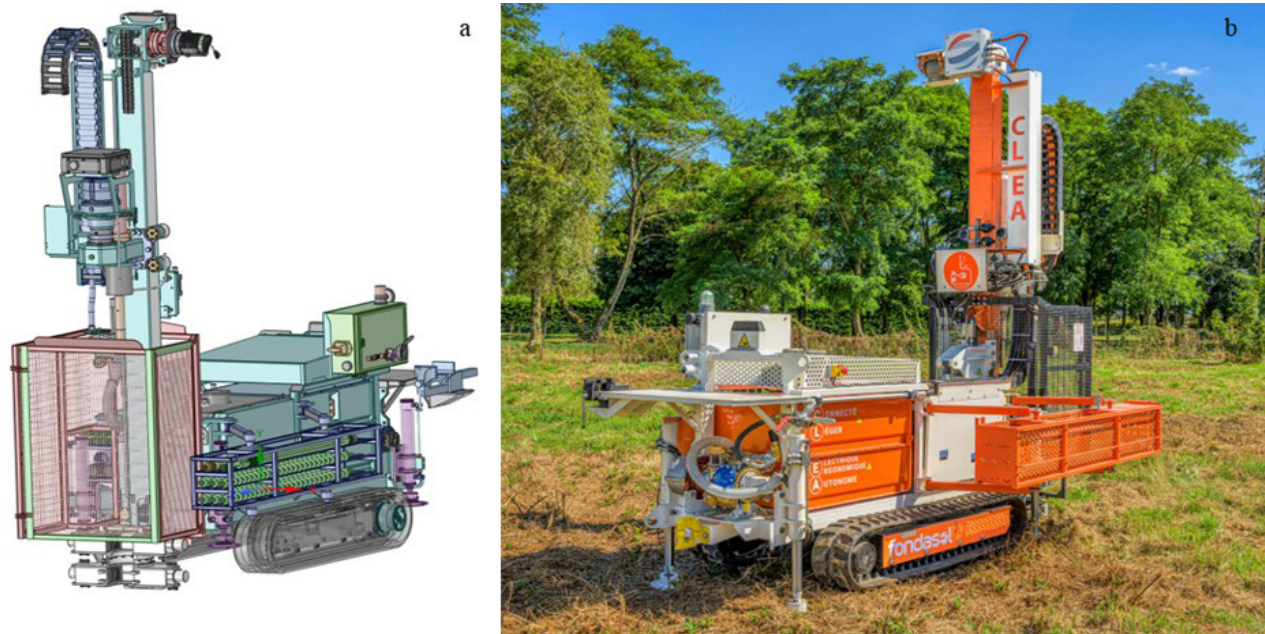


Figure 2. Electric drill rig: a) project prototype (a); finished machine (b).

now incorporated into the rig's chassis, occupying the space typically reserved for the IC engine. For this model the battery capacity was reduced, with the option of expanding it if necessary. With this new battery and the new electrical systems incorporated, autonomy fell to about 2 days (a bit over 40% of battery usage a day) during winter, when the lower temperature reduces battery life. Figure 3 illustrates all the motors and sensors installed on the proof of concept.

4. Project development

At first, a proof of concept was built over the chassis of a small 50 hp hydraulic drill rig, with the hydraulic mechanisms responsible for translating and rotating the drill string being replaced by electrical motors. The percussive hammer atop the mast and the vices used to separate the individual shafts from the drill string were still hydraulic pistons, but an electrical pump fed these systems alternatively, allowing the use of either one at a time. Meanwhile, the IC engine was still responsible for the rig's locomotion and set up.

For this first model, the battery bank used to power the various motors was kept separate in a detachable trailer as the IC engine remained in the chassis. This would also facilitate the adjustment of the battery capacity, in the case that the initial estimates on consumption were too far off. This initial estimate was made based on the fuel consumption of a traditional drill rig, aiming to give the prototype a similar performance.

As the controls on the machine were part of the replaced hydraulic systems, a new control pad was developed. Given that the new controls only send electronic signals to the motors instead of directly adjusting the flow and pressure of the hydraulic fluid, this new control pad could be detached from

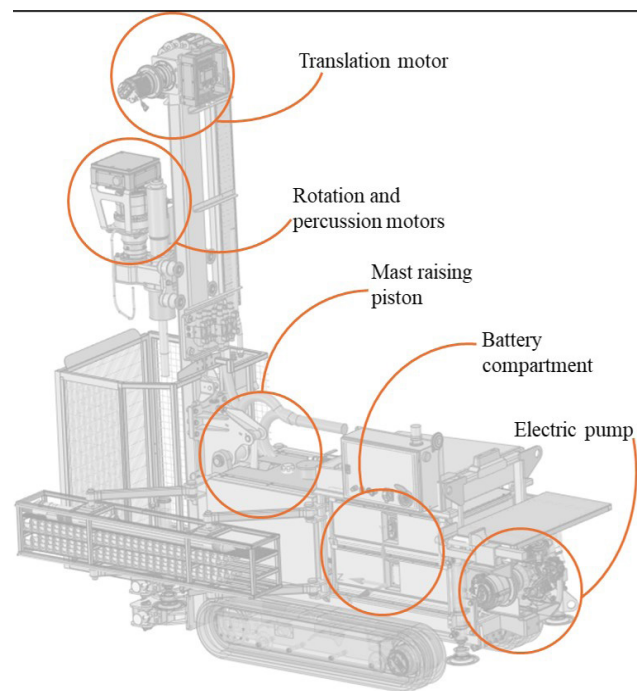


Figure 3. Scheme of all electric motors in the prototype.

the machine and carried by the operator. Detaching the pad from the machine has the advantage of allowing the operators to position themselves during the operation to better observe the drilling. It also gives the team more flexibility to operate in cramped spaces. However, as the locomotion systems were still conventionally powered, the hydraulic levers remained on the machine and the remote-control pad was only used during the drilling phase.

After careful calibration, this proof of concept was allocated to one of Fondasol's investigation teams and used in various real work sites in northwestern France, mostly in the region of Pays de la Loire. During this period, it was observed that the machine met expectations and even exceeded some: the drillers approved the ergonomic changes to the controllers and the much more silent workplace, and the energy consumption was below expected with an average of 12% of battery usage per day. One investigation made with this machine is presented in item 6 of this paper.

After the successful deployment of the proof of concept, development began on the second prototype. This second model has a fully custom chassis to house the batteries and no IC engine. Just like in the first model, an electrical pump powers hydraulic circuits for either the hammer or the clamps, being activated only when necessary. Unlike in the proof of concept, the locomotion and stabilization systems are now also hydraulic systems connected to electrical pumps.

The ergonomics of the rig were also improved in the new version: as the control pad had been detached from the machine, this allowed the installation of telescoping racks on both sides of the rig. These racks can be extended around the security cage during the drilling operation and make the various tools and drill shafts more easily available to the assistant driller.

The control pad also got an update and now is also responsible for logging in all measurements made during the entire drilling operation without the need for an external data logger, as is usual in drilling machines. This data is visible through a touchscreen in the pad, where the operator can select the project, work site and investigation point being drilled, and any other additional information they might deem necessary. This data can then be transmitted to the company's internal database via cellular network as soon as the drilling is marked as finished or manually transferred with a USB flash drive. As the portable control pad is also a prototype, its screen can be mirrored to quickly troubleshoot with the engineer even if they are offsite.

Included in this new control pad were also a few automations for repetitive tasks, accessible by pressing specific button combinations. These automations include using the clamps to unscrew the drill string and automatically adjusting the advance rate according to the rotation speed when drilling with continuous flight augers.

In the same manner as the proof of concept, the prototype was allocated to an investigation team and successfully used in real work sites in Northwestern France. One of the investigations made with the prototype is presented in item 7.

5. Electrical MWD measurement

The use of electric motors allows the monitoring of drilling parameters through the current and voltage. These electrical parameters are directly proportional to torque and rotation speed, respectively. This advantageously replaces the pressure transducers and the optical/magnetic sensors used to measure rotation speed typically used in conventional hydraulic rigs that only indirectly measure each parameter. The pressure transducers also have the disadvantage of not measuring the forces truly delivered to the

drill bit as they are placed beside the pumps and cannot account for pressure losses in the system (Peronne et al., 2021).

Unlike the typical measurements, the CLEA directly measures the current delivered to each motor, which is directly proportional to the torque applied on the drill bit. Instead of logging the values measured as a function of depth (typically each 2 cm), the proof of concept registered all the parameters as a function of time resulting in a much more detailed dataset. The newer prototype automatically displays this data as a function of depth, but the measurements are still logged every 0.4 seconds, and this more detailed dataset can still be accessed if necessary to evaluate the machine's performance.

The collected data can be displayed both as the conventional physical parameters in their respective units (MPa for downthrust and injection, Nm for torque, rpm for rotation speed) of measurement and in terms of energy spent or power in kW delivered to the motors. Calculating the power spent by each motor also has the advantage of providing the total energy spent during the survey, which can be used to confirm the battery indicator and visualize which motor spent the most energy at a given moment.

In the multiple investigations made with both rigs, it was seen that the rotation and percussion motors consistently spend the most energy while the energy used to translate the rotary table is almost negligible. The energy consumption on the injection pump varies greatly with the nature of the terrain, as to be expected, but also tends to be much lower than rotation and percussion.

6. Case study – worksite A

In the end of 2023, the proof of concept was deployed in the Loire Atlantique department of France, stationed in the city of Nantes. It took part in one project in the coastal city of La Baule and the drilling logs are shown in Figure 4 below. For this investigation, the drilling parameters are displayed in terms of electrical power consumed by each of the motors in the rig.

Figure 4 shows one of the drillings conducted for this project along with the lithology estimated by the driller, which consisted primarily of sandy layers, with a layer of soft clay and a bedrock of gneiss as well. The transition from the soft clay into the sandy clay layer can be easily identified at about 4 m by a drop in the advance rate and translation power as the rotation power rises. A slight increase in the injection power can also be seen. Finally, the bedrock can be quickly spotted by the need to activate the percussion at 14 m, but also by the decrease of the advance rate and the increase of rotation power.

From this investigation, we can see that the drilling logs from the electric machine can be interpreted much like those from a conventional hydraulic machine. Furthermore, monitoring electrical currents and voltages is easier and more precise than monitoring fluid pressure in a hydraulic system, making these logs potentially more precise than what could be acquired with a conventional machine. This happens because the points of measurement for the pressure in the hydraulic circuits are far from the motors and cylinders due to constraint in the machine's architecture which lead to energy losses not being accounted for.

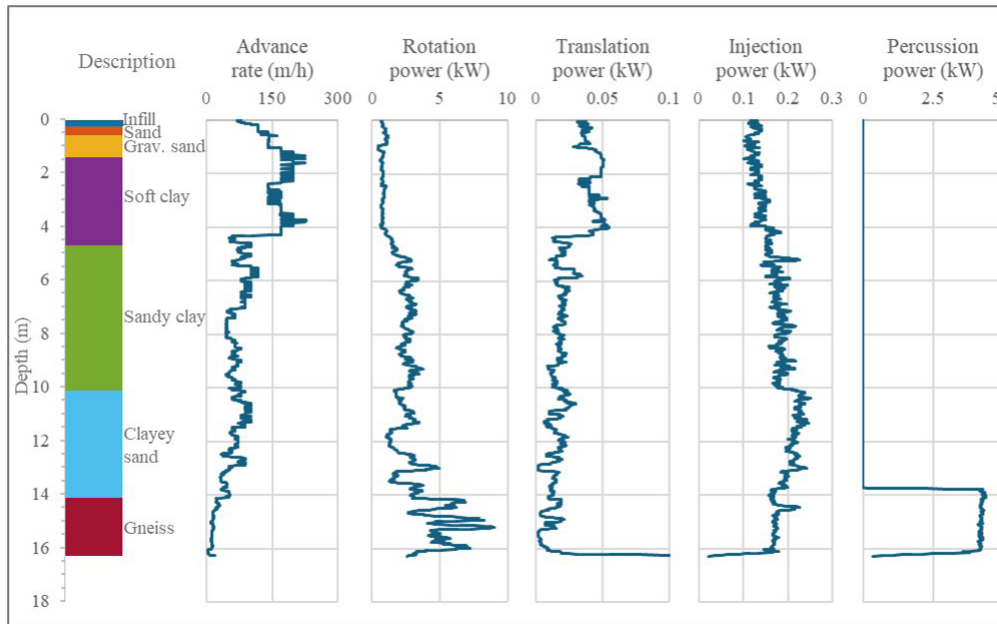


Figure 4. Drillings logs from an investigation campaign with the proof from concept.

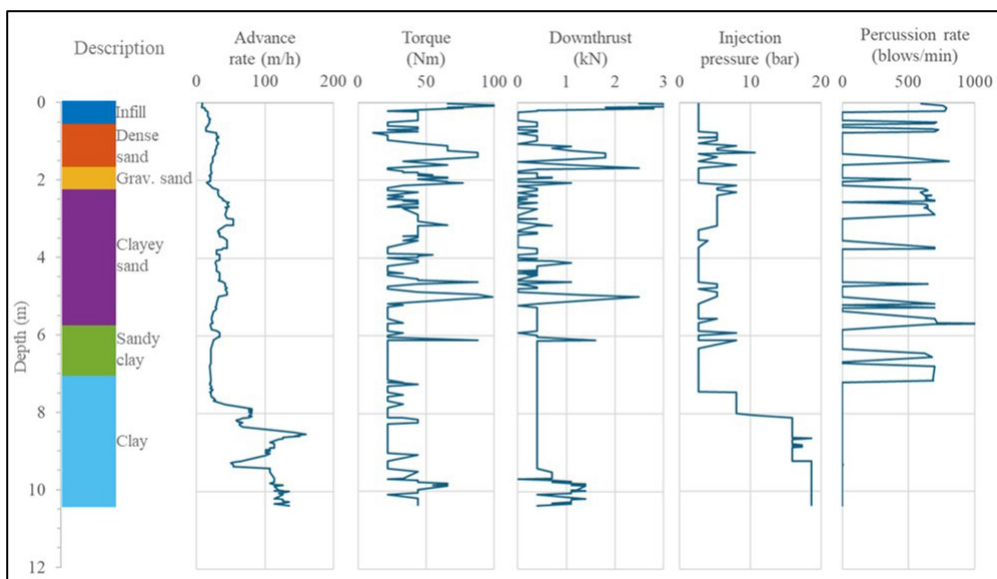


Figure 5. Drillings logs from an investigation campaign with the prototype.

7. Case study – worksite B

In 2024, the prototype was transferred to the department of Sarthe, stationed in the city of Le Mans, so that the machine could be tested in different types of terrain. It took part in one campaign in an industrial zone at the southern end of Le Mans, near the banks of the river Sarthe. The logs for one of the drillings from this campaign can be seen in Figure 5. For this project, the drilling parameters are displayed as the forces delivered to the drill bit, not as the power consumed by the electrical motors as in the previous example.

The lithology encountered consisted of sandy layers on top of a clayey layer. The transition between these layers still has a large quantity of sand mixed in and thus the drilling parameters show little change, but the rise in injection pressure after 7.5 m clearly indicates a rise in clay content. In this drilling the hammer had to be used intermittently because of the high lateral earth pressure that would frequently block the drill string, even though it wasn't a particularly hard soil. Due to this, the downthrust necessary to advance was greatly reduced and at some points only the weight of the assembly and the percussion was enough to make the drill bit advance at the desired speed.

8. Conclusion

This paper has presented the architecture of a new generation of drill rig that uses electrical motors instead of hydraulic ones powered by an internal combustion engine. The proof of concept and the prototype tested in situ have performed better than expected, with autonomy for over a week of surveying without recharging, all the while being more silent and not generating greenhouse gas emissions. Both machines can be quickly mastered by experienced drillers as they use the same control scheme as the traditional hydraulic ones, leading to similar productivity. The data collected by logging the current and voltage used by the various motors has been shown to be as detailed or more so than the data registered while using pressure transducers in a conventional drill rig. The data can be displayed as the conventional physical parameters delivered to the drill bit or in terms of power spent by each motor and can be interpreted in the same manner as regular MWD logging.

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Declaration of interest

The authors have no conflicts of interest to declare. All co-authors have observed and affirmed the contents of the paper and there is no financial interest to report.

Authors' contributions

Michel Rispal: conceptualization, funding acquisition, data curation, project administration, writing – review & editing. Catherine Jacquard: funding acquisition, supervision, writing – review & editing. Guilherme de Oliveira Souza: data curation, formal analysis, methodology, visualization, writing – original draft. Philippe Reiffsteck: methodology, supervision, writing – review & editing.

Data availability

Data generated and analyzed in the course of the current study are not publicly available due to them being proprietary data, but a complete or limited dataset can be made available by Fondasol upon reasonable request.

Declaration of use of generative artificial intelligence

This work was prepared without the assistance of any generative artificial intelligence (GenAI) tools or services. All aspects of the manuscript were developed solely by the author, who takes full responsibility for the content of this publication.

List of symbols and abbreviations

CLEA	Connected, light, electric and autonomous (in French)
IC	Internal combustion
MWD	Monitoring while drilling

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Insights into drop mass systems for predicting pile compressive resistance from dynamic load test energy measurements: a case study of the Tagus River Lezíria bridge and viaducts

Jaime Santos^{1,2#} , Jeniffer Viegas² 

Case Study

Keywords

Dynamic load testing
Exploratory data analysis
Supervised models
Pile foundation

Abstract

The present era is defined by the advent of big data, where large volumes of data are generated and stored daily. However, the true challenge lies in transforming this data into meaningful and actionable information. To extract insights from these vast datasets, there is a growing dependence on data-driven methods, many of which build upon traditional statistical foundations. These techniques enable the creation of models that enhance our understanding of diverse subjects and facilitate informed decision-making. This study focuses on the establishment and exploration of a database derived from dynamic load tests on piles (DLT). In DLT on piles, impact hammers are employed, characterized by their potential energy or kinetic energy just before impact. Testing on high-capacity drilled deep foundations presents several challenges, including the need for sufficient energy to mobilize compressive static resistance of the pile. The investigation delves into understanding the correlation among various DLT test variables and uncovering potential relationships using statistical models, such as linear and non-linear regression. The findings from this exploration have unveiled crucial insights, such as the influence of diameter on pile compressive resistance and stiffness and also the existence of a non-linear relationship between resistance and the maximum energy transferred to the pile.

1. Introduction

Dynamic load testing systems apply a mass to the pile head to induce load. This can be achieved either through a pile driving hammer, known as an impact driving system, or by dropping a mass, referred to as a drop mass system. Dynamic load testing can be conducted during the pile installation of precast concrete piles or steel piles (displacement piles) when driving with a hammer. Drop mass systems are employed to test cast-in-situ piles (bored piles, continuous flight auger, or other cast-in-situ piles) or testing associated with re-driving. These drop mass systems are characterized by their mass and maximum stroke (drop height), or the respective potential energy (mass $\times$ acceleration $\times$ stroke) or kinetic energy immediately just before impact.

Testing high capacity drilled deep foundations poses several challenges, one of which is determining the required energy level to mobilize the compressive static resistance of the pile. Hussein et al. (1996) conducted a wave equation

analytical study for the selection of a hammer for DLT of cast-in-place shafts. The studied shafts have ranges of diameters from 750mm to 1500mm and lengths from 10 to 30 diameters. Analysis results suggest as a general guideline that the hammer weight should be about 1.5% of the pile resistance.

According to EN ISO 22477-4:2018, the mass of the drop mass should be chosen to be greater than 2% of the design compressive static resistance of the pile (where the mass of the drop mass is expressed as a weight) (EN ISO, 2018). In very hard soils, piles resting on hard bedrock, or where a pile is installed with a rock socket, drop mass weights of 1% of the required design compressive static resistance can be sufficient to mobilize pile resistance.

Ideally, the applied energy or the stroke of the drop mass should be adjusted to achieve full mobilization of the pile skin friction and tip resistance. This study focuses on the establishment and exploration of a database derived from dynamic load tests on cast-in-place foundation piles of the Tagus River Lezíria Bridge River.

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2. Case study

The Tagus River Lezíria Bridge comprises the north viaduct, the main bridge, and the south viaduct with a total length of about 12 km. The viaducts are supported by piles with a diameter of 1500mm, and their lengths vary between 25 and 55 meters.

The construction methodology defined to install the piles is the following (Caputo, 2009):

- Placement of a guide-steel tube with 6.00m of length;
- Driving a temporary casing with the use of vibrator down to the design depth;
- Internal cleaning of the casing up to the design depth with a hydraulic rig;
- Placement of the steel reinforcement cage;
- Cleaning of the pile tip by “air-lift”;
- Pouring of the pile concrete simultaneously with the removal of the steel casing.

To validate the construction methodology and the pile design (COBA, 2005), a series of static and dynamic load tests were proposed by Instituto Superior Técnico (Santos, 2005).

Dynamic load tests were performed on 9 trial piles (6Φ800mm and 3Φ1500 mm) using specific equipment called SIPEX — Impact System for Trial Piles.

The SIPEX system employs the free fall of two 100 kN weights, each with a maximum drop height of 3 meters, capable of acting together or separately. The setup includes a guide tube, and an attached steel cap block placed over the pile. This configuration ensures a centered and vertically applied impact while protecting the pile head from potential damage (Figure 1). Although designed for testing 1500 mm diameter piles, the system can accommodate smaller diameter piles using a reducer.

Multi-blow dynamic testing technique was employed, with applied energy varying between 40 and 600 kJ (or kNm) (Paraíso & Santos, 2020).



Figure 1. SIPEX: Drop mass system 100+100 kN.

The CAPWAP (Case Pile Wave Analysis Program) method of analysis was adopted for signal matching to the dynamic measurements.

The load tests were carried out in 3 locations along the route of the viaducts, and a supplementary ground investigation campaign was defined, consisting of boreholes accompanied by SPT tests and CPTu tests. There are alluvial deposits (soft silty clay, sands) overlying gravels, boulders and the Miocene bedrock (Figure 2).

3. Methodology and results

In the current investigation, a range of variables derived from the DLT measurements are available for analysis. The CAPWAP analysis enables the evaluation of the maximum energy transferred from the hammer to the pile, the shaft and base resistance as well as the pile top displacement. Just start analyzing four independent variables: the diameter of the pile (D), the potential energy of the hammer or rated energy (EP), the maximum energy transferred from the hammer to the pile (EMX), the mobilized resistance (R), and the ratio between the mobilized resistance and the service load (R/Q_s). To gain a deeper understanding of the data and the interplay among these variables, an exploratory data analysis (EDA) was undertaken. This type of analysis enables to discern the strengths of relationships between variables and to further elucidate these connections. The data underwent processing utilizing the Python open-source language, leveraging prominent libraries including Pandas, Scipy, NumPy, Seaborn, and Matplotlib.

