Numerical Analysis and field behaviour of stone columns - strengthened soft clay deposit using PLAXIS 2D

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ABSTRACT: The field behaviour of a soft foundation with stone columns reinforcement at a coal and ore stockyard was detailed in this research, and the results were compared using numerical analysis. Along a 500-operational day period, the performance of a 130 m wide segment with two ore stacks was documented. An elaborate instrumentation system with 14 sensors was employed to track the stabilised area's serviceability behaviour. Using the PLAXIS 2D finite element code, a complementary numerical analysis was carried out in order to properly represent the increase in lateral earth pressure brought on by the placement of columns. The stone columns were transformed into comparable walls for the sake of the numerical analysis, which was conducted using a plane strain technique. The proposed plane strain model was able to accurately forecast the overall deformations of the reinforced foundation, according to the results. The behaviour of the simulated excess pore pressure curves was likewise consistent with the field data, with a peak value at the time of loading application and a slow decline during the consolidation phases.

KEYWORDS: Soil improvement; stone column; field test; soft soil; field instrumentation; in-situ testing; numerical analysis.

1. Introduction

Construction on soft soils is never simple because of the deposits' extreme compressibility and limited bearing capacity. One of the most adaptable and frequently used techniques for reducing and accelerating settlement, increasing load-bearing capacity, reducing horizontal deformations, and improving overall stability of embankments over soft soil deposits is the use of compacted granular columns (Poorooshas and Meyerhof 1996; Greenwood 1970; Almeida et al. 2018).

The first analytical solution was provided by Greenwood (1970) to determine the bearing capacity and settlement of a stiff foundation supported by a number of stone columns. Later, Priebe (1995) put out a technique based on the unit cell idea to calculate the settlement on an endless grid of vibro-replaced stone columns. Up to now, several researchers have developed theoretical methods for estimating bearing capacity and settlement of foundations reinforced by stone columns (e.g. Hughes and Withers 1974; Thorburn 1975; Aboshi et al. 1979; Balaam and Booker 1981, 1985; Bouassida et al. 2003; Pulko and Majes 2005; Castro and Sagaseta 2009; Zhang et al. 2013; Indraratna, Basack, and Rujikiatkamjorn 2013; Deb and Shiyamalaa 2016). Numerical analysis using finite element methods is also often used to predict the behaviour of stone columns (e.g., Dash and Bora 2013; Indraratna, Basack, and Rujikiatkamjorn 2013; Castro 2014; Tan, Ng, and Sun 2014; Ellouze et al. 2016). The bulk of these studies employ the unit cell method, which includes a single stone column and the effect zone that is dependent on the arrangement and spacing of the columns.

Field load testing, in addition to numerical and analytical studies, may be a suitable option for comprehending the behaviour of the stone columns since they offer useful information illuminating the real reaction of the composite system. But compared to numerical and analytical research, these tests are less accessible (see Mestat, Magnan, and Dhouib 2006; Yee and Raju 2007; Egan, Scott, and McCabe 2008; Weber et al. 2008; McCabe, Nimmons, and Egan 2009, for examples). Mestat, Magnan, and Dhouib (2006) described the behaviour of a test embankment enhanced with stone columns and erected on compressible clayey soil. Complementary numerical and analytical analyses showed that determining the settlement in such projects is a difficult and complex issue. They also came to the conclusion that a trustworthy interpretation of the behaviour of the composite ground requires the numerical analysis in conjunction with the instrumentation data.

Stone columns have been used in soft clayey soil with undrained strength values lower than 15 kPa, according to Yee and Raju's 2007 research. Over soft soils stabilised by stone columns with maximum lengths and diameters of 26 m and 1.2 m, respectively, road embankments up to 10 m high were built.

The majority of the studies that have been done so far investigate a single stone column and the soft soil around it using a unit cell axisymmetric model. The lateral boundaries' horizontal fixities prevent the unit-cell idea from being able to compute the horizontal soil deformation. Additionally, it is unknown how the compacted stone columns may affect the embankment's capacity to retain its stability.