Figure 3 showcases the correlation matrix for the aforementioned variables, while Figure 4 illustrates scatter plots for variable pairs. In this plot, piles with a diameter of 800 mm are depicted in green, and those with a 1500 mm diameter are represented in blue, complemented by variable histograms. A careful examination of these visualisations, along with the low correlation coefficients (-0.089 between diameter (D) and rated energy (EP), and 0.095 between D and mobilised resistance (R), indicates that the diameter is linearly independent of both variables, showing no meaningful linear association. As anticipated, a larger diameter would correspondingly increase resistance. Additionally, it is apparent that there exists a non-linear relationship between mobilized resistance (R) and maximum energy transferred from the hammer to the pile (EMX), with a correlation coefficient of 0.87 . The influence of pile diameter is also evident, separating the data into two groups, easily discernible by colour in the scatter plot.

The efficiency of the hammer is commonly assessed by calculating the ratio between the rated energy (EP) and the maximum energy transferred from the hammer to the pile (EMX). The maximum hammer height of drop is 3m leading to a 600 kNm rated energy, displayed in Figure 5 as EP . It is evident that the survey was carried out for several heights (multi-blow testing technique) since the weight of the hammer was kept constant and equal to 200 kN. As

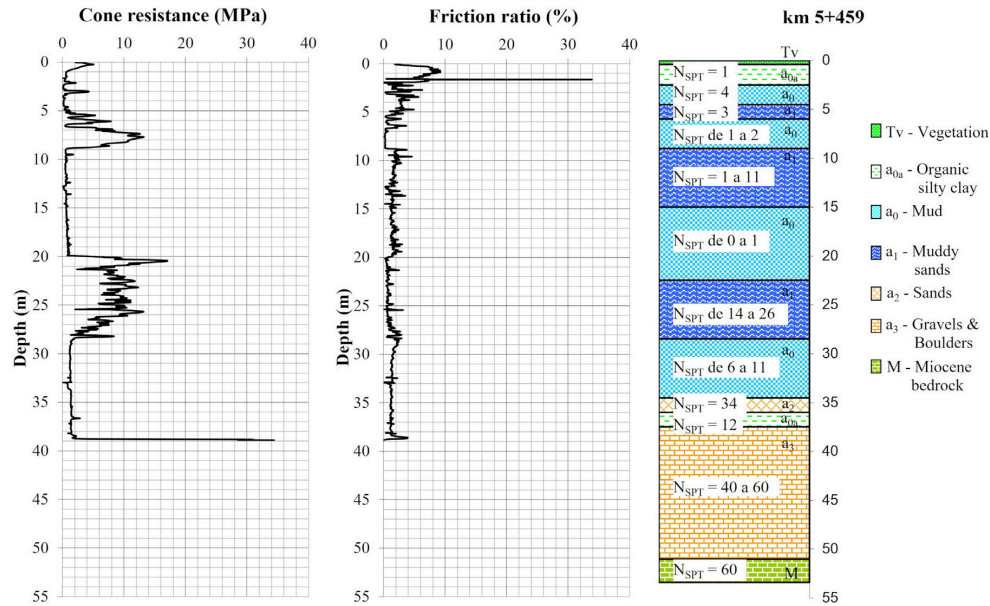


Figure 2. Typical Geological and Geotechnical Profile.

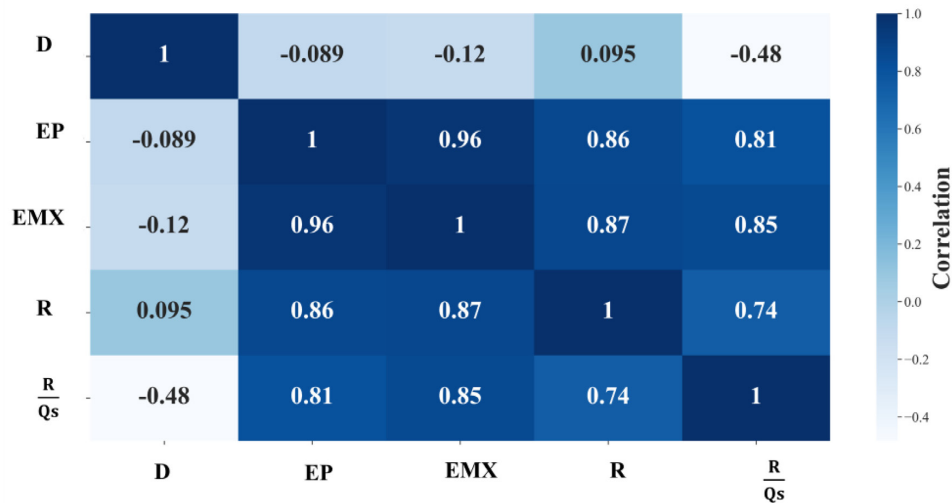


Figure 3. Correlation matrix for input variables.

depicted in Figure 5, it is evident that *EMX* is approximately 0.6 ($=1/1.69$) times *EP*, indicating a 40% energy loss.

In this study, the equivalent η coefficient was determined to be 0.024m, as depicted in Figure 6 ($\eta=EP/R$). This value aligns with existing literature; for instance, Paraíso & Costa (2006) reported a value of 0.033m, while Paikowsky (2004) presented a value of 0.025m. The dataset in this study displays lower dispersion, and the regression analysis yields a coefficient of determination (R^2) of 0.89, indicating a strong fit to the observed data.

It is well-known that not all energy can be transferred from the hammer to the pile. If this were possible, the coefficient η would represent the displacement associated

with the plastic deformation occurring at the lateral interface of the soil-pile and at the tip (Paraíso & Santos, 2020). As indicated by EN ISO 22477-4:2018, Rausche et al. (2008) and Hussein et al. (1996), recommends that the weight of the hammer should be 1% - 2% of the desired resistance. Figure 7 elucidates this reasoning, where the ratio between the maximum energy transferred from the hammer to the pile (*EMX*) and the rated energy (*EP*) is plotted against the ratio between the hammer weight (*W*) and the mobilized resistance (*R*).

It is evident that for the 800 mm diameter, the loss of energy is minimized when the weight/resistance ratio falls between 1% - 2% (shaded area in Figure 7), more

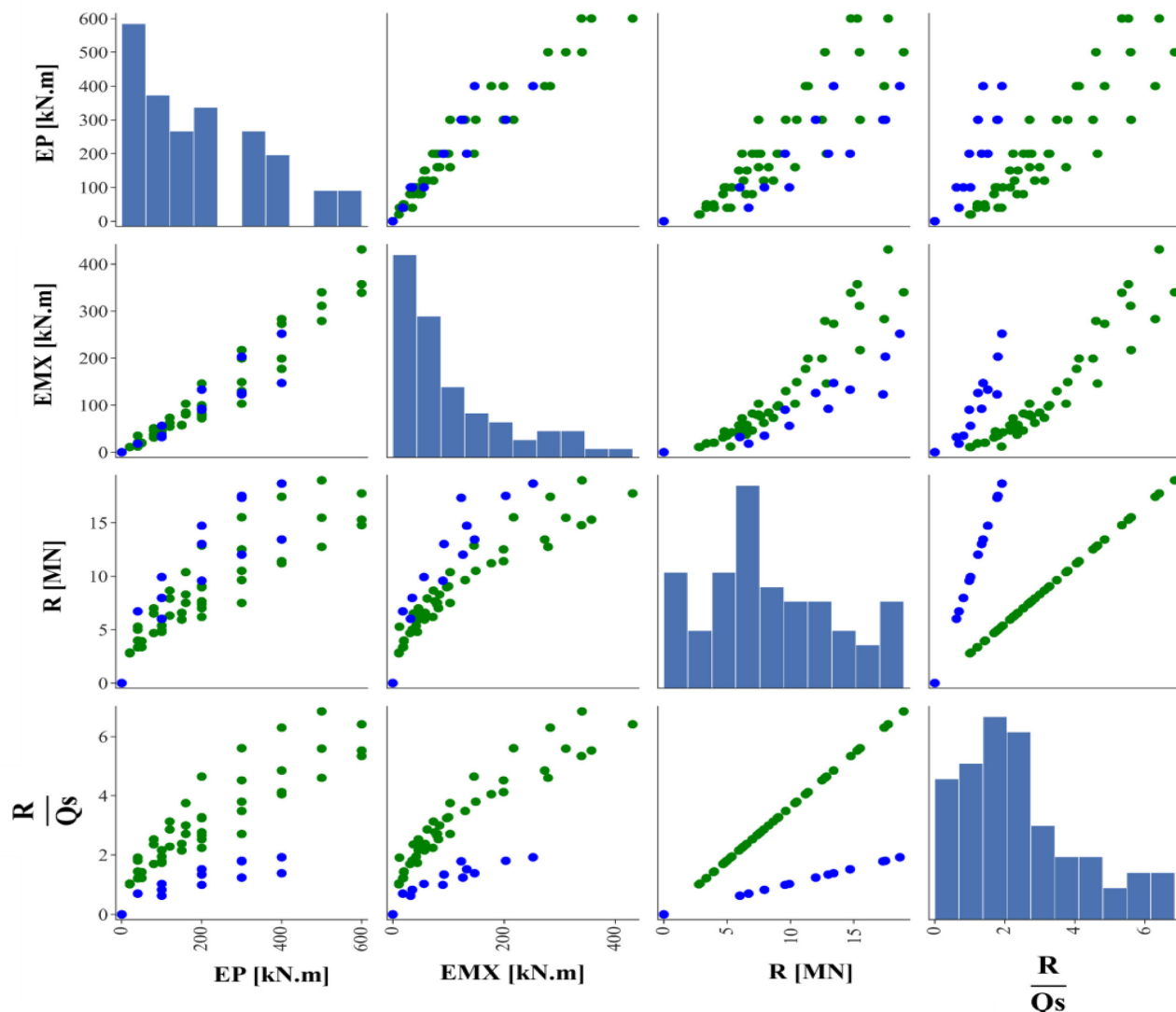


Figure 4. Scatterplot matrix depicting the relationships among the investigated variables. The green color represents the piles with 800 mm diameter and the blue the ones with 1500 mm.

precisely between 1% – 1.3%, in this study, for the blows that mobilized the maximum resistance. However, for the 1500 mm diameter, three points deviate from the trend, and within this interval, they still exhibit a lower energy ratio, approximately 40%. The efficiency of the hammer ranges from 30% to 87.5%, with a mean of 50.9%.

Upon closer examination of the *EMX* distribution for both diameters, as illustrated in Figure 8, it becomes evident that both distributions exhibit comparable means. However, the 1500 mm diameter piles display higher dispersion due to the limited number of samples, which could justify this misalignment with the trend in Figure 7.

While the presented results are intriguing, the aim is to offer practical insights beneficial for designers. A key interest lies in determining the hammer parameters required to attain a

specific resistance. In order to address this, a non-linear regression analysis was conducted between *EMX* and *R*, as presented in Figure 9. Various functions underwent testing, and the one presented yielded the lowest Root Mean Square Error (*RMSE*).

Notably, the equation for the 800 mm diameter is approximately double that of the equation for the 1500 mm diameter, revealing the influence of the diameter on the equation. It is important to emphasize that the regression was constrained to pass through the origin, reinforcing the expectation of zero energy being applied at zero resistance. This graphical representation underscores that the requisite energy for achieving a given resistance is consistently lower for the larger diameter, thereby justifying an increased cost in pile designs with a corresponding reduction in associated

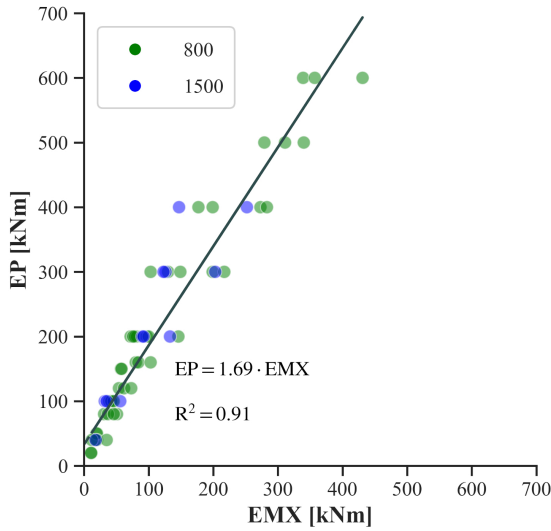


Figure 5. Relationship between the rated energy (EP) and the maximum energy transferred from the hammer to the pile (EMX), for both diameters 800mm (green) and 1500 mm (blue).

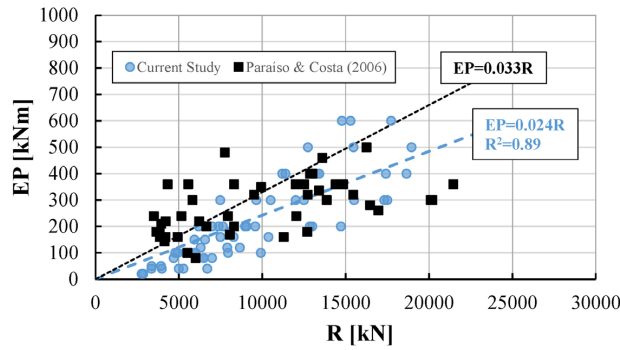


Figure 6. Comparison between the dataset of the current study (in blue) and the one presented in Paraíso & Costa (2006) (in black).

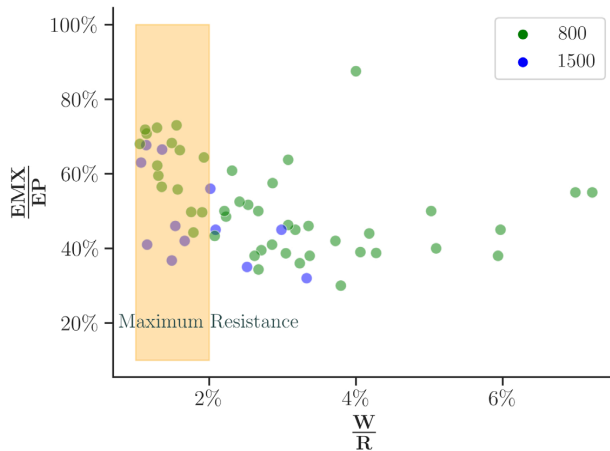


Figure 7. Relationship between the ratio of maximum energy transferred from the hammer to the pile (EMX) to rated energy (EP) and the ratio of hammer weight (W) to resistance (R). The orange area highlights the zone where the maximum resistance is mobilized.

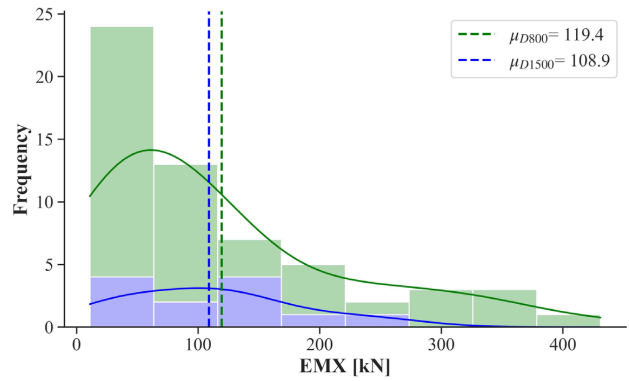


Figure 8. Distribution of the maximum energy transferred from the hammer to the pile (EMX) for both diameters. The green color represents the 800 mm diameter, while the blue represents the 1500mm. The dashed lines are the mean of each distribution.

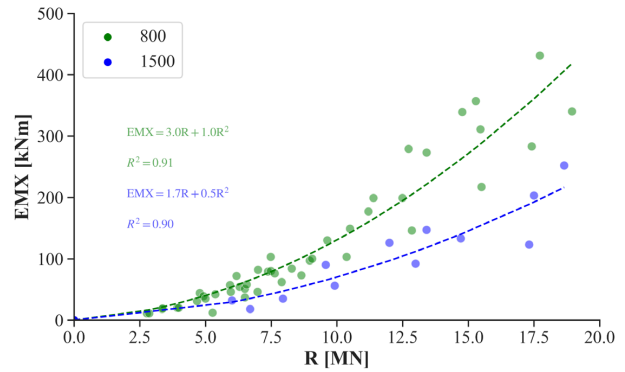


Figure 9. Non-linear relationship between the maximum energy transferred from the hammer to the pile (EMX) and the mobilized resistance (R). The green color represents piles with a 800 mm diameter, while the blue represents those with a 1500mm diameter.

equipment, or where the applied energy should be limited due to displacement issues.

Furthermore, three more variables are added to the analysis to gain a deeper understanding of our data (Figure 10): the mobilized lateral resistance (R_s), the mobilized base resistance (R_b), and the maximum pile top displacement (DMX).

Upon analyzing the decomposition of the total mobilized resistance (R) into its lateral component (R_s) and base component (R_b) alongside the maximum energy transferred from the hammer to the pile (EMX), it becomes evident that the nonlinearity in the R - EMX relationship primarily arises from the dominant influence of the lateral component (R_s), as illustrated in Figure 10. This becomes evident when the lateral resistance is approaching the ultimate value. The R_b - EMX relationship appears more linear, whereas the R_s - EMX plot exhibits nonlinearity. Furthermore, in the R_b - EMX plot, the influence of the pile diameter is significant, as it distinctly separates the data into two groups, highlighting

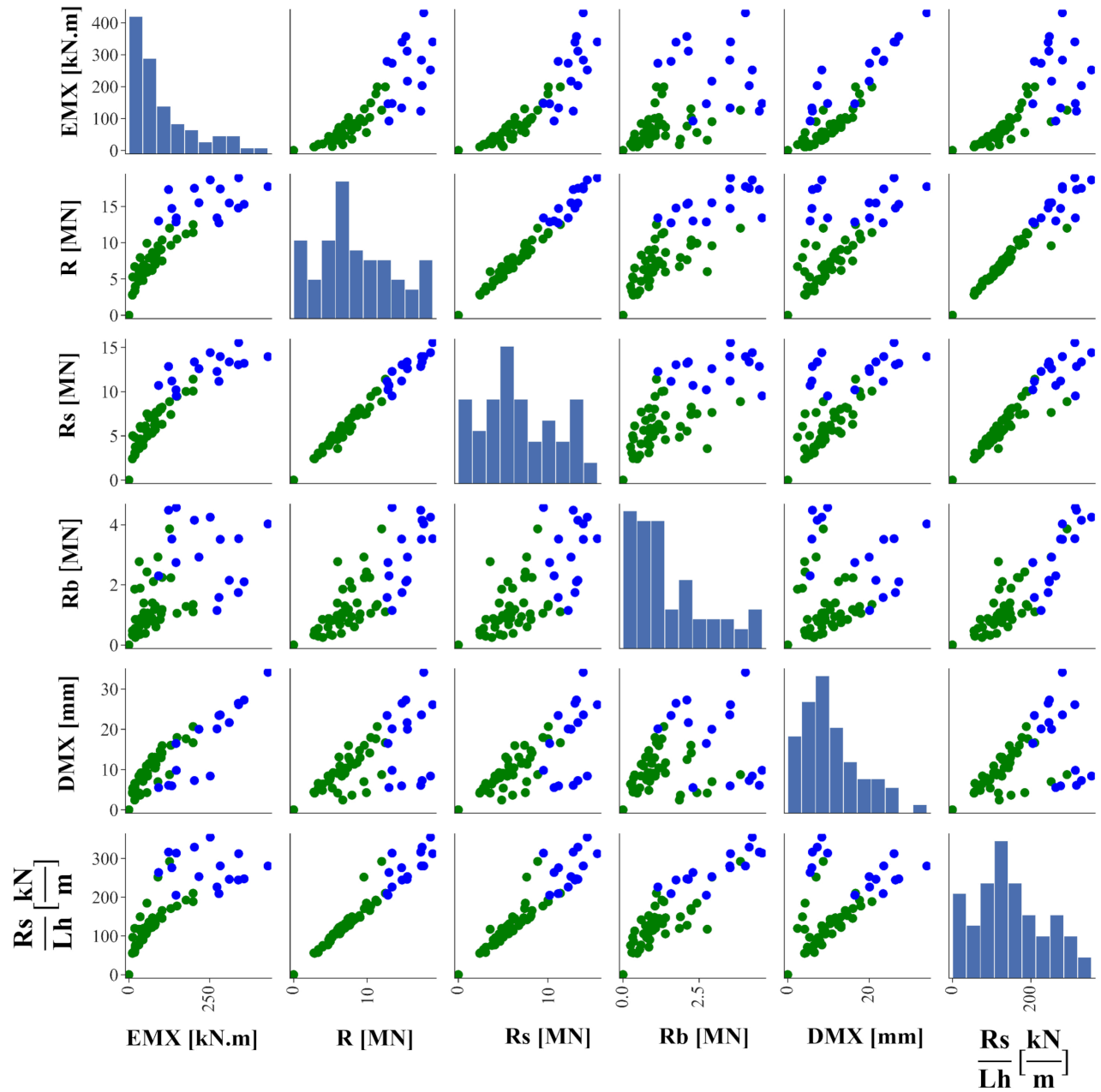


Figure 10. Scatterplot matrix depicting the relationships between mobilized resistance (R) and its components (R_s , R_b), the maximum pile top displacement (DMX), and the maximum energy transferred to the pile (EMX). The green color represents the piles with 800 mm diameter and the blue the ones with 1500 mm.

the influence of pile diameter. In contrast, the R_s - EMX plot shows much lower dispersion, suggesting less influence of the diameter. These trends appear to be logical since the base resistance varies with the square of the diameter, while the lateral resistance varies linearly with the diameter of the pile.

To gain a deeper understanding of these relationships, a regression analysis was conducted, with the variables grouped by pile diameter and linked to their corresponding piles (Figure 11). This approach allows us to assess the

influence of spatial variation and lithology on the variables. The dynamic pile load tests were carried out in 3 locations with distances of approximately 3 km between them.

Figure 11 summarizes the relationships involved with the maximum energy transferred to the pile (EMX). The green lines and crosses denote piles with an 800 mm diameter, whereas the blue lines and circles represent piles with a 1500 mm diameter.

The regression results in Figure 11 demonstrate a strong fit for R , R_s , and DMX . In contrast, R_b exhibits considerable

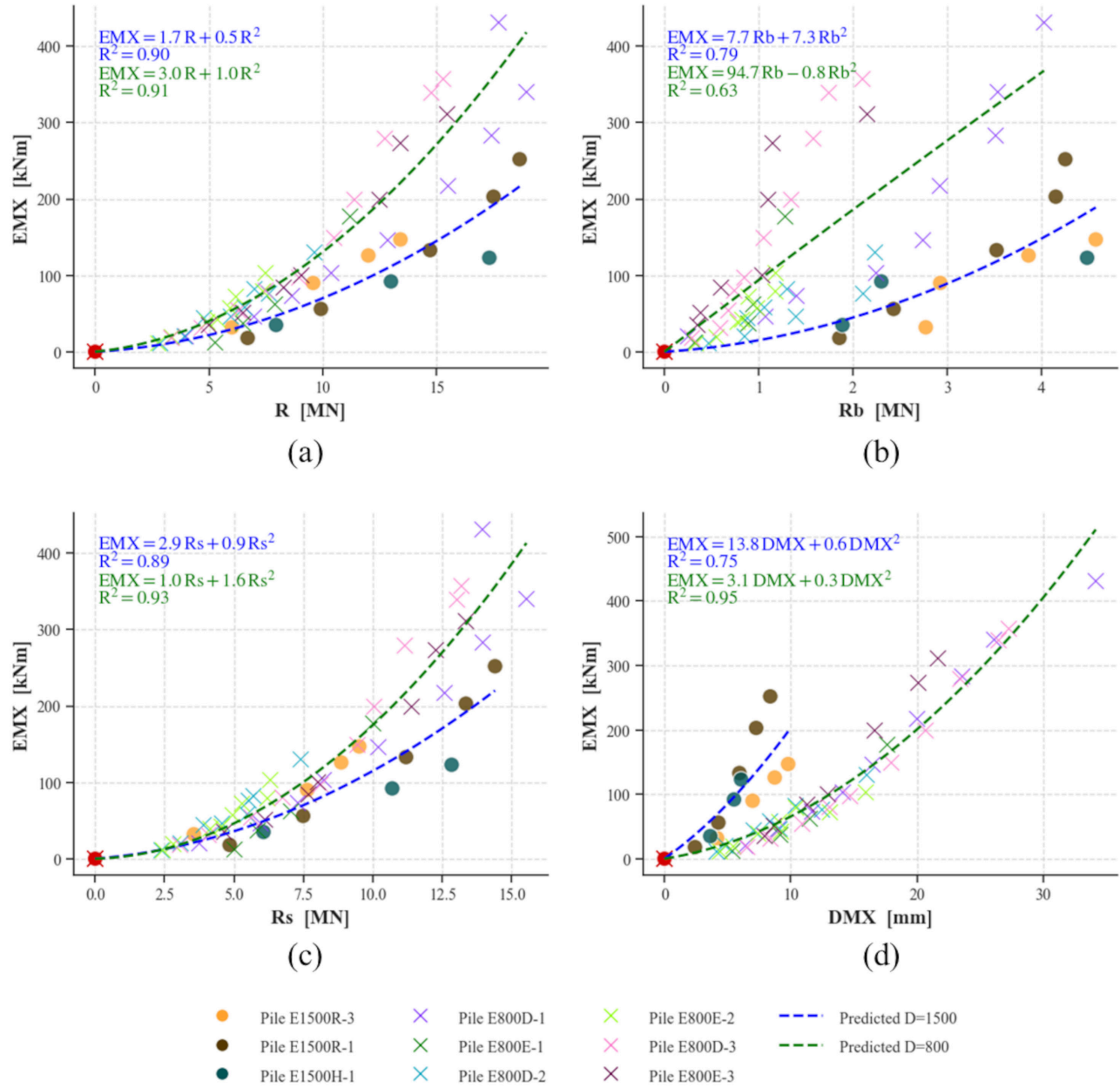


Figure 11. Non-linear relationship between the maximum energy transferred to the pile (EMX) and: (a) the total mobilized resistance (R), (b) the base resistance (R_b), (c) the lateral resistance (R_s), (d) the maximum pile top displacement (DMX).

scatter, largely attributable to variations in the embedding soil material, which ranges from Miocene deposits to bouldery formations. Additionally, installation effects during construction may influence the observed behavior. Besides, the R_b - EMX relationship appears quite linear, so it is not possible to derive any considerations about the ultimate base resistance. It is also important to highlight that the lateral resistance is dominant in all piles tested, representing more than 83% of the total resistance. The R_s - EMX curve exhibits

nonlinearity and physically should converge to asymptotic values that represent the ultimate lateral resistance.

For reasons of uniformity and simplicity in data processing, polynomial regression functions were used in the previous figures. While these functions offer flexibility in modeling nonlinear relationships, they lack asymptotic values, which can be a limitation when modeling physical problems. For such cases, hyperbolic functions are more suitable, as they can easily handle asymptotic behavior. For this reason, the lateral resistance divided by the total length R_s/L is plotted against EMX and fitted using hyperbolic functions in Figure 12.

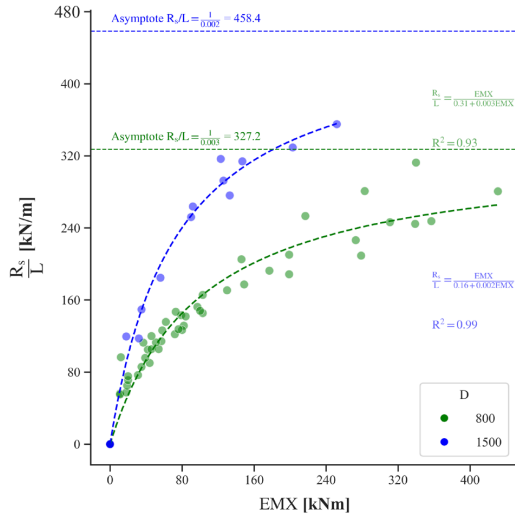


Figure 12. The relationship between the lateral resistance per unit length (R_s/L) and the maximum energy transferred to the pile (EMX).

The regression results shown in Figure 12 indicate a strong fit for both pile diameters ($R^2 \geq 0.93$). The asymptotic values of the regression curves represent the ultimate lateral resistance per unit length. The ratio $458.4 / 327.2 \approx 1.4$ is relatively close to the ratio of the pile diameters, $1500 / 800 \approx 1.87$. However, the 1500 mm diameter reaction piles are slightly shorter and have a reduced embedded length within the Miocene deposits or bouldery formations. This difference in geometry and soil interaction helps explain why, on average, the ultimate lateral resistance per unit length is lower for the larger-diameter piles.

Figure 13, similar to Figure 11, illustrates the relationship between maximum pile top displacement and the mobilized resistance components. The data follows a linear regression trend, confirming that piles with larger diameters experience significantly less displacement, as previously mentioned. This linear behavior arises because the DMX value represents the maximum displacement associated with a given blow and

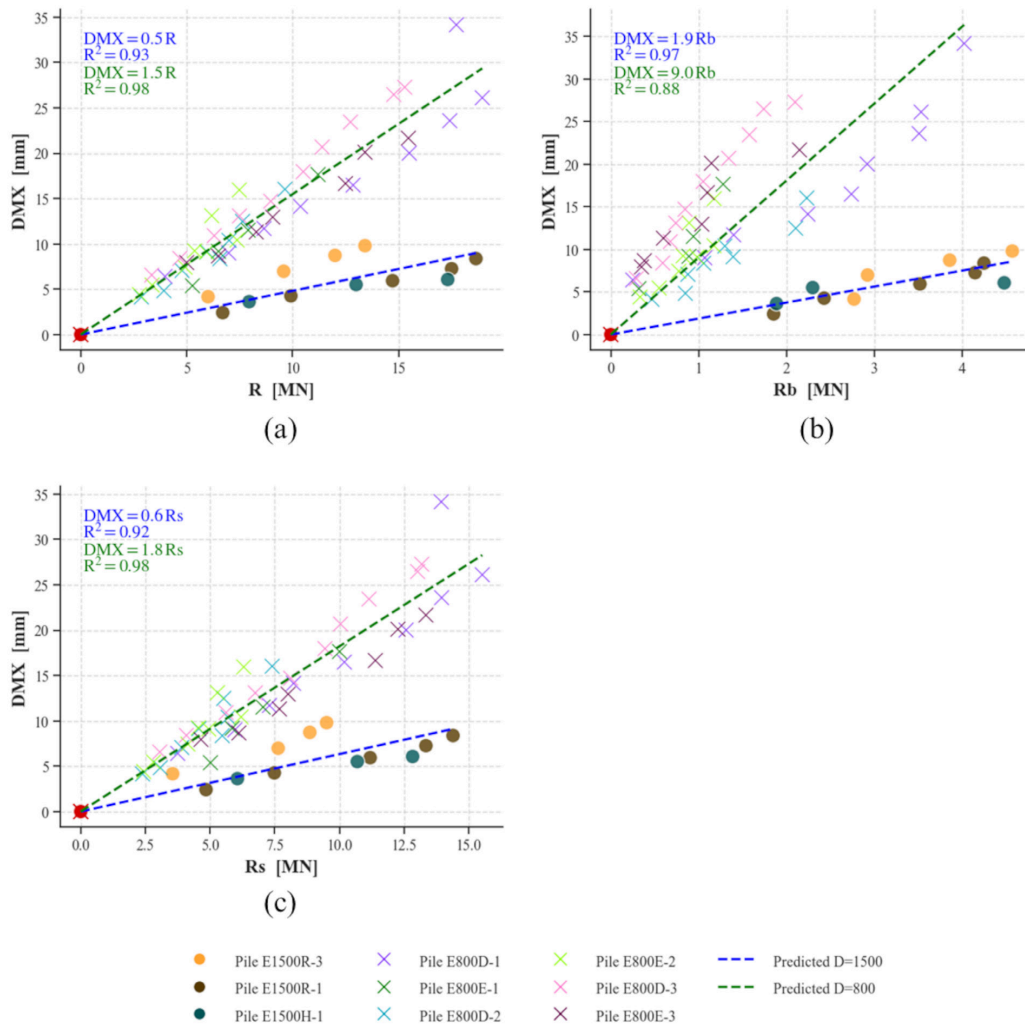


Figure 13. Linear relationship between the maximum displacement (DMX) and: (a) the total mobilized resistance (R), (b) the base resistance (R_b), (c) the lateral resistance (R_s).

does not account for the permanent displacement accumulated from previous blows. In this sense, it reflects the quasi-elastic response of the resistance during successive load-unload cycles, as observed in dynamic load tests involving multiple blows of increasing energy.

Examining the relationship between the pile's top maximum displacement (DMX) and its components, Rb and Rs , it is possible to observe a clear distinction between different diameters. Specifically, for the same resistance, piles with smaller diameters (green) exhibit three times greater displacement, likely due to movement along the soil–pile lateral interface. This interface displacement is governed by the thickness of the soil shear band and the relative displacement between the soil and the pile. In this case study, the soil type is essentially the same for all tested piles; however, the soil–pile relative displacement is influenced by pile shortening, i.e., by compressive stress. The ratio of the cross-sectional areas is approximately 3 ($1500^2 / 800^2 = 3.5$), which supports the trend observed in the data.

4. Conclusion

In this comprehensive investigation, variables obtained from the Dynamic Load Test (DLT) were examined, including the maximum potential energy of the hammer (rated energy, EP), the maximum energy transferred from the hammer to the pile (EMX), and the mobilized resistance (R). The interrelationships between these variables were also explored in detail. Practical implications for predicting pile compressive resistance from dynamic load test energy measurements are underscored and materialized through regression equations. The results obtained not only align with current literature but also pave the way for valuable insights. Specifically, the formulated equations for predicting pile compressive resistance demonstrate excellent agreement with the dataset, boasting a coefficient of determination (R^2) around 0.9 (Figures 6 and 9). This high level of correlation underscores the robustness and reliability of the equations in capturing the dynamics of the system.

In pursuit of a more comprehensive understanding, examining the relationships with additional variables - such as lateral resistance, tip resistance, and the maximum pile top displacement - separately, could significantly enhance the accuracy and applicability of the derived equations. For this case study, the lateral resistance is dominant in all piles tested. The hyperbolic functions used to predict the lateral resistance per unit length demonstrate an excellent fit, with a coefficient of determination greater than or equal to 0.93 (Figure 12). Additionally, the relationships between the lateral resistance and the pile top displacement offer valuable insights into the stiffness behavior of the soil–pile system (Figure 13).

This study stands as a significant contribution to the field, providing new practical tools for engineers and designers involved in pile-related projects, in particular at the Tagus

River zone in Lisbon, potentially contributing to cost and time savings by reducing the need for extensive in-situ pile testing.

This study also represents the initial step toward developing machine learning approaches — the exploratory data analysis phase. The available features were examined, their interrelationships assessed, and the most relevant variables for future predictive modeling identified. The next step involves expanding the dataset by incorporating additional variables, such as results from cone penetration tests (CPTu) and standard penetration tests (SPT), to further enhance the understanding of pile behavior with integrated physics knowledge. A key challenge will consist of the use of a dataset of sufficient size and quality to ensure robust training and validation of the model.

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Declaration of interest

The authors have no conflicts of interest to declare. All co-authors have observed and affirmed the contents of the paper and there is no financial interest to report.

Authors' contributions

Jaime Santos: conceptualization, methodology, data interpretation, writing – original draft. Jeniffer Viegas: formal analysis, software, methodology, writing – original draft.

Data availability

All data produced or examined in the course of the current study are included in this article.

Declaration of use of generative artificial intelligence

This work was prepared with the assistance of generative artificial intelligence (GenAI) OpenAI's ChatGPT with the aim of enhancing readability and refining language. The entire process of using this tool was supervised, reviewed and when necessary edited by the authors. The authors assume full responsibility for the content of the publication that involved the aid of GenAI.

List of symbols and abbreviations

a_0	Alluvial mud
a_{0a}	Alluvial Organic silty clay
a_1	Alluvial muddy sands
a_2	Alluvial sands
a_3	Alluvial gravels & boulders
CAPWAP	Case Pile Wave Analysis Program
CPTu	Cone penetration test
D	Pile diameter
DLT	Dynamic load test
DMX	Maximum pile top displacement
EDA	Exploratory data analysis
EMX	Maximum energy transferred from the hammer to the pile
EP	Potential energy of the hammer or rated energy
L	Pile length
M	Miocene bedrock
N_{SPT}	Number of blows in SPT tests
Q_s	Pile service load
R	Pile (mobilized) total resistance
R^2	Coefficient of determination
R_b	Pile (mobilized) base resistance
RMSE	Root mean square error
R_s	Pile (mobilized) lateral resistance
SIPEX	Impact System for Trial Piles
SPT	Standard penetration test
T_v	Vegetation
W	Hammer weight
η	Ratio between the potential energy and he mobilized pile resistance
μ_{D1500}	Mean EMX for piles with 1500 mm diameter
μ_{D800}	Mean EMX for piles with 800 mm diameter
Φ	Pile diameter

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Numerical simulation of responses of a grid-shaped soil improvement foundation (TNF) on silty ground subjected to consecutive earthquakes

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Tatsunori Matsumoto³ 

Case Study

Keywords

TNF
Liquefaction
Settlement
Silty ground
Finite element method (FEM)

Abstract

Liquefaction induced by earthquakes causes instability of buildings such as large average and/or relative settlements. Such occurrences can lead to damage to the building structures. Usually, when an earthquake occurs, saturated loose sandy ground is the most susceptible to liquefaction. Even for saturated silty ground with small SPT N -values, the possibility of liquefaction needs to be considered in the design process of the foundation. Tender Net Foundation (TNF) is a kind of shallow foundation with a combination of grid-shaped soil improvement to depths of 2 to 3 meters, concrete slab, and concrete single footings. In this paper, the dynamic analysis of an actual TNF on the silty ground subjected to three consecutive earthquakes was conducted. Measurements of the settlements of the TNF were initiated after the second earthquake. The silty ground was modeled by the UBC3D-PLM model. The ground accelerations recorded about 10 km away from the site area were used as the input accelerations in the dynamic analyses. The responses of the TNF as well as the ground during and after the earthquakes were analyzed. The numerical analyses simulated the measured settlements well. Furthermore, a comparison was conducted between the TNF and the conventional shallow foundation to demonstrate the advantages of the TNF in reducing the average and differential settlements.

1. Introduction

Tender Net Foundation (TNF) (Takeuchi Construction Inc., 2025) is a kind of shallow foundation with a combination of grid-shaped soil improvement (primary soil improvement), a flat soil improvement on the primary soil improvement (secondary soil improvement), concrete slab and concrete single footings (see Figures 1 and 2). The original soil is improved by blending with cement as a solidification material using a backhoe. Cement powder with lower elution of hexad chrome is mixed with the original soil at the site for environmental friendliness. The mass of cement per unit soil volume is varied so that the improved soil has an unconfined compression strength q_u of 300 to 450 kPa in common. Han Vo Cong et al. (2022) analyzed the TNF system subjected to vertical loading and demonstrated that the TNF system is an efficient foundation for reducing settlement and volume of soil improvement compared to the conventional shallow soil improvement method.

By improving the surface layer to depths of 2 to 3 meters in a grid shape, the TNF could be an efficient foundation for suppressing the damage to buildings when liquefaction occurs in the ground under the TNF.

In this paper, the dynamic analysis of a TNF for a one-story factory building on silty ground subjected to three consecutive earthquakes is conducted. Measurements of the settlements of the TNF were initiated one week after the second earthquake. The construction of the building was not completed at the first earthquake.

PLAXIS 3D FEM software (Bentley, 2022) was used for simulation analysis. The silty ground was modeled by the UBC3D-PLM model. The time histories of ground accelerations of three earthquakes recorded about 10 km away from the site area were used as the input accelerations in the dynamic analyses of the TNF. The responses of the TNF as well as the ground during and after the earthquakes were calculated. The calculation simulated the measured settlements well. The analysis showed that liquefaction occurs in the silty ground. To demonstrate the

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advantages of the TNF for suppressing the settlements induced by liquefaction, a similar analysis of a conventional shallow foundation was also carried out for comparison purposes.

2. Outline of building and ground

Figure 3 shows the plan view of the construction site and two boreholes (BHs). The target of this analysis is a TNF for the foundation of a factory constructed in Fukushima, Japan. The factory is a one-story building with a floor dimension of 30 m × 19.5 m and a height of 5.7 m.

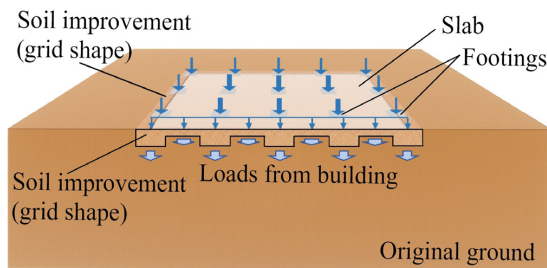


Figure 1. Section view of a TNF system.

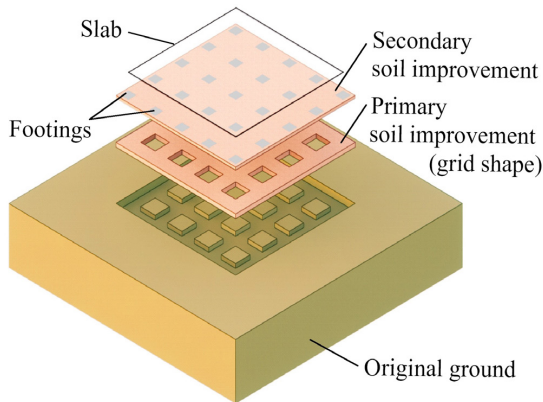


Figure 2. Composition of a TNF system.

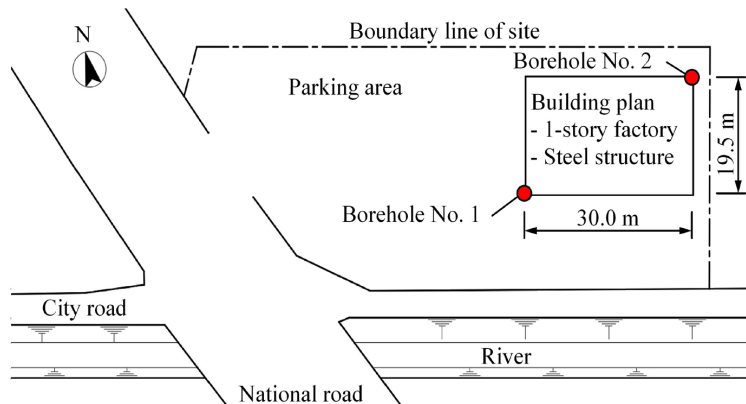


Figure 3. Plan view of the construction site and boreholes.

Figure 4 shows the profiles of the soil layers and the SPT N -values. The ground at each borehole consists of 7 soil layers. The sandy silt layer ② ($N=1$) exists at depths 0.65 to 2.85 m (2.2 m thick) at BH No. 1, and at depths 1.19 to 3.09 m (1.9 m thick) at BH No. 2. The silt layer ④ ($N=0 \sim 1$) exists at depths 3.85 to 13.65 m (9.8 m thick) at BH No. 1, and at 3.89 to 14.79 m (10.9 m thick) at BH No. 2. The groundwater level is 1.05 m at BH No. 1 and 1.19 m at BH No. 2 below the ground surface. Soil tests were conducted on the soil specimens of silt layers ② and ④ at only BH No.1.

Figure 5 shows the plan view of the TNF system used for the foundation of the factory building. The size of the TNF system is 33.8 m × 25.6 m having a primary soil improvement layer with a thickness of 1.5 m, and a secondary soil improvement layer with a thickness of 0.8 m. The thickness of the concrete footings is 0.4 m and the thickness of the concrete slab is 0.2 m.

3. History of earthquakes and settlement measurements

Figure 6 shows the construction progress, earthquake history, and measured settlements at the five points of the TNF (see Figure 5). The first severe earthquake (EQ) with Japan Meteorological Agency (JMA, 2023) Seismic Intensity scale (SI scale) of 5- (5 lower) occurred on 2016-11-22, about 3 months after the completion of the TNF and just before the completion of the building on 2016-11-29. The settlements were zero at the completion of the TNF. The second severe EQ with SI scale of 5- occurred on 2017-02-28. The settlement measurements were initiated on 2017-03-07, one week after the second severe EQ. Several moderate EQs with SI scale of 4 occurred after the second severe EQ, and the third severe EQ with SI scale of 6- occurred on 2021-02-13. The settlement measurements were made on 2021-05-20. Interestingly, large ground settlements were measured after the occurrence of earthquakes having SI scale of 5- or higher. For example, an earthquake with SI scale of 5- is one in which many people are frightened and feel the need to hold onto something stable.