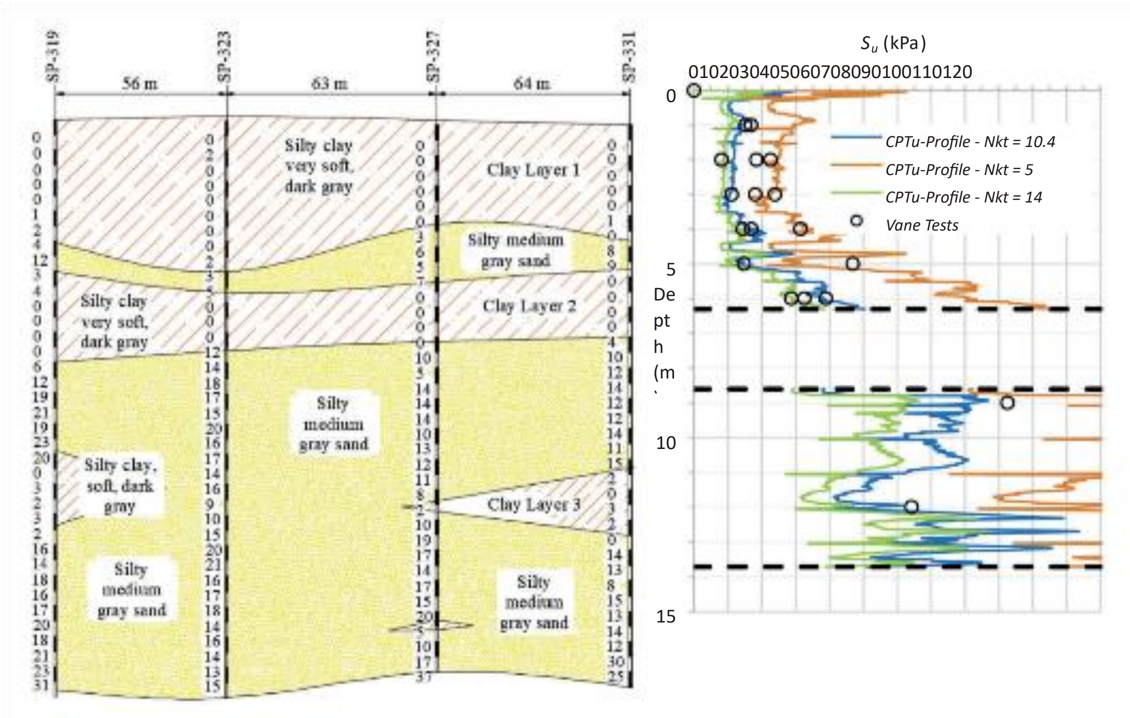
For the purpose of predicting the field performance of a soft soil reinforced by stone columns, the plane strain technique was used in the development of the current study (Wegner et al. 2009). The numerical study was carried out using the finite element programme PLAXIS 2D (Brinkgreve, Swolfs, and Engine 2011), where equivalent walls were used in place of the stone columns. As a result, a 130-meter-wide part of a coal and ore stockyard that included two ore stacks was investigated. loading and unloading cycles with a maximum vertical tension Throughout a 500-day period beginning with the application of the first load, at a pressure of around 120 kPa. The stockyard's ground treatment primarily aimed to regulate stability, lessen settlement, and expedite construction. In order to evaluate the settlement, lateral deformation of the clayey foundation, and excessive pore water pressure in soft clay behaviours of the composite ground, the stockyard was instrumented.

2. Geotechnical profile

The upper soft clay layer (Clay Layer 1), which makes up the majority of this profile and is 6.5 to 7.5 m thick at the test site, is represented in Figure 1(a) as the soil stratigraphy. There was a layer of sand between 1.0 and 3.0 m thick, which was followed by a second layer of soft clay (Clay Layer 2) that ranged in thickness from 3.0 to 5.0 m. Almeida et al. (2014a), Almeida and Marques (2013), and Hosseinpour et al. (2017b, 2016) are a few examples. The soft soil layers' plasticity index Ip ranged from 30% to 120%, and their natural water content ranged from 30% to 150%, according to the characterisation tests that were carried out. The SPT boreholes used for in-situ observation revealed that the water table level was roughly 1.0 m below the surface of the land. The geotechnical site investigation programme in the current study included six dilatometer tests, 14 boreholes of standard penetration tests (SPTs), 20 vertical cone penetration tests with pore pressure measurement (CPTu), 13 verticals of vane shear tests (VSTs), and 16 undisturbed soil sampling extracted using stationary equipment. Shelby Using VST data coupled with CPTu, the undrained shear strength of the soft clay strata Su was determined (Lunne, Robertson, and Powell 1997; Robertson and Cabal 2015). By linking CPTu with VST, the following formula was utilised to arrive at the most popular empirical cone factor Nkt:

$$\text{Nkt ¼ qT } \sigma\text{v0} \quad (1)$$

where Su(VT) is the vane shear strength and qT is the corrected tip resistance, and v0 is the in-situ total vertical stress at the same depth. For each depth (5–14) at which vane shear tests were conducted, values of the empirical cone factor Nkt were calculated. The average value of Nkt obtained was equal to 10.7, which is fairly close to the typical values for Rio de Janeiro's soft clay reported by Almeida and Marques (2013) and Hosseinpour et al. (2017). Figure 1(b) depicts the representative Su profile for the CPTu tests, as well as the maximum and lowest profiles based on the deposit's Nkt values and the layer limitations, with average Su values of around 12 kPa and 60.