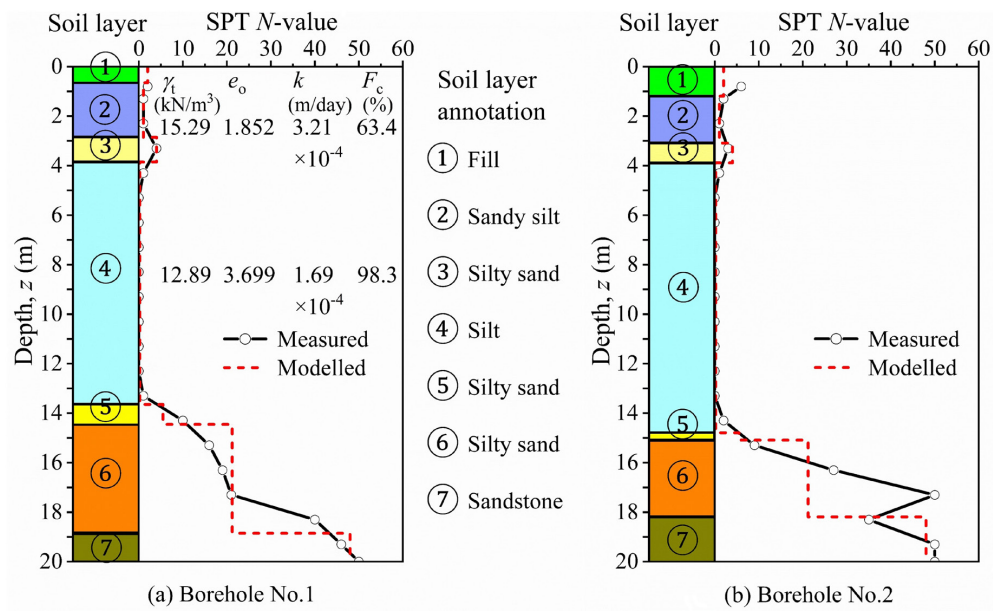


Figure 4. Profiles of soil layers and SPT N -values.

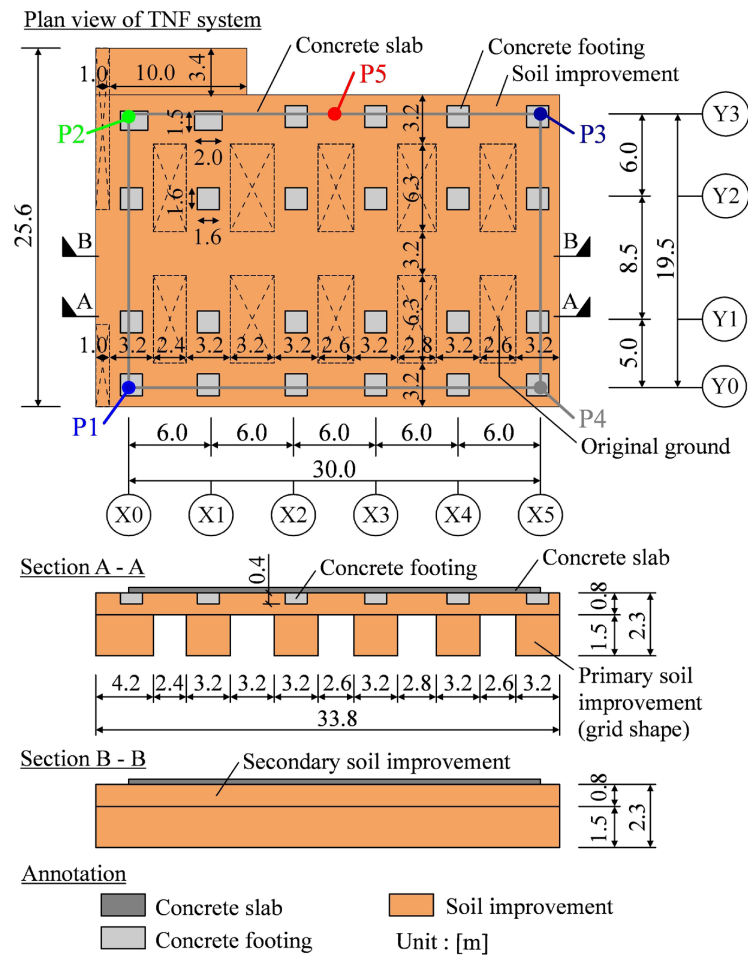


Figure 5. Plan view of the TNF system.

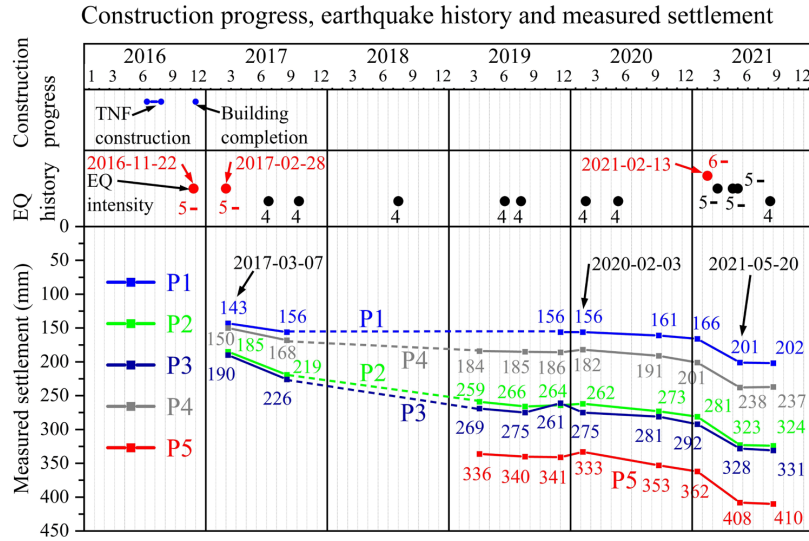


Figure 6. Construction progress, earthquake history, and measured settlements.

Hanging objects such as lamps swing violently. Dishes in cupboards and items on bookshelves may fall. Small cracks may form and liquefaction may occur in the ground. Refer to JMA (2023) for more details on *SI* scales.

Figure 7 shows the distributions of relative settlements of the TNF measured on 2017-03-07, on 2020-02-03 and on 2021-05-20 (98 days, 1161 days and 1633 days after the building completion), respectively. Measurements of the settlements of the TNF on 2017-03-07 were made at the four corners of the building. Those on 2020-02-03 were made at each position of the columns of the building. Those on 2021-05-20 were made at the periphery of the building. The measured relative settlements Δw were defined as the difference between the settlement at given points and the settlement at point P1 (X_0, Y_0).

It is seen that the maximum value of Δw and the average settlement w_{ave} on 2021-05-20 increased compared with those on 2020-02-03. That is, the maximum value of Δw was 186 mm and settlement at the reference point P1 was 156 mm in Figure 7b. Those were 207 mm and 201 mm in Figure 7c. Hence, the influence of earthquakes is strongly suspected as the cause of the differential settlement.

To investigate the cause of settlements of the TNF, the dynamic FEM analysis is conducted in this research.

4. Dynamic FEM analysis

4.1 Analysis conditions

Figure 8 shows the PLAXIS 3D FEM analysis model. The ground was modeled by 7 soil layers. The thickness of fill layer ① is 0.65 m at BH No.1 and 1.19 m at BH No.2. The dead loads of the building were applied as the point loads on the footings and the live load was applied as the

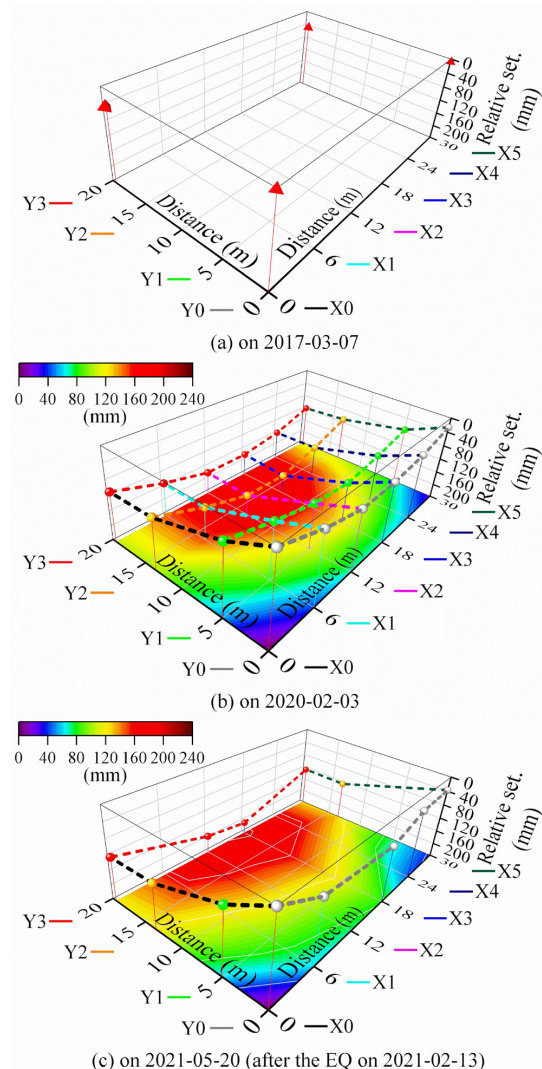


Figure 7. Distribution of measured relative settlements of the TNF.

uniform surface load of 3 kPa on the slab. The earthquake loads (accelerations) were applied to the bottom of the ground.

Figure 9 shows the analysis phases. The analysis phases were set following the construction progress and earthquake history. In this research, earthquakes with seismic intensity scale SI of 5- or higher on 2016-11-22 (1st EQ, $SI = 5-$), on 2017-02-28 (2nd EQ, $SI = 5-$), and on 2021-02-13 (3rd EQ, $SI = 6-$) were considered. The seven earthquakes with SI of 4 between 2017-02-28 and 2021-02-13 were omitted in this particular analysis because the settlements caused by

these earthquakes were considered negligible. The ground accelerations of the earthquakes recorded about 10 km away from the site area were used as input accelerations of the three earthquakes in the dynamic analyses at Phases 3, 5 and 6. Free-field conditions were set at the outer side surfaces of the ground model. Undrained conditions were set for the ground below the groundwater level, while drained conditions were set for the ground above the groundwater level.

In the other phases, consolidation phenomena were considered. Horizontal displacements of the outer side surfaces of the ground model were fixed in the consolidation analyses.

Figure 10 shows the input acceleration data used in the dynamic analyses. The acceleration data were obtained from K-NET strong-motion seismograph network (Kyoshin Network, 2022) and were converted to acceleration data into x - and y -directions.

Table 1 shows the physical and mechanical properties of each soil layer. The thicknesses of soil layers, t_1 (in Borehole No. 1) and t_2 (in Borehole No. 2), along with the SPT N value for each soil layer, were determined based on the soil test results (see Figure 4). The wet unit weight γ_s , initial void ratio e_0 and coefficient of permeability k of the sandy silt layer (2) and the silt layer (4) were obtained from soil test results (see Figure 4), while the corresponding properties for the other soil layers were assumed to be typical value of each soil type. In this analysis, the silt, sandy silt and silty sand layers (soil layers (2), (3), (4), (5) and (6)) were modeled using the UBC3D-PLM model, while the other soil layers (soil layers (1) and (7)) were modeled using the linear elastic model.

Some rigorous constitutive models for liquefaction have been proposed, *ie.* SANISAND by Dafalias & Manzari (2004), t_{ij} model by Nakai et al. (2011) and so on. However, these models typically require a wide range of advanced laboratory test results to accurately determine input parameters. In this project, only SPT N -values were available, along with limited basic soil test data conducted on the silt layers (2) and (4)

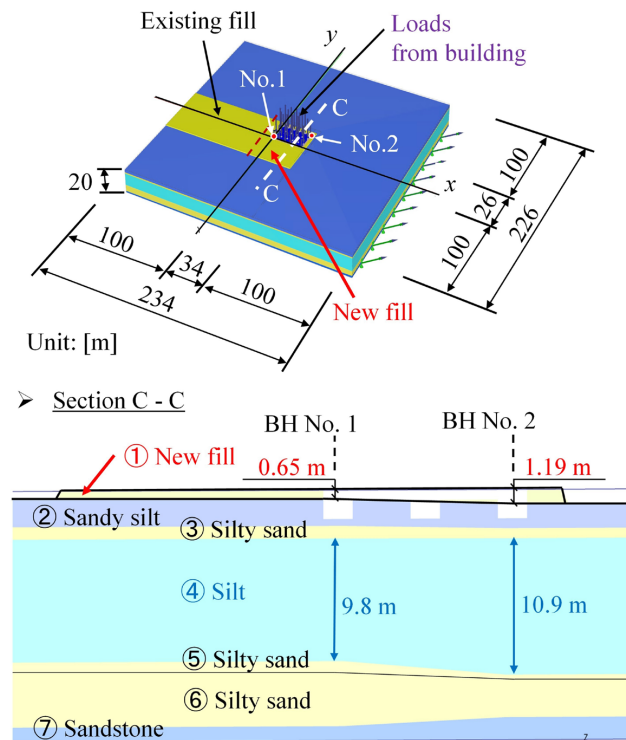


Figure 8. PLAXIS 3D FEM analysis model.

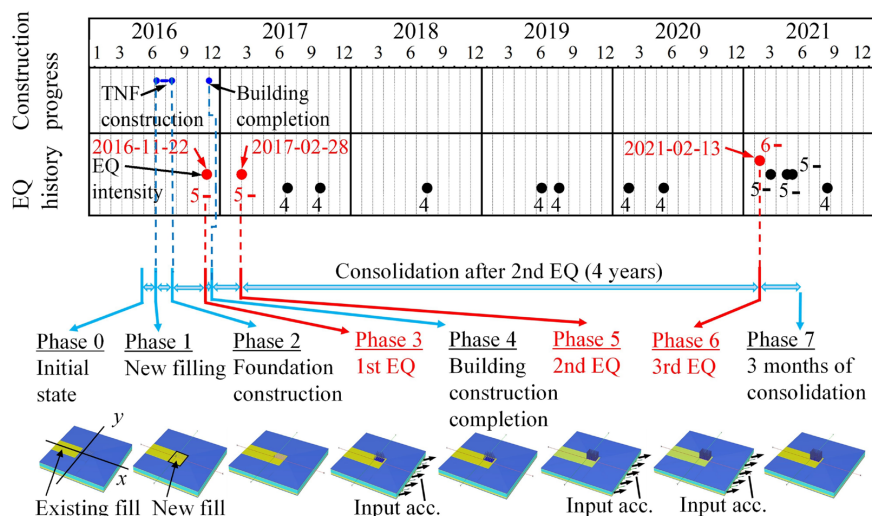


Figure 9. Analysis phases.

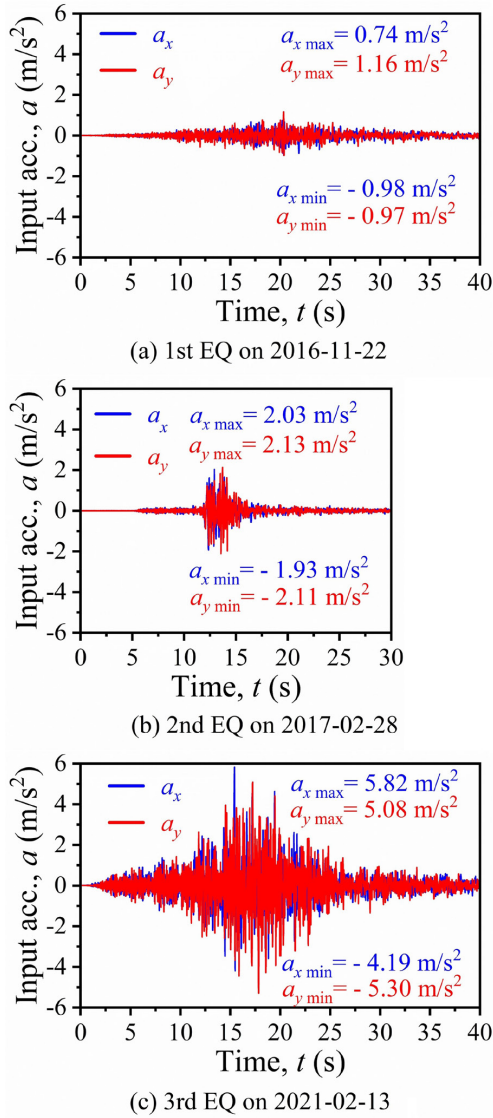


Figure 10. Input acceleration data.

at borehole BH No.1. The available data were not enough to use more complex models. Therefore, a more practical approach was adopted using the UBC3D-PLM model, which allows input parameters to be empirically estimated based on SPT N -values and basic soil properties.

Table 2 shows the parameters of the linear elastic model applied to soil layers ① and ⑦.

The SPT N -value in each layer was used for estimating shear wave velocity V_s using Equation 1 specified in Japan Road Association (2012).

$$V_s \text{ (m/s)} = \begin{cases} 100N^{1/3} & \text{for clay} \\ 80N^{1/3} & \text{for sand} \end{cases} \quad (1)$$

Shear modulus G was calculated using Equation 2, in which g is the gravitational acceleration ($g = 9.8 \text{ m/s}^2$).

$$G = \frac{\gamma_t}{g} V_s^2 \quad (2)$$

Young's modulus E was calculated using Equation 3. In Equation 3, Poisson's ratio ν of each soil layer was assumed as shown in Table 2.

$$E = 2(1+\nu)G \quad (3)$$

Table 3 shows the parameters of the UBC3D-PLM model applied to soil layers ②, ③, ④, ⑤ and ⑥.

The relative density D_r was estimated by means of Equation 4, as proposed in Tokimatsu & Yoshimi (1983). In Equation 4, σ'_v is the effective vertical stress in kgf/cm^2 (equivalent to 98 kPa). ΔN_f is the increment of the N -value based on the fine grain content rate F_c specified in The Building Center of Japan (2019). The F_c values of the sandy silt layer ② and the silt layer ④ were obtained from soil test results (see Figure 4), while the F_c values of the other soil layers were assumed to be 10%, 15% and 5% for layers ③, ⑤ and ⑥, respectively.

$$D_r = 21 \sqrt{\frac{N}{\sigma'_v + 0.7} + \frac{\Delta N_f}{1.7}} \text{ (%) } \quad (4)$$

The constant volume friction angle ϕ_{cv} was calculated using Equation 5, as specified in Japan Road Association (2012). In Equation 5, σ'_v is the effective vertical stress in kPa.

$$\phi_{cv} = 4.8 \ln \left(\frac{170N}{\sigma'_v + 70} \right) + 21 \text{ (}^\circ\text{)} \quad (5)$$

The peak friction angle ϕ_p was calculated using Equation 6 following PLAXIS 3D Manuals (Bentley, 2022).

$$\phi_p = \phi_{cv} + \frac{(N_1)_{60}}{10} + \max \left(0; \frac{(N_1)_{60} - 15}{5} \right) \text{ (}^\circ\text{)} \quad (6)$$

The corrected SPT value $(N_1)_{60}$ was calculated using Equation 7 following PLAXIS 3D Manuals (Bentley, 2022).

$$(N_1)_{60} = \frac{D_r^2}{15} \quad (7)$$

The cohesion c of all soil layers was assumed to be 0. The other parameters of the UBC3D-PLM model, as listed

in Table 3, were either calculated or set following PLAXIS 3D Manuals (Bentley, 2022).

Table 4 shows the material parameters of concrete and soil improvement used for the TNF system. The parameters of the soil improvement of the TNF were estimated using Equations 1, 2 and 3 assuming an N -value of 20 and γ_t of 17 kN/m³.

4.2 Analysis results

Figure 11 shows the calculated settlement curves of the TNF at five points. Large settlements occurred during each earthquake. Settlements were largest in the 3rd EQ with

$SI=6$ -. It is seen that the earthquakes also caused differential settlements of the TNF.

The analysis of a case of no earthquake was also conducted to evaluate the influence of the earthquakes. The calculated settlement curve at point P5 of this case is added in Figure 11. It is seen that the earthquakes were the main cause of large settlements of the TNF.

Figure 12 shows the effective vertical stress σ'_v during the 1st EQ. The σ'_v of all soil layers of the ground decreased during the earthquake. At the end of the 1st EQ, the σ'_v of silty ground (layers ② and ④) and sandy ground (layer ③)

Table 1. Physical and mechanical properties of each soil layer.

Layer No.	Soil type	Soil model	t_1 (m)	t_2 (m)	SPT N	γ_t (kN/m ³)	e_0	k (m/day)
①	Fill	Linear elastic	0.65	1.19	2	17.00	1.0	8.64
②	Sandy silt	UBC3D-PLM	2.20	1.90	1	15.29	1.852	3.21×10^{-4}
③	Silty sand	UBC3D-PLM	1.00	0.80	4	17.00	0.9	8.64×10^{-3}
④	Silt	UBC3D-PLM	9.80	10.90	0~1	12.89	3.699	1.69×10^{-4}
⑤	Silty sand	UBC3D-PLM	0.80	0.30	5.5	17.00	0.9	8.64×10^{-3}
⑥	Silty sand	UBC3D-PLM	4.40	3.10	21.2	17.00	0.8	8.64×10^{-3}
⑦	Sandstone	Linear elastic	1.15	1.81	48	20.00	0.5	0

Legend: see List of symbols and abbreviations.

Table 2. Parameters of the linear elastic model applied to soil layers ① and ⑦.

Layer No.	Soil type	Soil model	SPT N	γ_t (kN/m ³)	V_s (m/s)	G (kPa)	ν	E (kPa)
①	Fill	Linear elastic	2	17.00	100.8	1.76×10^4	0.3	4.58×10^4
⑦	Sandstone	Linear elastic	48	20.00	363.4	26.95×10^4	0.2	64.69×10^4

Legend: see List of symbols and abbreviations.

Table 3. Parameters of the UBC3D-PLM model applied to soil layers ②, ③, ④, ⑤ and ⑥.

Layer No.	D_r (%)	ϕ_{cv} (°)	ϕ_p (°)	$(N_1)_{60}$	c (kPa)	k_B^c	k_G^c	k_G^p	R_f	m_c	n_c	n_p	σ_t (kPa)	f_{dens}	f_{Epost}
②	59.7	22.7	24.4	15.8	0	763	1090	920	0.73	0.5	0.5	0.4	0	1.0	1.0
③	54.1	29.3	30.6	13.0	0	714	1021	618	0.75	0.5	0.5	0.4	0	1.0	1.0
④	64.5	22.7	25.2	18.5	0	803	1148	1277	0.71	0.5	0.5	0.4	0	1.0	1.0
⑤	62.8	30.3	32.6	17.5	0	789	1127	1139	0.72	0.5	0.5	0.4	0	1.0	1.0
⑥	67.2	36.2	39.2	20.1	0	826	1179	1525	0.70	0.5	0.5	0.4	0	1.0	1.0

Legend: see List of symbols and abbreviations.

Table 4. Material parameters of concrete and soil improvement used for the TNF system.

Material	Soil model	γ_t (kN/m ³)	V_s (m/s)	G (kPa)	ν	E (kPa)
Concrete (Slab, Footing)	Linear elastic	24	2001	9.79×10^6	0.2	23.50×10^6
Soil improvement	Linear elastic	17	271	1.28×10^5	0.25	3.20×10^5

Legend: see List of symbols and abbreviations.

decreased to nearly 0 kPa. It is seen that liquefaction occurs even in the silty ground.

Figure 13 shows the excess pore water pressure p_{excess} during the 1st EQ. The p_{excess} of the whole ground increased during the earthquake, and the p_{excess} at the end of the 1st EQ was much larger than those before the earthquake, especially in silty ground (layers ② and ④). The p_{excess} at the end of the 1st EQ caused consolidation settlements of the TNF and the surrounding ground after the earthquake as shown in Figure 11.

Figure 14 compares the calculated settlements and the measured settlements of the TNF on 2017-03-07, 2020-02-03 and 2021-05-20, respectively. As shown in Figure 14a, the

calculated settlements closely match the measured settlements at the four corners of the foundation (at the measurement points). As shown in Figure 14b, although the calculated settlements are smaller than the measured settlements, the distributions of calculated settlements align well with the distributions of measured settlements. In Figure 14c, although the number of measurement points is less, the trend of the distribution of the calculated results remain consistent with that of the measured results.

Figure 15 compares the calculated relative settlements and the measured relative settlements of the TNF on 2017-03-07, 2020-02-03 and 2021-05-20, respectively. In Figure 15a, comparing the distribution of relative settlements is difficult due to the measured settlement data are only available at the four

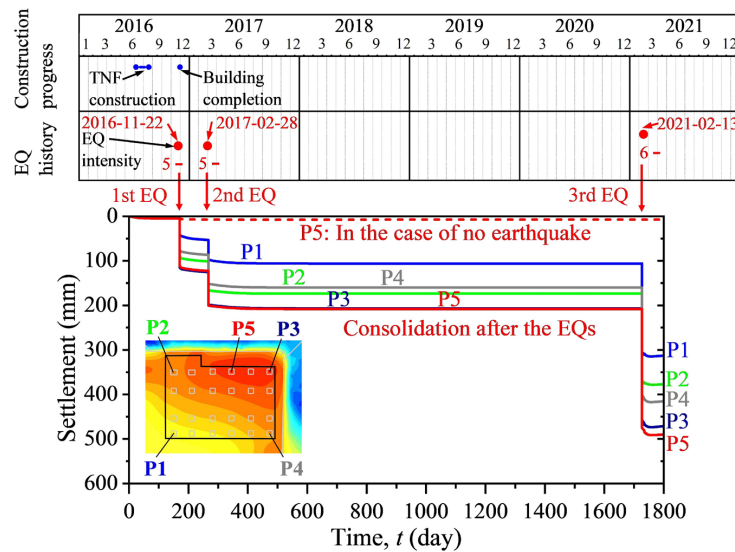


Figure 11. Calculated settlement curves of the TNF.

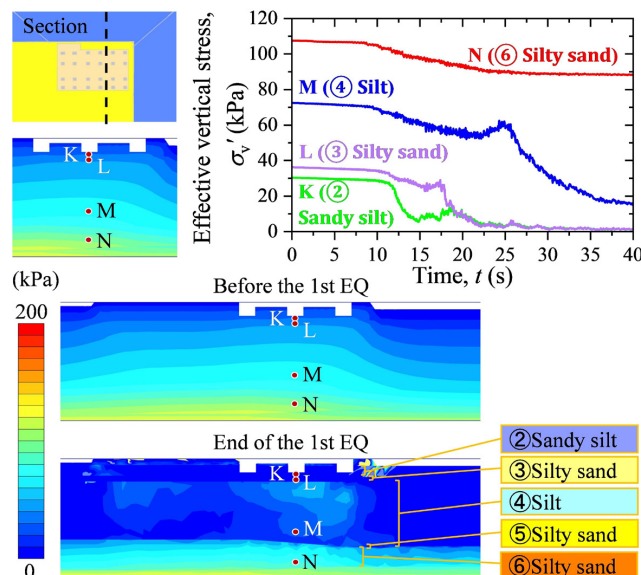


Figure 12. Effective vertical stress during the 1st EQ.

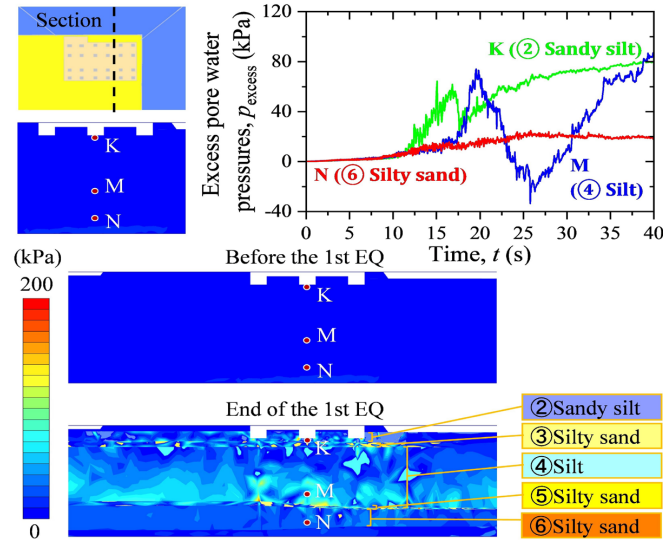


Figure 13. Excess pore water pressure during the 1st EQ.

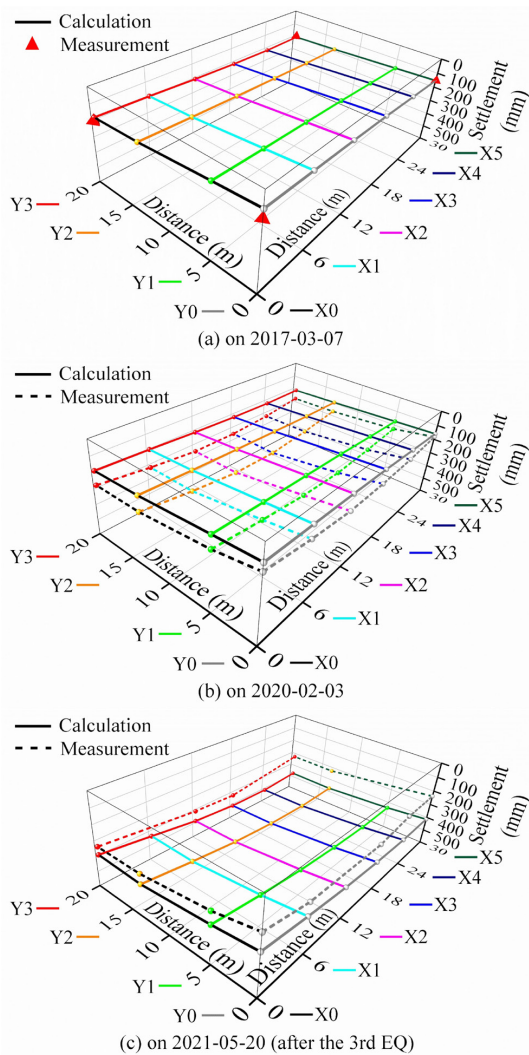


Figure 14. Comparison of calculated and measured settlements of the TNF.

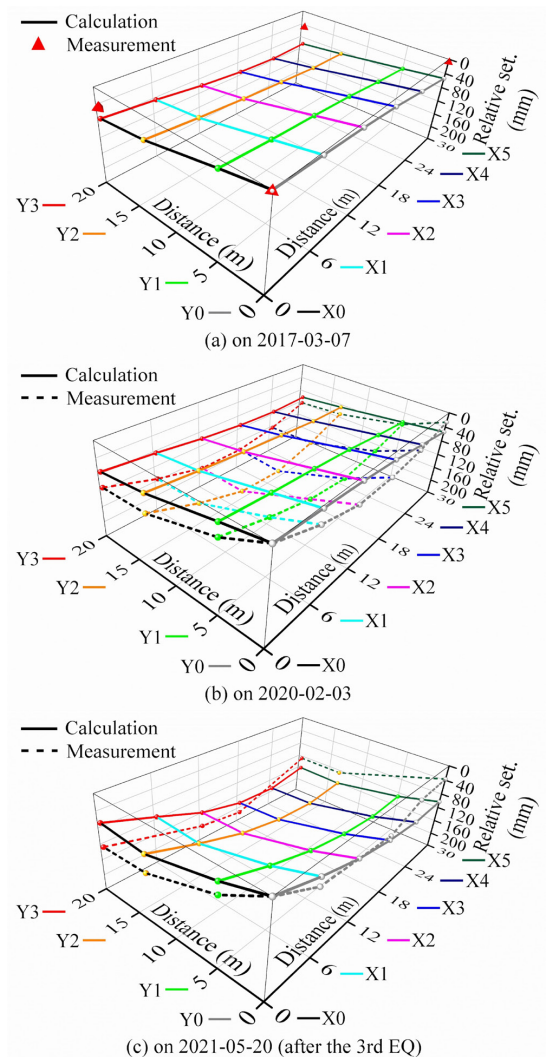


Figure 15. Comparison of calculated and measured relative settlements of the TNF.

corners of the foundation. However, in Figures 15b and 15c, the distributions of relative settlements in the measurement and the calculation are nearly the same. Hence, the calculation simulated well the measured settlements.

Figure 16 shows the comparison of the calculated and measured relative settlements on 2021-05-20 (after the 3rd EQ). It is seen that the calculation simulated the measured settlements well. Along the Y-axis direction (Figure 16a), the maximum settlement occurred at the center points (X3 axis) in both the measurement and the calculation, and the settlement gradually decreased toward the edges. Along the X-axis direction (Figure 16b), the calculated maximum settlements occurred between at the Y2 axis and Y3 axis. Along X0 axis and X5 axis, similar results were measured. Although there were only two measurement points along the X3 axis, it is inferred that a similar trend occurred.

To demonstrate the efficient performance of the TNF for suppressing the settlements induced by liquefaction, a

similar analysis of a conventional shallow foundation shown in Figure 17 was conducted.

Figure 18 shows the comparison of the calculated settlement curves at point P5 between the TNF and the conventional shallow foundation. It is seen that the settlement of the TNF is suppressed compared to the settlement of the conventional shallow foundation, especially in the 3rd EQ ($SI = 6$).

Figure 19 shows the comparison of the distribution of calculated relative settlements between the TNF system and the conventional shallow foundation on 2021-05-20 (after the 3rd EQ). It is seen that the differential settlement in the case of the TNF is much suppressed compared to the conventional shallow foundation. This demonstrates the advantages of the TNF for suppressing the settlements induced by liquefaction, highlighting its advantages over conventional foundation methods.

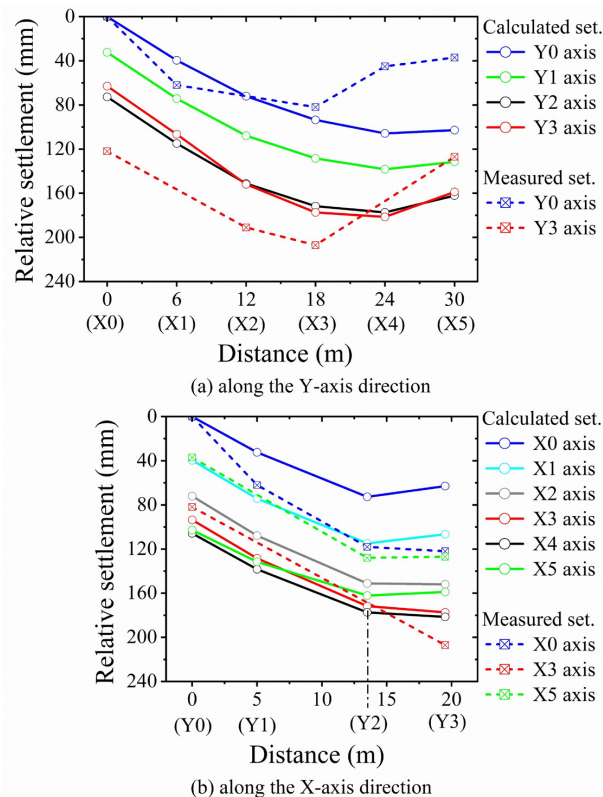


Figure 16. Comparison of calculated settlements and measured settlements on 2021-05-20 (after the 3rd EQ).

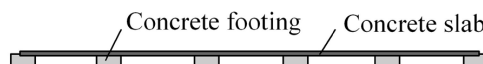


Figure 17. Section view of a conventional shallow foundation.

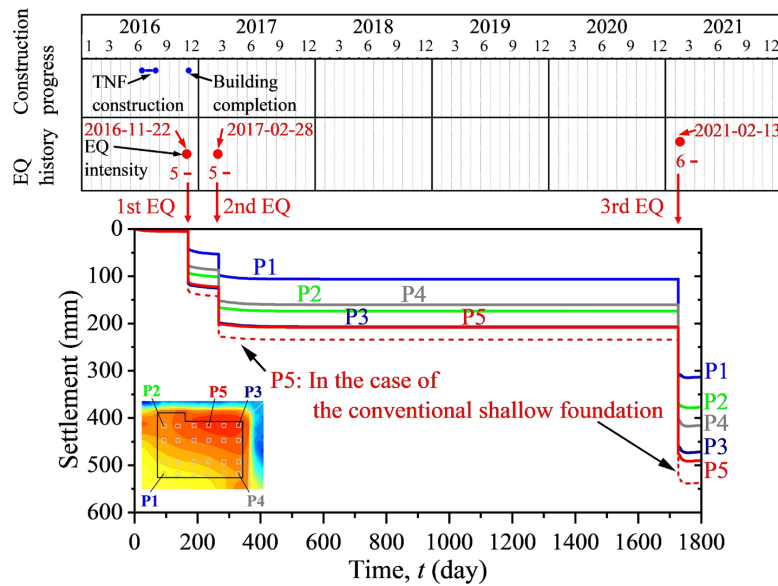


Figure 18. Comparison of calculated settlement curves between the TNF and the conventional shallow foundation.

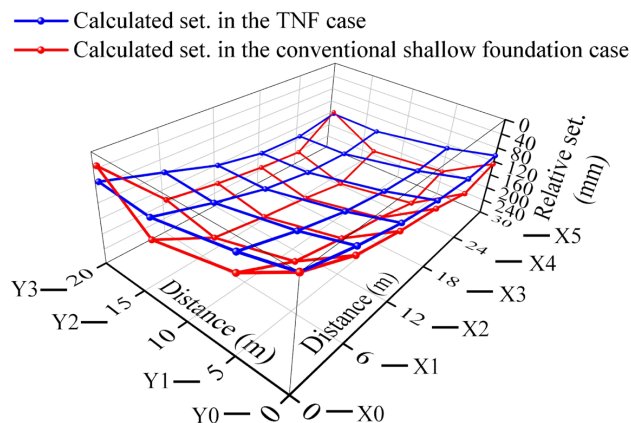


Figure 19. Comparison of the distribution of calculated relative settlements between the TNF system and the conventional shallow foundation on 2021-05-20 (after the 3rd EQ).

5. Conclusions

In this research, the dynamic analysis of the actual TNF on the silty ground subjected to three consecutive earthquakes was conducted. The responses of the TNF as well as the ground during and after the earthquakes were calculated to investigate the occurrence of liquefaction in the silty ground. Comparisons of the calculated settlements and the measured settlements after the second and third earthquakes were conducted. An analysis of a case of no earthquake was also conducted to investigate the influence of the earthquakes. Furthermore, a similar dynamic analysis of a conventional shallow foundation was also carried out

to compare its performance in suppressing the settlements induced by liquefaction with the TNF's performance. From comparisons between the measured and calculated results, the main conclusions are drawn as follows:

1. The dynamic calculation of the TNF simulated the measured settlements well;
2. Earthquakes were the main cause of settlements and differential settlements of the TNF in this project, because settlements of the TNF from the analysis of the case of no earthquake were much less than those caused by the earthquakes;
3. Liquefaction occurred even in the silty ground;
4. The TNF is a more efficient foundation for suppressing the settlements induced by liquefaction compared to the conventional shallow foundation.

In the next step, a sensitivity analysis would be needed to investigate the influences of soil parameters on settlement results, enhancing the practical value of this research.

Declaration of interest

The authors have no conflicts of interest to declare. All co-authors have observed and affirmed the contents of the paper and there is no financial interest to report.

Authors' contributions

Han Vo-Cong: formal analysis, methodology, visualization, writing – original draft. Kinji Takeuchi: conceptualization, resources. Yasuo Tomono: project administration, supervision, investigation. Tatsunori Matsumoto: conceptualization, supervision, validation.

Data availability

No dataset was generated or evaluated in the course of the current study; therefore, data sharing is not applicable.

Declaration of use of generative artificial intelligence

This work was prepared without the assistance of generative artificial intelligence (GenAI) tools.

List of symbols and abbreviations

a_x	Acceleration into x-direction
a_y	Acceleration into y-direction
c	Cohesion
e_0	Initial void ratio
f_{dens}	Densification factor
f_{Epost}	Post-liquefaction factor
g	Gravitational acceleration
k	Coefficient of permeability
k_B^e	Elastic bulk modulus factor
k_G^e	Elastic shear modulus factor
k_G^p	Plastic shear modulus factor
m_e	Rate of stress-dependency of elastic bulk modulus
n_e	Rate of stress-dependency of elastic shear modulus
n_p	Rate of stress-dependency of plastic shear modulus
p_{excess}	Excess pore water pressure
q_u	Unconfined compression strength
t_1	Thickness of soil layer in Borehole No. 1
t_2	Thickness of soil layer in Borehole No. 2
w_{ave}	Average settlement
BH	Borehole
D_r	Relative density
E	Young's modulus
EQ	Earthquake
F_c	Fine grain content rate
FEM	Finite Element Method
G	Shear modulus
JMA	Japan Meteorological Agency
$(N_1)_{60}$	Corrected SPT value
R_f	Failure ratio
SI	Seismic Intensity scale
SPT	Standard Penetration Test
TNF	Tender Net Foundation

V_s	Shear wave velocity
γ_t	Wet unit weight
ν	Poisson's ratio
σ_t	Tension cut-off
σ_v'	Effective vertical stress
ϕ_{cv}	Constant volume friction angle
ϕ_p	Peak friction angle
ΔN_f	Increment of the N -value
Δw	Maximum differential settlement

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REVIEW ARTICLE

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Seepage in a flood protection levee – first geotechnical centrifuge test results

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Review Article

Keywords

Centrifuge modelling of a flood protection levee
Transient seepage flow
Numerical modelling of transient seepage flow

Abstract

Many flood protection levees in Europe were built more than 100 years ago. These levees often do not meet the current flood protection requirements due to increased level of safety requirements, higher damage potential in the valley plains and due to higher peak discharges or water levels expected with changing climatic conditions. A first series of centrifuge tests on two idealized cross-sections of the river Rhine flood protection levee have been carried out in the geotechnical centrifuge at Delft University of Technology in order to study the transient seepage behaviour of a horizontally layered levee consisting of layers with coarse and fine-grained material. Main features and design considerations of a specially manufactured flood simulator for the geotechnical centrifuge which allows replicating scaled flood events with predefined durations and intensities are presented. Furthermore, measured values of the pore pressure during the investigated flood event are reported and discussed in comparison to the results of finite element modelling of the levee. Finally, the potential impacts of the hydraulic boundary conditions on the seepage behaviour in the physical and numerical models of the levee are critically discussed.

1. Introduction

Many existing flood protection levees no longer satisfy current safety standards, particularly in the context of climate change. The increasing frequency and intensity of heavy precipitation, together with the rising zero-degree isotherm in alpine regions, leads to more severe flood events characterized by higher water levels (NCCS, 2021). This is illustrated by the example of the flood protection levees along the Alpine Rhine, which are currently being restored in sections. The original straightening and damming of the Rhine started in the 19th century (Zur Rhein-Hochwasserkatastrophe, 1927) and no longer fulfils current flood protection requirements (Tschanz, 2018).

A conservative engineering design approach assumes steady-state seepage flow through the levee and is often considered when investigating levee stability or developing measures against stability problems (CIRIA, 2013). However, it seems reasonable to consider transient seepage conditions, as flood events in alpine regions often only have a duration of 12 to 48 h (Aschwanden & Schädler, 1988). This could offer a potential for cost optimisation in project planning of remediation measures. Research results on a model river-embankment consisting of silty-sandy material in a geotechnical centrifuge by Giretti et al. (2022) show that the saturation

front in the levee reaches the air-side of the levee only after an unrealistic long flood period of more than 100 days. Mayor (2013) reports similar results in his large-scale tests at the river Rhône levee consisting of silty-sandy material. Even an eleven-day long flood was not enough for the saturation front to reach the air-side of the levee. With further tests, however, Mayor showed that the time until the saturation front reaches the air-side of the levee depends directly on its permeability and its initial degree of saturation.

However, if transient flow conditions are to be considered in the design, a more detailed understanding of the evolution of pore water pressure in the levee body is required to ensure safe design. Physical modelling under transient flow conditions can provide valuable data for better interpretation and potentially validation of numerical calculation methods.

A model-levee is used because the construction of a 1:1 scale model levee is complex and costly (Mayor, 2013; Toromanovic et al., 2020). A geotechnical centrifuge is used to increase the acceleration field and thus the effective stresses in the model to ensure comparable results of the model tests with the prototype (Madabhushi, 2014). A geometrically N -times smaller levee is subjected to an N -times larger g -level as the effective stresses are generally accepted to be linearly correlated with depth (Wood, 2004; Askarinejad et al., 2015).

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Another advantage of centrifuge modelling is the scaling of time for seepage processes. Assuming the intrinsic permeability does not differ between model and prototype and that water is used as the saturating fluid in the model, it can be shown that the seepage velocity v in a centrifuge model (index m) is N -times faster than in the prototype (index p) (Thusyanthan & Madabhushi, 2003):

$$v_m = N v_p \quad (1)$$

Consequently, it can be shown that the seepage process is N^2 -times faster in the centrifuge model (Garnier et al., 2007) compared to the prototype, as the length of the seepage path l in the model is N -times smaller:

$$t_m = \frac{l_m}{v_m} = \frac{\frac{1}{N} l_p}{N v_p} = \frac{1}{N^2} t_p \quad (2)$$

This means that at a g -level of $N = 100$ a prototype flood with a duration of $t_p = 72$ hours is modelled within $t_m = 25.9$ seconds.

2. Method

In this study, the investigation of flow behaviour within a layered flood levee is primarily conducted through physical centrifuge modelling. Subsequently, numerical modelling was performed for one cross-section - with the same level of details and complexity similar to common engineering practice - to complement the experimental findings through a comparative assessment.

2.1 Centrifuge modelling

The experiments were conducted at $100g$ in the geotechnical centrifuge at the Delft University of Technology which has a radius of 1.22 m and a basket size of 400x500x500 mm (Allersma, 1994; Zhang & Askarinejad, 2021). A flood simulator was designed and constructed for this purpose. A flood wave could be generated inflight with the help of a water tank and servo coupled valves. In addition, a constant water level was maintained at the downstream side of the levee. The flood simulator has outer dimensions of $L \times H \times W = 615 \times 200 \times 120$ mm, with 15 mm thick aluminium panels and a transparent side wall made of 15 mm thick acrylic glass. Figure 1 shows a model levee installed in the flood simulator in the centrifuge basket. The water tank for the flood wave (1.6 litres) is situated on top of the flood simulator.

The model levee was constructed according to the prototype of two typical cross-sections of the river Rhine levee (Figure 2). On top of the permeable Rhine gravel lies a less permeable layer of flood deposits on which the

gravelly levees were built (cross-section A as given in prototype scale in Figure 2). Field investigations revealed that the less permeable layers of flood deposits below the levee do not always extend to the water side. Therefore, a second cross-section B was investigated in which this layer only extends to approximately below the crest of the levee (see Figure 2). In the following, cross-section A is referred to as levee A and cross-section B as levee B.

The model levees were constructed at a scale of 1:100 as shown for levee A in Figure 3. The crest of the levee is 80 mm higher than the air-side terrain and has a base width of approximately 350 mm. It is important that the underlying permeable Rhine gravel is modelled as well since it may influence the seepage behaviour. Geotechnical properties of the materials used in the model are summarised in Table 1,

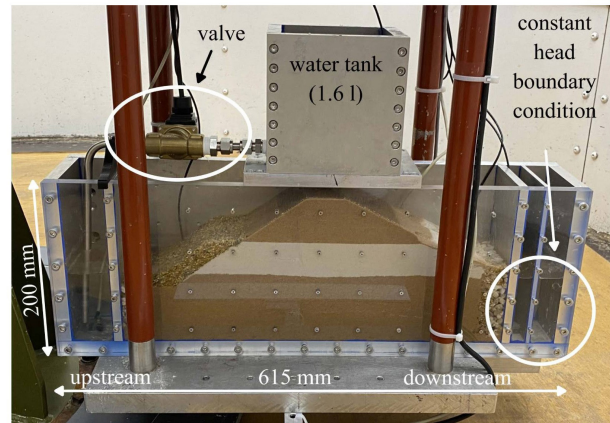


Figure 1. Model levee A inside the flood simulator.

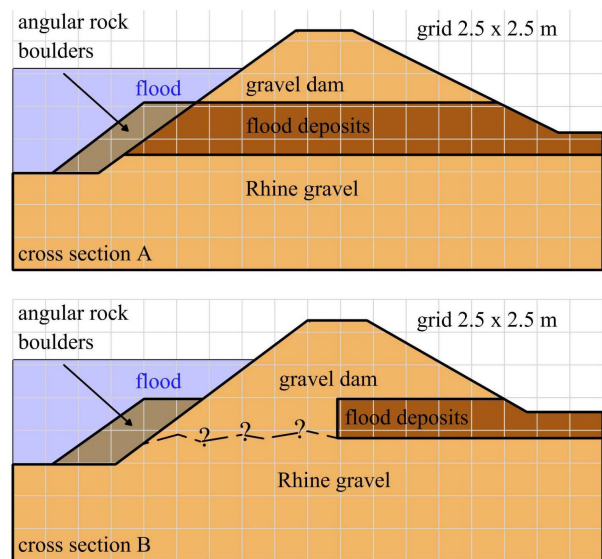


Figure 2. Typical cross sections (A & B) of the river Rhine levee in prototype scale.

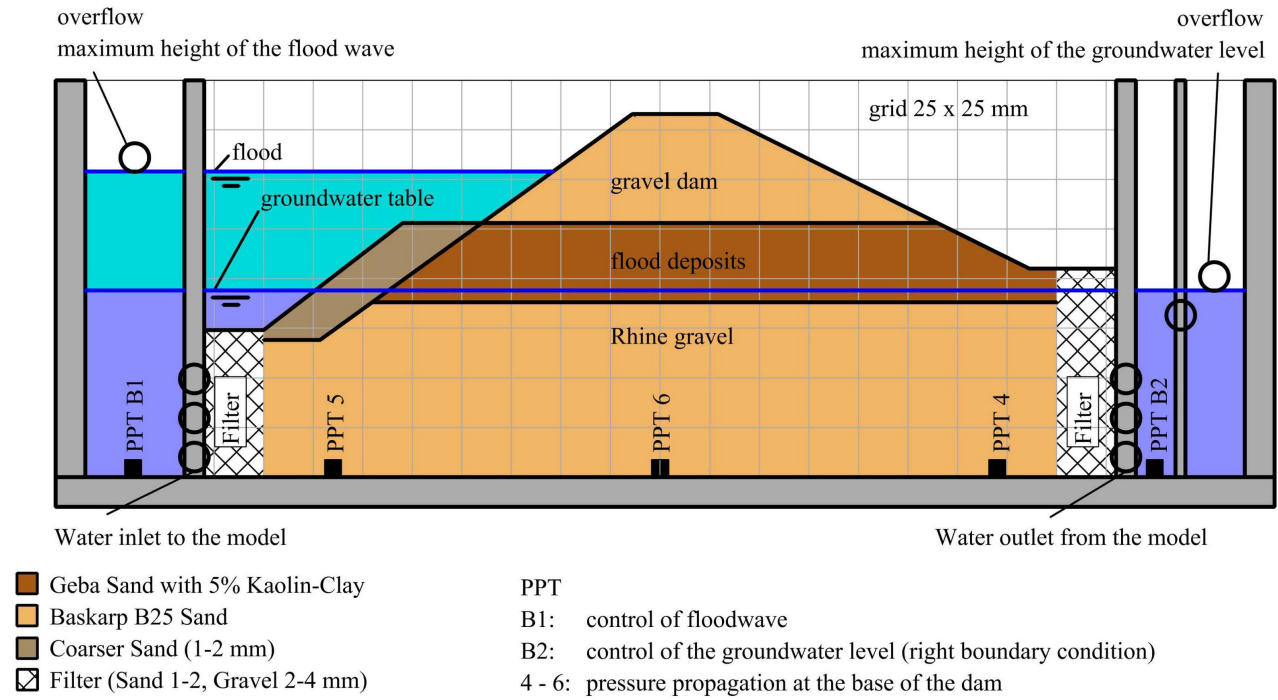


Figure 3. Geometry of levee A built into the flood simulator including positions of PPTs. Dimensions in [mm].

Table 1. Geotechnical properties of the levee materials used in the centrifuge test.

Soil type	$e^{(1)}$ [-]	k_v [m/s]	ϕ'_{cv} [°]
Baskarp B25 Sand	0.69	$(4.6 - 5.8) \cdot 10^{-5}$	32 - 34
Geba Sand with 5% Kaolin Clay	0.64	$(2.3 - 2.6) \cdot 10^{-6}$	$\sim 31^{(2)}$
Geba Sand with 10% Kaolin Clay	0.56	$(4.0 - 5.8) \cdot 10^{-7}$	$\sim 31^{(2)}$

⁽¹⁾ voids ratio after compaction at $w = 10\%$. ⁽²⁾ value applies for Geba Sand without clay.

where e is the voids ratio, k_v is the corresponding coefficient of the vertical permeability and ϕ'_{cv} the critical state angle of friction. In prototype, the Rhine gravel and the Rhine levee have a similar permeability, so both materials are modelled with Baskarp B25 sand (Pol et al., 2021). For the less permeable layer of flood deposits a mixture of Geba Sand (Maghsoudloo et al., 2021) with 5% Kaolin Clay for levee A and 10% Kaolin Clay for levee B was used. The permeabilities were determined with constant head tests (Head & Epps, 2011). The values determined in the tests correspond to the vertical permeability k_v .

The soil materials were placed in the flood simulator in layers with a water content w of 10% and compacted uniformly. At the same time, a two-stage filter made of coarser material was installed at the up- and downstream boundary of the model. The filter is made of gravel 2-4 mm and sand 1-2 mm and is therefore significantly coarser than the Baskarp B25 sand with a grain size d_{70} of 0.26 mm (Pol et al., 2021).

After the entire flood simulator was filled, the levee was shaped by removing the excess material. At the same time, samples were taken to check the layer density.

Next, the river-side berm, which in the prototype is made of angular rock boulders as an erosion protection at the base of the levee (Figure 2), was constructed in the model. For this purpose, the berm was built up from coarse grained 1-2 mm sand, see Figure 4.

Three Pore water Pressure Transducers (PPTs) were installed at the bottom of the model (Figure 3) to record the propagation of pore water pressure in the levee. To check the boundary conditions, two PPTs were installed in the upstream inlet tank and in the downstream outlet tank, respectively.

Figure 5 shows the planned flood characteristics in the prototype scale for the tests. The water levels refer to the heights from the base of the flood simulator (Figure 3) and are given in prototype scale hereafter, with the corresponding model scale in parentheses: The riverbed accordingly lies at a height of 7.4 m (74 mm). The goal was to model a flood wave consisting of a pre-wave of 13 m (130 mm), a flood peak of 15.4 m (154 mm) and a post-wave of 12.3 m (123 mm) by opening and closing the valves, starting from an initial river level at 9.4 m (94 mm) (steady state with 2 m (20 mm) high river, see Figure 3). Water was used as the fluid for the centrifuge experiments. The model time for seepage processes is therefore reduced by a factor of $1:100^2$ according to Equation 2, since the model was tested at 100 g. This means that the entire flood event with a duration of 60 h at prototype scale translates to 21.6 s in the centrifuge

experiment. The experiments were recorded through the acrylic glass using a digital camera.

The test procedure is described hereafter with the corresponding prototype time steps, model time is given

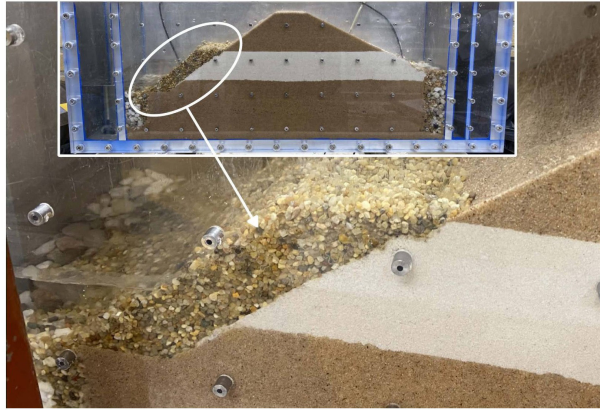


Figure 4. Upstream berm made from coarse grained 1-2 mm sand.

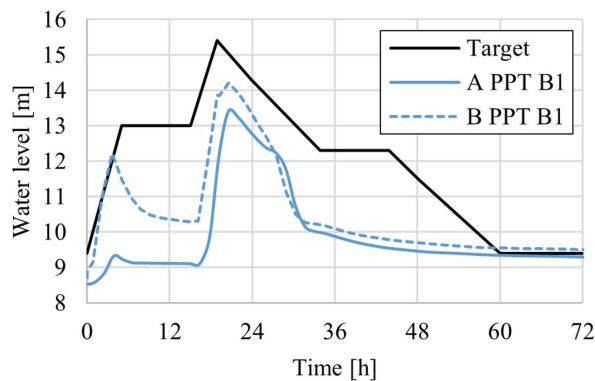


Figure 5. Target hydrograph of the flood wave in prototype scale compared to the actual measurements of PPT B1 achieved during test for levees A and B (riverbed is located at 7.4 m).

in parentheses: After the centrifuge reaches 100 g ($t = 0$), the initial water level of 9.4 m (94 mm) is kept constant for 14 d (2 min) to allow stationary conditions to be established. Subsequently, the flood wave is applied stepwise within 60 h (21.6 s) as shown in Figure 5. Finally, the model is kept under 100 g for another 35 d (5 min) to restore initial conditions.

After the centrifuge test, the model is checked for any signs of erosion, piping or stability problems. However, the desired flood wave could not be achieved because some of the valves could not be opened or closed properly at 100 g (Figure 5). Thus, it was difficult to maintain a constant water level with the water available in the tank due to the loss of water through seepage.

2.2 Comparative numerical model

A comparative numerical model of the levee A is developed at prototype scale in the program PlaxFlow2D (PLAXIS CONNECT Edition V20.04.00, 2020) with the same boundary conditions as given in the experiment. Special attention was paid to the following points:

- The flood wave is modelled according to the data measured in the centrifuge test (i.e., PPT B1 hydrograph in Figure 5).
- The permeable berm on the water side made of coarse sand is not modelled to minimise the computational effort. It is assumed that due to the higher permeability, it does not significantly influence the seepage behaviour.
- A constant head boundary condition is set on the air-side as given in the centrifuge model test.
- Figure 6 shows the FE model with the corresponding boundary conditions and the mesh. The initial water level of the river lies 2 meters above the riverbed and corresponds to the height of the constant head boundary condition on the right edge of the model (initially no hydraulic gradient). The variable water level is indicated in light blue on the left side of the levee.

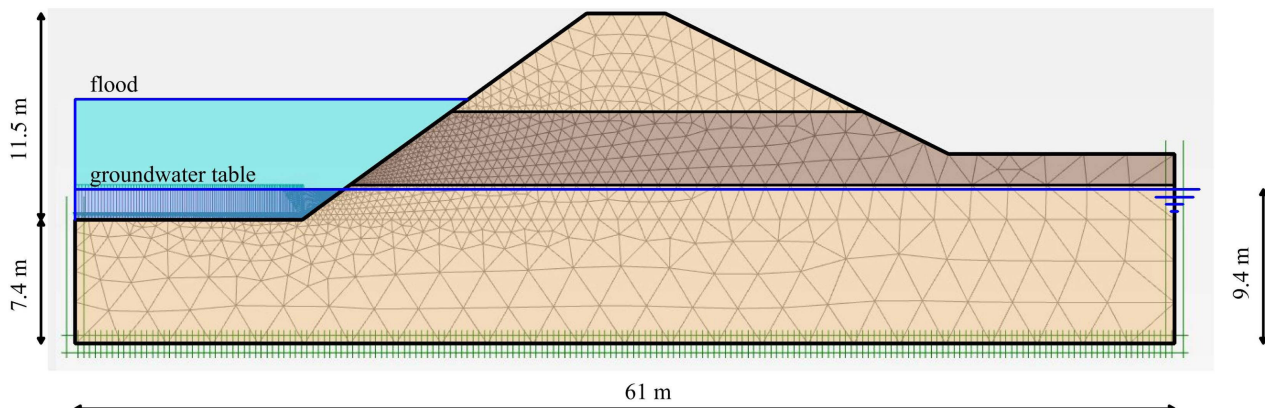


Figure 6. FE model geometry with mesh and boundary conditions for the PlaxFlow2D calculation.

The pressure conditions are compared at points that are always saturated (PPTs in Figure 3). However, it must be assumed that the modelling of the unsaturated levee area above has an influence on the results. For a comparative model, the unsaturated zones of the levee and their behaviour are described using the van Genuchten (1980) model implemented in the software. Thus, with the help of the dimensionless volumetric water content Θ (Equation 3), the relative hydraulic permeability K_r of partially saturated soils can be described (Equation 4).

$$\Theta = \frac{(\theta - \theta_r)}{(\theta_s - \theta_r)} \quad (3)$$

$$K_r(\Theta) = \Theta^{\frac{1}{2}} \left[1 - \left(1 - \Theta^{\frac{1}{m}} \right)^m \right]^2 \quad (4)$$

In Equation 3, θ describes the actual volumetric water content, θ_r is the residual and θ_s is the saturated volumetric water content. The parameter m is linked to the soil-water retention curve given in Equation 5 as $m = 1 - 1/n$:

$$\theta = \theta_r + \frac{(\theta_s - \theta_r)}{\left[1 + (\alpha \cdot h)^n \right]^{\frac{1}{m}}} \quad (5)$$

In Equation 5, α , n and m are model parameters describing the curvature of the water retention curve after van Genuchten (1980) and h is the pressure head (suction is positive). However, no soil-water retention curves were available for the materials used. To be able to carry out an approximate calculation and compare it to the centrifuge tests, parameter sets stored in PLAXIS were used, see Table 2. The parameter sets were selected on the basis of the particle size distribution in such a way that the numerical calculation models produced the best possible comparison to the centrifuge test results. Due to the chosen procedure, some general points must be emphasised:

- The calculations represent an initial estimate. The actual water retention curves and the associated parameters α and n may differ from those used in the present model.
- A difference between drying and wetting curves can be observed in reality (Benson et al., 2014). This is not taken into account in the numerical model.
- As there is no information on the ratio of saturated horizontal permeability k_h to vertical permeability k_v and a reliable estimate is difficult, no distinction was made.

3. Results

Both levees did not exhibit any stability problems during the centrifuge tests. Furthermore, it became evident that the entire model (i.e., the centrifuge basket) tilts slightly during the experiments, which results in a water level that is inclined relative to the flood simulator. This phenomenon occurs when the centre of gravity is not in the axis of the centrifuge basket's hinge. However, due to the flood wave and the seepage, the centre of gravity is constantly changing - which explains the phenomenon (the model was installed perpendicular to the flight direction to avoid curvature of the water surface). Thus, at the beginning of the test, the water table at PPT B1 was lower at about 8.4 m than at the end of the test at about 9.3 m (Figure 5).

Analysis of images from the centrifuge tests showed that the flood wave effectively acting on the levee body reached its maximum level almost one second later than in the flood tank where the measurement of PPT B1 was carried out (Figure 3). This delay is due to the fact that water needs to flow through the gravel filter and water inlet holes (3 holes with $d = 6$ mm; Figure 3). One second corresponds to a delay of approximately 2.7 hours in prototype scale. This delay is taken into account in the comparison with the comparative numerical model.

3.1 Comparison of centrifuge test results for levee A and B

Immediately after the tests, the air-side slopes of both levees were examined. A difference between levee A and B was observed. Seepage paths were able to form above the continuous flood deposit layer of levee A, which led to a water outflow at the air-side embankment, resulting in sand washouts and surface erosion (see Figure 7, top). However, this phenomenon could not be observed for levee B (see Figure 7, bottom). Figure 7 also shows probable flow paths. For levee B in particular, the dark discolouration of the Baskarp B25 Sand indicates a change in the degree of saturation, although it is not possible to determine whether the dark sand is fully saturated. In contrast, no change in colour could be detected in the lighter-coloured Geba Sand Kaolin Clay mixture with the existing resolution.

Table 2. Input parameters for the transient seepage analysis.

Parameter	Geba Sand with 5% Kaolin Clay	Baskarp B25 Sand
$k_h = k_v$ [m/s]	$2.3 \cdot 10^{-6}$	$5.0 \cdot 10^{-5}$
e_{init} [-]	0.64	0.69
θ_r [-]	0.01	0.01
θ_s [-]	0.34	0.32
α [1/m]	1.7	5.21
n [-]	1.717	2.374

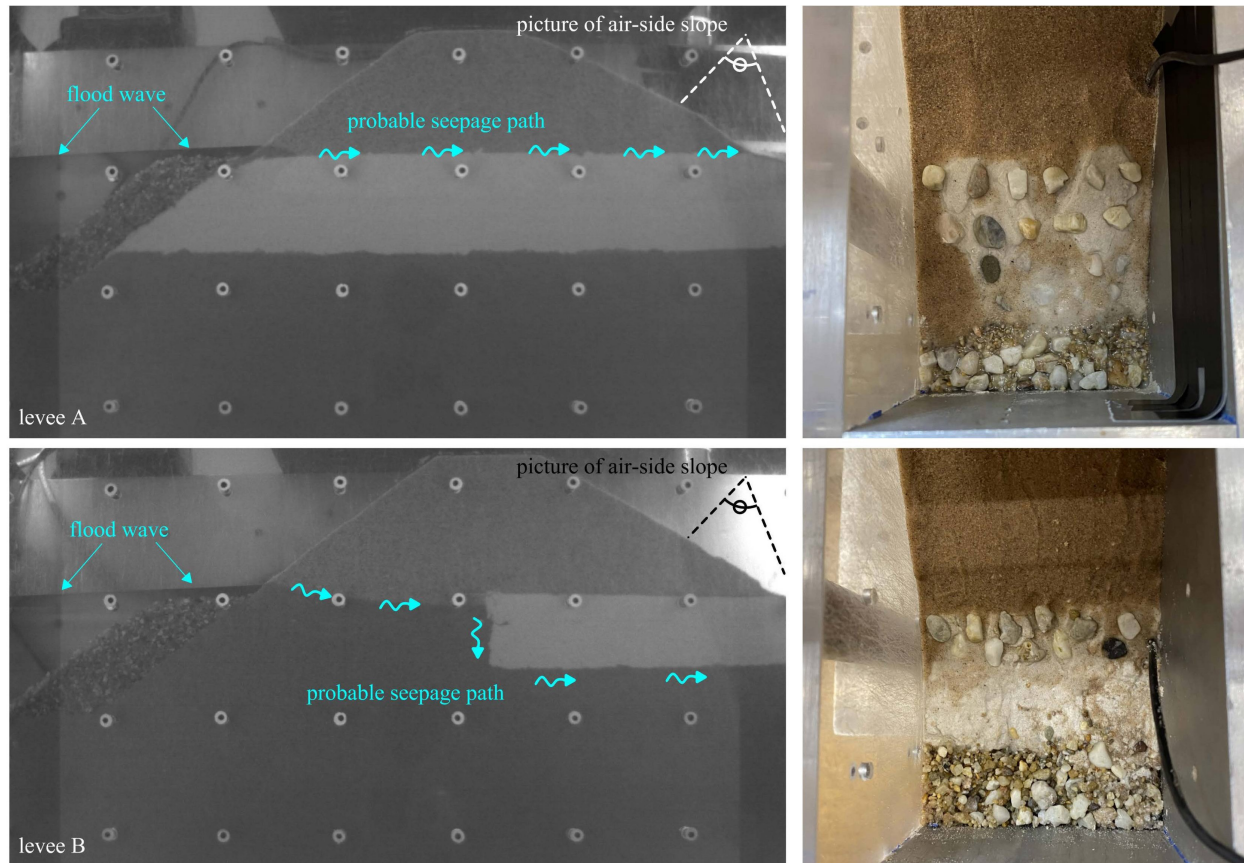


Figure 7. Left: Image of levee A (top) and B (bottom) during the centrifuge test, approximately at the flood peak. Right: Air-side slopes after the tests.

PPT B1 shows that the flood wave for levee B is approximately one meter higher than for levee A, see Figure 5. The pre-wave is also much more pronounced for Levee B. In order to be able to directly compare the two levee cross-sections, the PPT measurements are normalised in relation to the maximum value of PPT B1 in Figure 8. The initial letter stands for the corresponding levee cross-section (A PPT B1 for the measurement of PPT B1 at levee cross-section A). The time is given at prototype scale. The following observations from the comparison of the two centrifuge model tests should be pointed out:

- There is a significant damping of the flood wave upon entering the levee. PPT 5 reaches approximately 85% of the peak of PPT B1 with a distinct delay in time.
- The damping of the flood wave in relation to the maximum pressure head at PPT B1 is practically identical in the two cross-sections. For example, PPT6 in the centre of the levee shows for both levees a pressure head of approximately 3/4 of the maximum flood wave.
- The development of the pore water pressure over time, however, appears to be different in the two cross-sections. The pressure peaks are reached faster in cross-section B compared to cross-section

A. However, it must be emphasised that not only the cross-section is different, but also the saturation of the levee fill during the main flood wave is probably greater in cross-section B due to the larger pre-wave.

3.2 Comparison of centrifuge test results for levee A to a comparative numerical model

Results obtained from the centrifuge test for levee A are compared to the results of a comparative numerical model (section 2.2). As previously outlined, it is necessary to add a time increment of 2.7 hours to the measurement of A PPT B1 in order to model the flood wave acting on the levee (A PPT B1 + 2.7 h is designated as ‘flood’ in Figure 9). By comparing the pressure changes from the centrifuge test with those from the comparative numerical model (FEM in Figure 9), the following observations can be made: The infiltration behaviour of the numerical model differs from the physical model. For the latter, it is noticeable that there is a significant damping of the flood wave upon entering the levee (see PPT5) and the propagation of the flood wave within the levee is significantly slower on the water side than towards the air-side. This becomes clear by comparing the shifting of the pressure peaks over time.

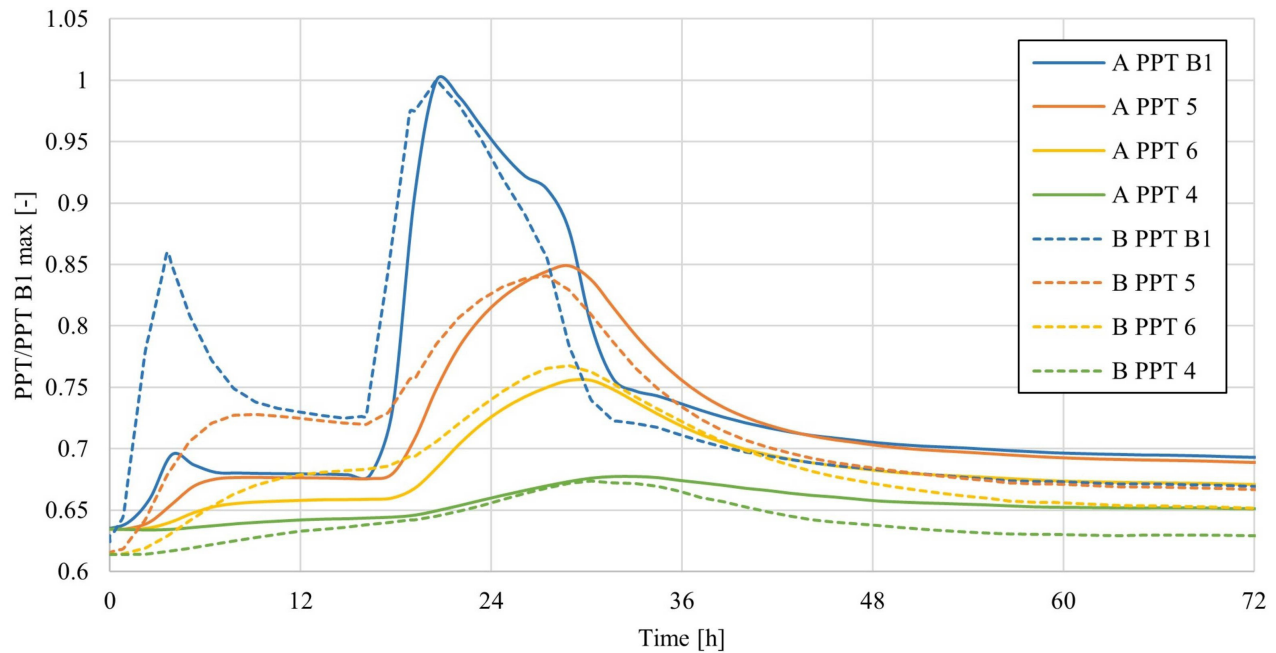


Figure 8. Comparison of normalised PPT measurements of levee A and levee B (time in prototype scale).

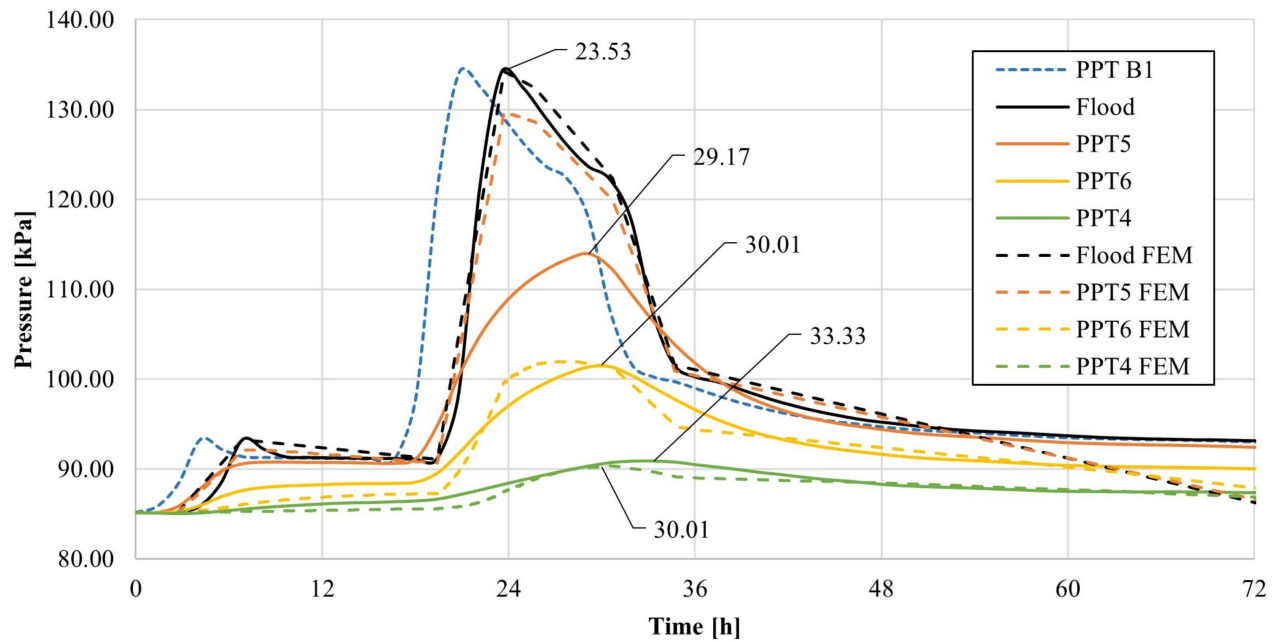


Figure 9. Comparison of PPT measurements of levee A and the FEM simulation (time in prototype scale).

The delay between the flood wave and PPT5 is about 5.6 hours, whereas it is only about 3.3 hours between PPT6 and PPT4. The numerical simulation, on the other hand, shows almost no damping for PPT5.

Nonetheless, PPT6 and PPT4 show relatively good agreement at least in the damping of the pressure magnitude.

In the centrifuge test it took about 10 hours to reach the maximum pore pressure at the air-side of the levee (peak of PPT4 compared to peak of 'flood'), whereas in the numerical model there are only about 7 hours in between. Furthermore, all pore water pressures almost reach the initial value again at the end of the numerical simulation.

4. Discussion and conclusions

A comparison of the behaviour of two different cross-sections in the centrifuge model (Figure 8) shows that the damping magnitude of the flood wave does not differ significantly with different soil stratification. This result is worth mentioning because different flow paths are formed in the two models. Based on the test data of pore water pressure increase over time, it cannot be clearly determined whether the faster propagation of the flood wave through levee B is primarily due to the fact that the saturation of the levee fill material and thus the permeability is higher, or whether it is due to the different soil stratification.

Comparing the test results of the physical model test to those of the comparative FEM simulation (Figure 9), a difference in time of rise and magnitude of the pore water pressure due to high water levels could be observed. The largest deviations could be found on the water side of the levee. The numerical model overestimates the speed at which the flood wave infiltrates the levee and underestimates the damping of the pore water pressures on the water side. The difficulties in predicting the pore water pressure distribution and the development of the saturation front in levees were already pointed out by Vaughan (1994) in his Rankine lecture, since most models do not take stress dependent permeability in terms of overburden pressure and heterogeneity of the material into account. In addition, there are considerable uncertainties in soil permeability measurements (anisotropy etc.). However, the behaviour observed in the physical model compares well to the numerical analysis considering the large scaling factor for time of $t_p = 100^2 t_m$ (Equation 2).

Regarding the question of whether transient seepage conditions for the design or assessment of flood protection levees should be considered and what benefit the results of centrifuge model tests in this context have, the following aspects should be pointed out:

- Centrifuge model tests and comparative numerical simulations lead to a better understanding of the levee behaviour. The well controllable test conditions in the centrifuge models serve to assess the quality of numerical simulations.
- The transfer of the present findings to the prototype scale poses a number of challenges to project engineers. For example, major uncertainties lie not only in the investigation and variability of relevant geotechnical parameters (water retention curve, permeability and anisotropy, friction angle, etc.) but also in the description of the initial state in terms of levee saturation, to which great importance must be given when analysing transient seepage conditions (Mayor, 2013). However, the initial degree of levee saturation is generally unknown in practice.

A further topic to be addressed is the influence of the hydraulic boundary conditions, as it is reasonable to assume, that the type of the air-side boundary condition not only has an influence on the levee stability, but also on the overall seepage behaviour. This is particularly relevant as the centrifuge tests are intended to represent the conditions within the prototype levee in order to more realistically portray the behaviour under flood conditions. The tests presented in this paper are based on hydraulic boundary conditions on the air-side of the levee corresponding to a constant hydrostatic water pressure (Figure 10 a).

If the levee was constructed on a widespread fine-grained layer, as it is the situation in parts of the Alpine Rhine, some excess pore water pressures may develop beneath the bottom of the levee on the air-side during a flood. As an extreme case, there could be a vertical impermeable boundary such as a construction pit closure in the levee footing area, see Figure 10 b. The pore water pressure distribution beneath the bottom of the levee on the air-side therefore depends directly on the permeability and the distance of the air-side boundary condition (this applies to physical and numerical models as well as to prototypes) and must be carefully chosen.

The following observations may be considered to improve the results from further experiments:

- More PPTs should be installed on the upstream side of the levee to better quantify the flood wave propagation into the levee close to the riverbed. This is where the largest differences between centrifuge tests and numerical simulation have been observed. An additional PPT should also be installed at the level of the riverbed above the gravel filter to record the actual flood wave acting on the levee.
- Attention should be paid to the boundary condition on the upstream side. To enhance control of water flow, it is

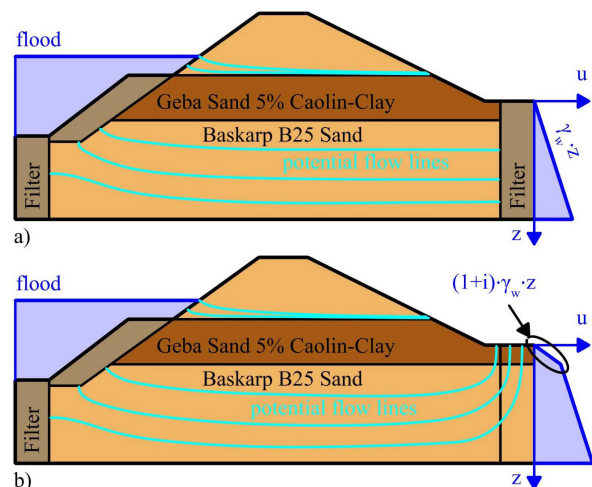


Figure 10. Variation of the boundary conditions on the air-side of the levee and their potential influence: a) constant head boundary as modelled in the first test series; b) impermeable boundary – excess pore water pressure assumed below the interlayer as a result.

recommended to maintain full permeability between the inlet water tank and the model compartment by drilling additional holes. It is also recommended to install an additional impermeable foil between the gravel filter and the levee base (i.e. Rhine gravel in the prototype, see Figure 3), given that flood loading in the prototype is predominantly characterised by inflow from the river bed and not by horizontal inflow to the base of the levee.

- The initial degree of saturation of the levee above the seepage line is unknown - but this significantly influences the infiltration time (Mayor, 2013). The installation of tensiometers to measure suction could therefore be advantageous (Askarinejad et al., 2018; Giretti et al., 2022).
- The intended flood wave, which is inspired by real flood events, could not be achieved due to the complexity of the test control. If the flood wave characteristics are to be simplified for further tests, possible saturation changes of the levee before the flood should be considered.
- Although the comparison of numerical simulation results and centrifuge model tests shows good agreement (with one exception mentioned above), additional laboratory tests should be carried out to investigate soil behaviour in more detail. In this context, two aspects of soil behaviour are of particular interest: i) Anisotropic permeability: isotropic permeability was assumed in the numerical simulations ($k_h = k_v$), whereas artificially compacted soils often exhibit anisotropic permeability ($k_h/k_v > 1$). ii) Estimation of soil water retention curves (SWRCs): it is recommended that SWRCs are determined with additional laboratory tests for further simulations. So far, SWRCs have been estimated on the basis of grain size distributions.
- It should be considered whether the test may be carried out with viscous fluid instead of water to reduce the time scaling factor (Wood, 2004). This could help to extend the simulated flood duration to achieve better experimental control and data collection.

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Declaration of interest

The authors have no conflicts of interest to declare. All co-authors have observed and affirmed the contents of the paper and there is no financial interest to report.

Authors' contributions

Gregor Portmann: conceptualization, data curation, formal analysis, methodology, investigation, visualization, writing – original draft. André Arnold: conceptualization, funding acquisition, methodology, project administration, resources, supervision, validation, writing – review & editing. Yuen Zhang: methodology, investigation, writing – review & editing. Amin Askarinejad: conceptualization, methodology, supervision, project administration, resources, validation, writing – review & editing.

Data availability

The datasets generated and analysed in the course of the current study are available from the corresponding author upon request.

Declaration of use of generative artificial intelligence

This work was prepared with the assistance of generative artificial intelligence (GenAI) DeepL (<https://www.deepl.com>) to provide assistance in translating text elements from German into English. The entire process of using this tool was supervised, reviewed and when necessary edited by the authors. The authors assume full responsibility for the content of the publication that involved the aid of GenAI.

List of symbols and abbreviations

d	diameter
d_{70}	70% of the soil grains are finer than the given value
e	voids ratio
e_{init}	initial voids ratio after compaction
g	gravitational acceleration, 9.81 m/s ²
h	pressure head (suction is positive)
k	coefficient of permeability, k_v : for vertical and k_h : for horizontal permeability
l	seepage path length, l_m : in the model, l_p : in the prototype respectively
m	model parameters describing the curvature of the water retention curve after van Genuchten (1980)
n	model parameters describing the curvature of the water retention curve after van Genuchten (1980)
t	time, t_m : in model scale, t_p : in prototype scale respectively
v	seepage velocity, v_m : in the model, v_p : in the prototype respectively
w	water content
FEM	Finite Element Method
H	height of the flood simulator
K_r	relative hydraulic permeability of partially saturated soils

L	length of the flood simulator
N	centrifuge acceleration level (N times earth gravity)
PPT	Porewater-Pressure-Transducers
SWRC	Soil Water Retention Curve
W	width of the flood simulator
α	model parameters describing the curvature of the water retention curve after van Genuchten (1980)
ϕ'_{cv}	critical state angle of friction
θ	volumetric water content
θ_r	residual volumetric water content
θ_s	saturated volumetric water content
Θ	dimensionless water content

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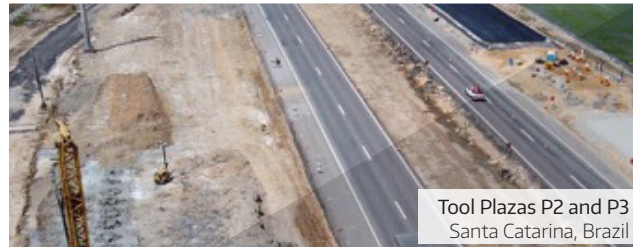
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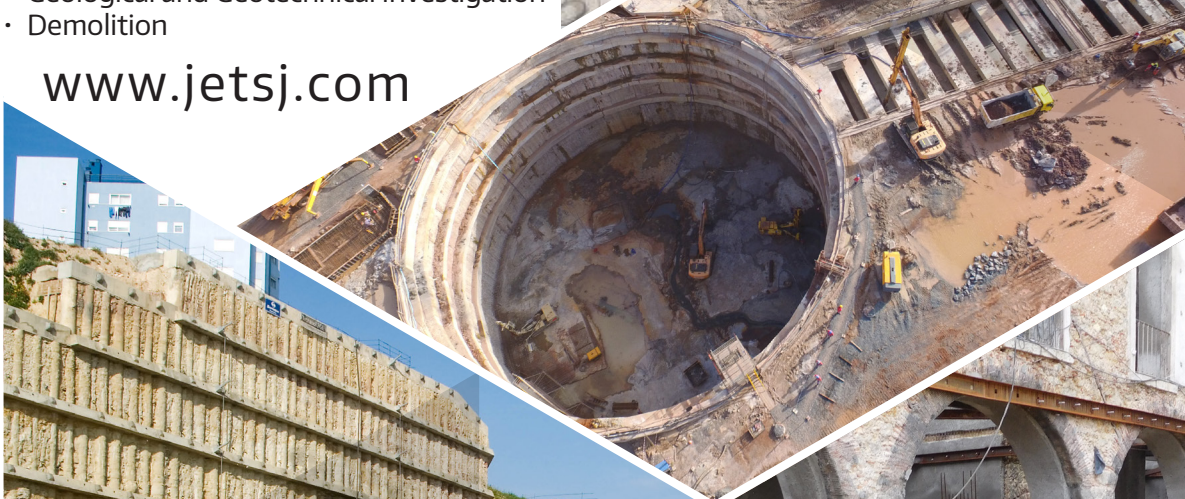
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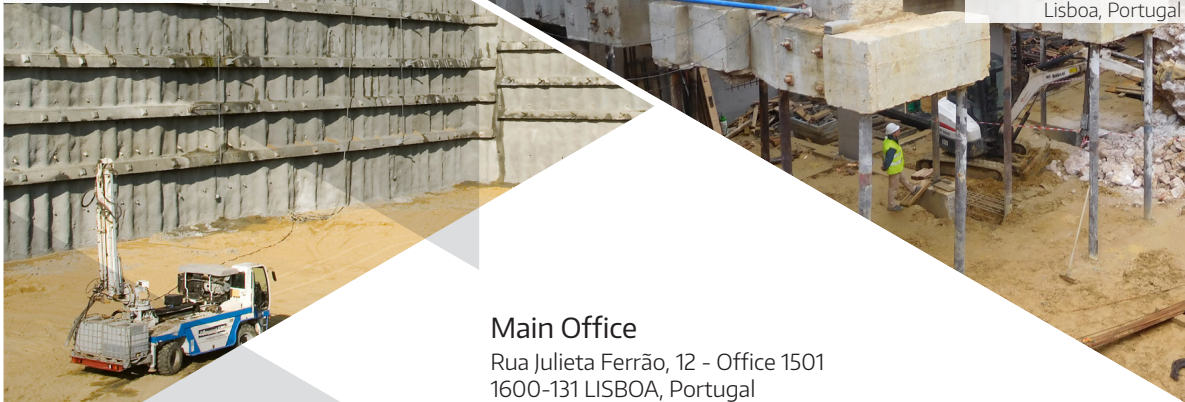
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Authors are responsible for selecting the correct category when submitting their manuscript. However, the manuscript category may be altered based on the recommendation of the Editorial Board. Authors are also requested to state the category of paper in their cover letter.

When submitting a manuscript for review, the authors should indicate the category of the manuscript, and is also understood that they:

- assume full responsibility for the contents and accuracy of the information in the paper;
- assure that the paper has not been previously published, and is not being submitted to any other journal for publication.

2. Paper length

Full-length manuscripts (Article, Case Study) should be between 4,000 and 8,000 words. Review articles should have up to 10,000 words. Technical Notes have a word count limit of 3,500 words. Discussions have a word count limit of 1,000 words. These word count limits exclude the title page, notation list (e.g., symbols, abbreviations), acknowledgments and references, but include any appendices. Each single column and double column figure or table is considered as equivalent to 150 and 300 words, respectively.

3. Scientific style

The manuscripts should be written in UK or US English, in the third person and all spelling should be checked in accordance with a major English Dictionary. The manuscript should be able to be readily understood by a Civil Engineer and avoid colloquialisms. Unless essential to the comprehension of the manuscript, direct reference to the names of persons, organizations, products or services is not allowed. Flattery or derogatory remarks about any person or organization should not be included.

The author(s) of Discussion Papers should refer to himself (herself/themselves) as the reader(s) and to the author(s) of the paper as the author(s).

The International System (SI) units must be used. The symbols are recommended to be in accordance with Lexicon in 14 Languages, ISSMFE (2013) and the ISRM List of Symbols. Use italics for single letters that denote mathematical constants, variables, and unknown quantities, either in tables or in the text.

4. Submission requirements and contents

A submission implies that the following conditions are met:

- the authors assume full responsibility for the contents and accuracy of the information presented in the paper;
- the manuscript contents have not been published previously, except as a lecture or academic thesis;
- the manuscript is not under consideration for publication elsewhere;
- the manuscript is approved by all authors;
- the manuscript is approved by the necessary authorities, when applicable, such as ethics committees and institutions that may hold intellectual property on contents presented in the manuscript;
- the authors have obtained authorization from the copyright holder for any reproduced material;
- the authors are aware that the manuscript will be subjected to plagiarism check.

The author(s) must upload two digital files of the manuscript to the Soils and Rocks submission system. The size limit for each submission file is 20 MB. The manuscript should be submitted in docx format (Word 2007 or higher) or doc format (for older Word versions). An additional PDF format file of the manuscript is also required upon submission. Currently, the journal is not accepting manuscripts prepared using LaTeX.

The following documents are required as minimum for submission:

- cover letter;
- manuscript with figures and tables embedded in the text (doc or docx format);
- manuscript with figures and tables embedded in the text for revision (PDF format);
- permission for re-use of previously published material when applicable, unless the author/owner has made explicit that the image is freely available.

4.1 Cover letter

The cover letter should include: manuscript title, submission type, authorship information, statement of key findings and work novelty, and related previous publications if applicable. When authors opt for the double-blind peer review process, information on Acknowledgements and Authors' Contributions also should be included.

4.2 Title page

The title page is the first page of the manuscript and must include:

- A concise and informative title of the paper. Avoid abbreviations, acronyms or formulae. Discussion Papers should contain the title of the paper under discussion. Only the first letter of the first word should be capitalized.
- Full name(s) of the author(s). The first name(s) should not be abbreviated. The authors are allowed to abbreviate middle name(s).
- The corresponding author should be identified by a pound sign # beside his/her and in a footnote.

- The affiliation(s) of the author(s), should follow the format: Institution, (Department), City, (State), Country. Names of institutions and programs are to be presented in full and in the original language of the of the institution or in the English version for non-Latin writing.
- Affiliation address and e-mail must appear below each author's name.
- The 16-digit [ORCID](#) of the author(s) – mandatory
- Link to the authors' CV Lattes – mandatory for authors affiliated with Brazilian institutions only
- Main text word count (excluding abstract and references) and the number of figures and tables

If authors chose the double-blind peer review process, the title page cannot include any authorship detail, such as name, institution, email, and ORCID.

4.3 Permissions

Figures, tables or text passages previously published elsewhere may be reproduced under permission from the copyright owner(s) for both the print and online format. The authors are required to provide evidence that such permission has been granted at the moment of paper submission.

4.4 Declaration of interest

Authors are required to disclose conflicting interests that could inappropriately bias their work. For that end, a section entitled "Declaration of interest" should be included following any acknowledgments and prior to the "Authors' contributions" section. In case of the absence of conflicting interests, the authors should still include a declaration of interest.

4.5 Authors' contributions

Authors are required to include an author statement outlining their individual contributions to the paper according to the CRediT taxonomy (as per <https://credit.niso.org>). The minimum requirements of contribution to the work for recognition of authorship are: a) Participate actively in the discussion of results; b) Review and approval of the final version of the manuscript. A section entitled "Authors' contributions" should be included after the declaration of interest section, and should be formatted with author's name and CRediT role(s), according to the example:

Samuel Zheng: conceptualization, methodology, validation. Olivia Prakash: data curation, writing - original draft preparation. Fatima Wang: investigation, validation. Kwame Bankole: supervision. Sun Qi: writing - reviewing and editing.

Do not include credit items that do not follow the Taxonomy established by CRediT Contributor Roles Taxonomy.

The authors' contributions section should be omitted in manuscripts that have a single author. In case the authors opt for the double-blind review process, the authors' contributions should be stated in the cover letter and omitted from the manuscript.

4.6 Data availability

A data availability statement must be included in any original research article. Authors should include information on where

data supporting the findings reported in the article can be found, if applicable. Persistent hyperlink to make available archived datasets evaluated or produced during the study are to be included. “Data” is the minimal dataset necessary to understand, replicate and develop based on the results presented in the article. Cases in which sharing research data is not possible, data availability is still to be stated in the manuscript including any requirements for access.

4.7 Use of generative artificial intelligence for textual, visual, and data enhancement

The use of generative artificial intelligence (GenAI) has proven to be a valuable tool for translation and for improving the quality of texts, images, graphic elements, as well as for the treatment and compilation of data and results in scientific publications. However, Soils and Rocks considers that at no time can GenAI tools be considered as the author or co-author of the work since authorship and co-authorship are exclusively attributed to human action. Such tool should be used as aid and should be always supervised by humans.

The use of Generative Artificial Intelligence to enhance text, images, graphic elements, treatment, and compilation of data and results, must be explicitly declared in a section entitled “Declaration of use of generative artificial intelligence”. Such a declaration should follow the template presented below and be positioned after the “Data availability” section.

Declaration of use of generative artificial intelligence

This is a statement example. This work was prepared with the assistance of generative artificial intelligence (GenAI) [*specify the name of the tool/service*] with the aim of [*specify how GenAI was used and for what elements of the manuscript*]. The entire process of using this tool was supervised, reviewed and when necessary edited by the authors. The authors assume full responsibility for the content of the publication that involved the aid of GenAI.

5. Plagiarism checking

Soils and Rocks follows the ethics of the Committee on Publication Ethics (COPE), of the Code of Ethics American Educational Research Association, the Code of Good Scientific Practice - FAPESP and the Council of Science Editors (CSE) concerning publication. Any material published in is in agreement with the SciELO Guidelines on Best Practices for Strengthening Ethics in Scientific Publication.

Submitted papers are expected to contain at least 50 % new content and the remaining 50 % should not be verbatim to previously published work.

All manuscripts are screened for similarities. Currently, the Editorial Board uses the plagiarism checker Plagius (www.plagius.com) to compare submitted papers to already published works. Manuscripts will be rejected if more than 20 % of content matches previously published work, including self-plagiarism. The decision to reject will be under the Editors’ discretion if the percentage is between 10 % and 20 %.

The Editorial Board will be responsible for analyzing reports of plagiarism, duplicate or redundant publication after the articles have been published. The article will be removed immediately

from the Journal if the complaint is confirmed. Authors may also be subjected to the penalties.

Corrections and retractions may be corrected through an Erratum in case that errors and shortcomings do not imply in “Misconduct in publication”, which will be recognized as fabrication, falsification, and plagiarism (as per SciELO Guidelines on Best Practices for Strengthening Ethics in Scientific Publication).

IMPORTANT OBSERVATION: Mendeley software plug-in (suggested in this guide) for MS-Word can be used to include the references in the manuscript. This plug-in uses a field code that sometimes includes automatically both title and abstract of the reference. Unfortunately, the similarity software adopted by the Journal (Plagius) recognizes the title and abstract as an actual written text by the field code of the reference and consequently increases considerably the percentage of similarity. Please do make sure to remove the abstract (if existing) inside Mendeley section where the adopted reference is included. This issue has mistakenly caused biased results in the past. The Editorial Board of the journal is now aware of this tendentious feature.

6. Formatting instructions

The text must be presented in a single column, using ISO A4 page size, left, right, top, and bottom margins of 25 mm, Times New Roman 12 font, and line spacing of 1.5. All lines and pages should be numbered.

The text should avoid unnecessary italic and bold words and letters, as well as too many acronyms. Authors should avoid capitalizing words and whenever possible to use tables with distinct font size and style of the regular text.

Figures, tables and equations should be numbered in the sequence that they are mentioned in the text.

Abstract

Please provide an abstract between 150 and 250 words in length. Abbreviations or acronyms should be avoided. The abstract should state briefly the purpose of the work, the main results and major conclusions or key findings.

Keywords

A minimum of three and a maximum of six keywords must be included after the abstract. The keywords must represent the content of the paper. Keywords offer an opportunity to include synonyms for terms that are frequently referred to in the literature using more than one term. Adequate keywords maximize the visibility of your published paper.

Examples:

Poor keywords – piles; dams; numerical modeling; laboratory testing

Better keywords – friction piles; concrete-faced rockfill dams; material point method; bender element test

List of symbols

A list of symbols and definitions used in the text must be included before the References section. Any mathematical constant,

variable or unknown quantity should appear in italics. List lower case characters first, then upper case. Latin alphabet characters must appear before Greek alphabet characters.

Appendix

Use the Appendix when necessary to provide supporting or supplementary information that is part of the study (e.g. tables, graphs, or datasets). The Appendix should be placed after the References. For more than one appendix, organize the titles with letters like Appendix A, Appendix B, and Appendix C.

6.1 Citations

References to other published sources must be made in the text by the last name(s) of the author(s), followed by the year of publication. Examples:

- Narrative citation: [...] while Silva & Pereira (1987) observed that resistance depended on soil density
- Parenthetical citation: It was observed that resistance depended on soil density (Silva & Pereira, 1987).

In the case of three or more authors, the reduced format must be used, e.g.: Silva et al. (1982) or (Silva et al., 1982). Do not italicize “et al.”

Two or more citations belonging to the same author(s) and published in the same year are to be distinguished with small letters, e.g.: (Silva, 1975a, b, c.).

Standards must be cited in the text by the initials of the entity and the year of publication, e.g.: ABNT (1996), ASTM (2003).

6.2 References

A customized style for the Mendeley software is available and may be downloaded from:

<https://cs1.mendeley.com/styles/583367631/soilsandrocks-4>

Full references must be listed alphabetically at the end of the text by the first author's last name. Several references belonging to the same author must be cited chronologically. Provide DOI (Digital Object Identifier) whenever available.

Some formatting examples are presented here:

Journal Article

Bishop, A.W., & Blight, G.E. (1963). Some aspects of effective stress in saturated and partly saturated soils. *Géotechnique*, 13(2), 177-197. <https://doi.org/10.1680/geot.1963.13.3.177>.

Castellanza, R., & Nova, R. (2004). Oedometric tests on artificially weathered carbonatic soft rocks. *Journal of Geotechnical and Geoenvironmental Engineering*, 130(7), 728-739. [https://doi.org/10.1061/\(ASCE\)1090-0241\(2004\)130:7\(728\)](https://doi.org/10.1061/(ASCE)1090-0241(2004)130:7(728)).

Fletcher, G. (1965). Standard penetration test: its uses and abuses. *Journal of the Soil Mechanics Foundation Division*, 91, 67-75.

Indraratna, B., Kumara, C., Zhu S-P., Sloan, S. (2015). Mathematical modeling and experimental verification of fluid flow through deformable rough rock joints. *International Journal*

of Geomechanics, 15(4): 04014065-1-04014065-11. [https://doi.org/10.1061/\(ASCE\)GM.1943-5622.0000413](https://doi.org/10.1061/(ASCE)GM.1943-5622.0000413).

Garnier, J., Gaudin, C., Springman, S.M., Culligan, P.J., Goodings, D., Konig, D., ... & Thorel, L. (2007). Catalogue of scaling laws and similitude questions in geotechnical centrifuge modelling. *International Journal of Physical Modelling in Geotechnics*, 7(3), 01-23. <https://doi.org/10.1680/ijpmg.2007.070301>.

Bicalho, K.V., Gramelich, J.C., & Santos, C.L.C. (2014). Comparação entre os valores de limite de liquidez obtidos pelo método de Casagrande e cone para solos argilosos brasileiros. *Comunicações Geológicas*, 101(3), 1097-1099 (in Portuguese).

Book

Lambe, T.W., & Whitman, R.V. (1979). *Soil Mechanics, SI version*. John Wiley & Sons.

Das, B.M. (2012). *Fundamentos de Engenharia Geotécnica*. Cengage Learning (in Portuguese).

Head, K.H. (2006). *Manual of Soil Laboratory Testing - Volume I: Soil Classification and Compaction Tests*. Whittles Publishing.

Bhering, S.B., Santos, H.G., Manzatto, C.V., Bognola, I., Fasolo, P.J., Carvalho, A.P., ... & Curcio, G.R. (2007). *Mapa de solos do estado do Paraná*. Embrapa (in Portuguese).

Book Section

Yerro, A., & Rohe, A. (2019). Fundamentals of the Material Point Method. In *The Material Point Method for Geotechnical Engineering* (pp. 23-55). CRC Press. <https://doi.org/10.1201/9780429028090>.

Sharma, H.D., Dukes, M.T., & Olsen, D.M. (1990). Field measurements of dynamic moduli and Poisson's ratios of refuse and underlying soils at a landfill site. In *Geotechnics of Waste Fills - Theory and Practice* (pp. 57-70). ASTM International. <https://doi.org/10.1520/STP1070-EB>.

Cavalcante, A.L.B., Borges, L.P.F., & Camapum de Carvalho, J. (2015). Tomografias computadorizadas e análises numéricas aplicadas à caracterização da estrutura porosa de solos não saturados. In *Solos Não Saturados no Contexto Geotécnico* (pp. 531-553). ABMS (in Portuguese).

Proceedings

Jamiolkowski, M.; Ladd, C.C.; Germaine, J.T., & Lancellotta, R. (1985). New developments in field and laboratory testing of soils. *Proc. 11th International Conference on Soil Mechanics and Foundation Engineering*, San Francisco, August 1985. Vol. 1, Balkema, 57-153.

Massey, J.B., Irfan, T.Y. & Cipullo, A. (1989). The characterization of granitic saprolitic soils. *Proc. 12th International Conference on Soil Mechanics and Foundation Engineering*, Rio de Janeiro. Vol. 6, Publications Committee of XII ICSMFE, 533-542.

Indraratna, B., Oliveira D.A.F., & Jayanathan, M. (2008b). Revised shear strength model for infilled rock joints considering overconsolidation effect. *Proc. 1st Southern Hemisphere International Rock Mechanics Symposium*, Perth. ACG, 16-19.

Barreto, T.M., Repsold, L.L., & Casagrande, M.D.T. (2018). Melhoramento de solos arenosos com polímeros. *Proc. 19º Congresso Brasileiro de Mecânica dos Solos e Engenharia Geotécnica*, Salvador. Vol. 2, ABMS, CBMR, ISRM & SPG, 1-11 (in Portuguese).

Thesis

Lee, K.L. (1965). *Triaxial compressive strength of saturated sands under seismic loading conditions* [Doctoral dissertation]. University of California at Berkeley.

Chow, F.C. (1997). *Investigations into the behaviour of displacement pile for offshore foundations* [Doctoral thesis, Imperial College London]. Imperial College London's repository. <https://spiral.imperial.ac.uk/handle/10044/1/7894>.

Araki, M.S. (1997). *Aspectos relativos às propriedades dos solos porosos colapsíveis do Distrito Federal* [Master's dissertation]. University of Brasília (in Portuguese).

Sotomayor, J.M.G. (2018). *Evaluation of drained and non-drained mechanical behavior of iron and gold mine tailings reinforced with polypropylene fibers* [Doctoral thesis, Pontifical Catholic University of Rio de Janeiro]. Pontifical Catholic University of Rio de Janeiro's repository (in Portuguese). <https://doi.org/10.17771/PUCRio.acad.36102>.*

* official title in English should be used when available in the document.

Report

ASTM D7928-17. (2017). Standard Test Method for Particle-Size Distribution (Gradation) of Fine-Grained Soils Using the Sedimentation (Hydrometer) Analysis. *ASTM International, West Conshohocken, PA*. <https://doi.org/10.1520/D7928-17>.

ABNT NBR 10005. (2004). Procedure for obtention leaching extract of solid wastes. *ABNT - Associação Brasileira de Normas Técnicas*, Rio de Janeiro, RJ (in Portuguese).

DNIT. (2010). Pavimentação - Base de solo-cimento - Especificação de serviço DNIT 143. *DNIT -Departamento Nacional de Infraestrutura de Transportes*, Rio de Janeiro, RJ (in Portuguese).

USACE. (1970). Engineering and Design: Stability of Earth and Rock-Fill Dams, Engineering Manual 1110-2-1902. *Corps of Engineers*, Washington, D.C.

Web Page

Soils and Rocks. (2020). *Guide for Authors*. Soils and Rocks. Retrieved in September 16, 2020, from <http://www.soilsandrocks.com/>.

6.3 Artworks and illustrations

Each figure should be submitted as a high-resolution image, according to the following mandatory requirements:

- Figures created as a TIFF or JPEG file format with minimum resolution of 500 dpi.

- Size the figures according to their final intended size. Single-column figures should have a width of up to 82 mm. Double-column figures should have a maximum width of 170 mm.
- Use Times New Roman for figure lettering. Use lettering sized 8-10 pt. for the final figure size.
- Lines should have 0.5 pt. minimum width in drawings.
- Titles or captions should not be included inside the figure itself.

Figures must be embedded in the text near the position where they are first cited. Cite figures in the manuscript in consecutive numerical order. Denote figure parts by lowercase letters (a, b, c, etc.). Please include a reference citation at the end of the figure caption for previously published material. Authorization from the copyright holder must be provided upon submission for any reproduced material.

Figure captions must be placed below the figure and start with the term "Figure" followed by the figure number and a period. Example:

Figure 1. Shear strength envelope.

Do not abbreviate "Figure" when making cross-references to figures.

All figures are published in color for the electronic version of the journal; however, the print version uses grayscale. Please format figures so that they are adequate even when printed in grayscale.

Accessibility: Please make sure that all figures have descriptive captions (text-to-speech software or a text-to-Braille hardware could be used by blind users). Prefer using patterns (e.g., different symbols for dispersion plot) rather than (or in addition to) colors for conveying information (then the visual elements can be distinguished by colorblind users). Any figure lettering should have a contrast ratio of at least 4.5:1

Improving the color accessibility for the printed version and for colorblind readers: Authors are encouraged to use color figures because they will be published in their original form in the online version. However, authors must consider the need to make their color figures accessible for reviewers and readers that are colorblind. As a general rule of thumb, authors should avoid using red and green simultaneously. Red should be replaced by magenta, vermillion, or orange. Green should be replaced by an off-green color, such as blue-green. Authors should prioritize the use of black, gray, and varying tones of blue and yellow.

These rules of thumb serve as general orientations, but authors must consider that there are multiple types of color blindness, affecting the perception of different colors. Ideally, authors should make use of the following resources: 1) for more information on how to prepare color figures, visit <https://jfly.uni-koeln.de/>; 2) a freeware software available at <http://www.vischeck.com/> is offered by Vischeck, to show how your figures would be perceived by the colorblind.

6.4 Tables

Tables should be presented as a MS Word table with data inserted consistently in separate cells. Place tables in the text near the

position where they are first cited. Tables should be numbered consecutively using Arabic numerals and have a caption consisting of the table number and a brief title. Tables should always be cited in the text. Any previously published material should be identified by giving the original source as a reference at the end of the table caption. Additional comments can be placed as footnotes, indicated by superscript lower-case letters.

When applicable, the units should come right below the corresponding column heading. Horizontal lines should be used at the top and bottom of the table and to separate the headings row. Vertical lines should not be used.

Table captions must be placed above the table and start with the term “Table” followed by the table number and a period. Example:

Table 1. Soil properties.

Do not abbreviate “Table” when making cross-references to tables. Example:

Table 1. Soil properties

Parameter	Symbol	Value
Specific gravity of the sand particles	G_s	2.64
Maximum dry density (Mg/m ³)	$\rho_{d(max)}$	1.554
Minimum dry density (Mg/m ³)	$\rho_{d(min)}$	1.186
Average grain-size (mm)	d_{50}	0.17
Coefficient of uniformity	C_u	1.97

6.5 Mathematical equations

Equations must be submitted as editable text, created using MathType or the built-in equation editor in MS Word. All variables must be presented in italics.

Equations must appear isolated in a single line of the text. Numbers identifying equations must be flushed with the right margin. International System (SI) units must be used. The definitions of the symbols used in the equations must appear in the List of Symbols.

Do not abbreviate “Equation” when making cross-references to an equation.

7. Reviewing procedures

All manuscripts are evaluated following a **single-blind** or a **double-blind** peer-review process. In a single-blind evaluation

the reviewers are kept anonymous. In a double-blind evaluation, both the reviewers and authors are not identified to each other.

Authors **must opt** for either a single or double-blind evaluation process during the submission of their manuscripts. The Associated Editor appointed to your manuscript chooses appropriate reviewers based on the manuscript’s subject and may or may not appoint reviewers suggested by the authors.

Manuscripts submitted to the Journal enters a pre-review assessment by the Editor-in-Chief who decides whether it can be sent for further review. Three criteria are considered in this first evaluation: i) adequacy to the Journal’s scope; ii) compliance with the instructions in the Guidelines for Authors; iii) paper is an original manuscript. Following, an associate editor is assigned to the submission and the paper is sent at least to two independent reviewers, which evaluates the scientific content. Reviewers may recommend to the Associate Editor either: a) accept as it is; b) accept with minor revision by authors; c) accept with major revision by authors; d) reject encouraging further resubmission after extensive changes; e) reject without any possibility of further resubmission.

Typically, two to three evaluations are required before the Associated Editor makes a decision recommendation to the Editor-in-Chief, which may be either: a) accept for publication; b) requires format revision; c) requires content revision due to similarity check; d) requires minor revision; e) requires major revision; f) reject. Currently, invited reviewers are given a deadline of 40 days to provide their assessments.

The deadline for authors to submit revised manuscripts is 20 days for minor changes, whereas for major changes it is 30 days.

8. Instructions for authors opting for a double-blind evaluation procedure

Authors opting for a double-blind evaluation of the manuscript must: 1) Clearly indicate their option for a double-blind evaluation in the cover letter; 2) Remove from the submitted manuscript files any indication of authorship;

- First page: remove all authorship details, such as institution, email, and ORCID;
- Acknowledgements and Authors’ Contributions: please omit these sections since they often can reveal details regarding the manuscript’s authorship. The information from these sections should be **included in the cover letter** and, at a later stage, in the manuscript after it is accepted for publication.

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EDITORIAL

Editorial to the Special Issue: 18th ECSMGE, Lisbon 2024

António Alberto Correia, José Mateus de Brito, Cristiana Ferreira

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The influence of the construction methodology on the modelled response of shafts

António M. G. Pedro, David M. G. Taborda, Lucas M. Repsold, Jorge Almeida e Sousa

Structural health monitoring and inspection of dams based on unmanned aerial vehicles photogrammetry with 3D model reconstruction

Daniel Teixeira Leite, João Marcelino, Nuno Manuel da Costa Guerra, João Miguel Gomes Pires Manso, Nuno Miguel Cavalheiro Marques

Large-scale testing of free-ended piles in unsaturated expansive clays under cyclic and monotonic lateral loading

Ruan A. Murison, Tiago A. V. Gaspar, Gerhard Heymann, Schalk W. Jacobsz, Ashraf S. Osman

A strategic and novel field trial to investigate the dynamic performance of cement-stabilised chalk as embankment fill for high-speed rail earthworks

Ouarda Boumendjel-Game, Harry Saroglou, Matthew Fleetwood, Mike Black

A coupled hydromechanical model including fracture propagation along the dam foundation interface for gravity dam's stability assessment

Maria Luísa Braga Farinha, Nuno Monteiro Azevedo, Sérgio Oliveira

Estimation of the permeability coefficient of recycled concrete aggregates using machine learning algorithms:

AdaBoost and artificial neural networks

Justyna Dzięcioł, Wojciech Sas

Experimental research of the effects of adding different chemical reagents to clayey soil on improving its physical and mechanical properties

Elefterija Zlatanović, Nemanja Marinković, Nebojša Davidović, Zoran Bonić, Nikola Romić

Influence of anisotropic stress states on soil-structure interface behavior: a new experimental approach in a modified hollow cylinder apparatus

Sabine Gehring, Hans Henning Stutz

Evaluation of slope failure through the assessment of meteorological conditions and earthworks management

Nguyen Quang Tuan, Duc Bui Van, Piotr Osinski, Nguyen Van Manh, Ngoc Anh Do, Eugeniusz Koda

Design of the Vienna metro shaft: probabilistic analysis and settlement forecasting of the soil-structure system via Monte Carlo simulation

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Interpretation of large diameter preliminary pile tests in Ampthill Clay, West Walton and Oxford Clay formations for Thame Valley Viaduct

Sergio Vitorino, Hoe-Chian Yeow

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New electric powered drill rig to meet sustainable geotechnics

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Insights into drop mass systems for predicting pile compressive resistance from dynamic load test energy measurements: a case study of the Tagus River Lezíria bridge and viaducts

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Han Vo-Cong, Kinji Takeuchi, Yasuo Tomono, Tatsunori Matsumoto

REVIEW ARTICLE

Seepage in a flood protection levee – first geotechnical centrifuge test results

Gregor Portmann, André Arnold, Yuen Zhang, Amin Askarinejad