(a)

Figure 1. Geotechnical profile of the subsoil at the test area: (a) soil stratigraphy; (b) Su profile by piezocone and vane shear tests

3. Field instrumentation

Figure 2 shows the position of the examined part on the stockyard's basic structure, along with ore and coal stacks. One-meter-diameter stone is present in the chosen section. Columns with a center-to-centre spacing of 2.20 metres below the ore stacks and 1.75 metres below the stack-reclaimers were placed in a square mesh using the vibro-replacement dry technique.

The 40 m wide northern stack and the 50 m wide southern stack, with stone-column lengths of 11.1 m and 11.6 m, respectively, are included in the investigated portion. The usable widths for the north and south sides, respectively, are 35.0 m and 45.0 m because 2.5 m on each side of the stacks is not needed. The instruments utilised in the current investigation are listed below.

- HPG North and HPG South, two horizontal profilometer gauges;
- Four Electrical piezometers, located in the centre of clay Layer 1 - Northern Part: PZ-N2 and PZ-N1; Southern Part: PZ-S2 and PZ-S1;
- Eight Settlement Sensors: located below the ore stack - Northern Part: SS-N4, SS-N3, SS-N2 and SS-N1; Southern Part: SS-S4, SS-S3, SS-S2 and SS-

The upper surface of the stockyard was covered with a 2.5-m-thick dredged sand working platform to offer a solid surface for fieldwork and column assembly. A bidirectional high-strength geogrid with an axial tensile strength of up to 1600 kN/m was then layered on top of the stone columns after they had been put in position.

To protect the stone columns/geogrids from harm during fieldwork, a 0.90 m thick layer of granular material (shown in Figure 3) was also laid on top of the geogrid. Sand and gravel that had been thoroughly compressed made up this granular layer. In Figure 3, the stone columns have been left off to provide a clearer view of the instruments.

The maximum potential height is 8.9 m for the northern stack and 11.4 m for the southern stack since the stack material in the analysed part consisted of pellets of iron ore with an approximately 27° angle of repose, a value close to the 26° figure stated in the literature. At least twice a day, stacks dumped in the stockyard were controlled by measuring their length and width as well as their total weight (calculated for the quantity of input and output material). By cross-referencing the stack controls with the piezometers installed in the examined area and checking the stack controls with regard to the number of days of loading or unloading, the applicable stockyard loading was determined. The height of the ore stack on the chosen dates was then specified and verified in the field based on the precise weight of the iron ore pellets that had been calculated in the lab.

Figure 3. Instrumentation positions in the studied section (dimensions are in metre).

The results of the numerical computations will then be reported (Item 5) together with the applied vertical strains on the northern and southern stacks. In certain circumstances, the fluctuation in the average applied vertical stress on the stacks of pellets reached 100 kPa in a couple of hours rather than a day, which is far higher than in regular earthwork services.

4. Finite element analysis

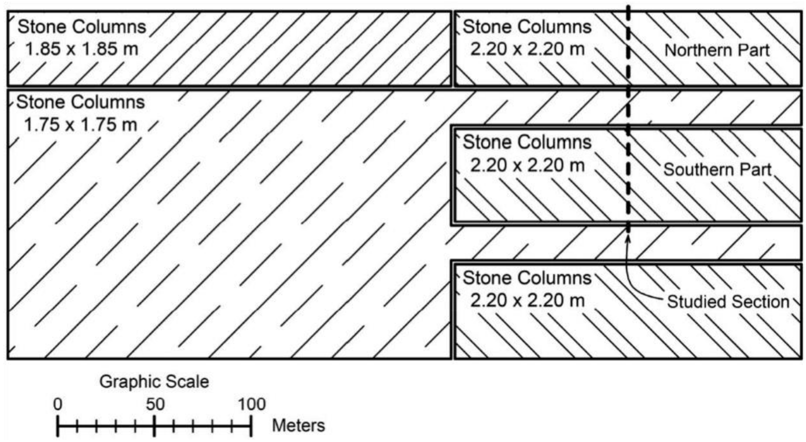
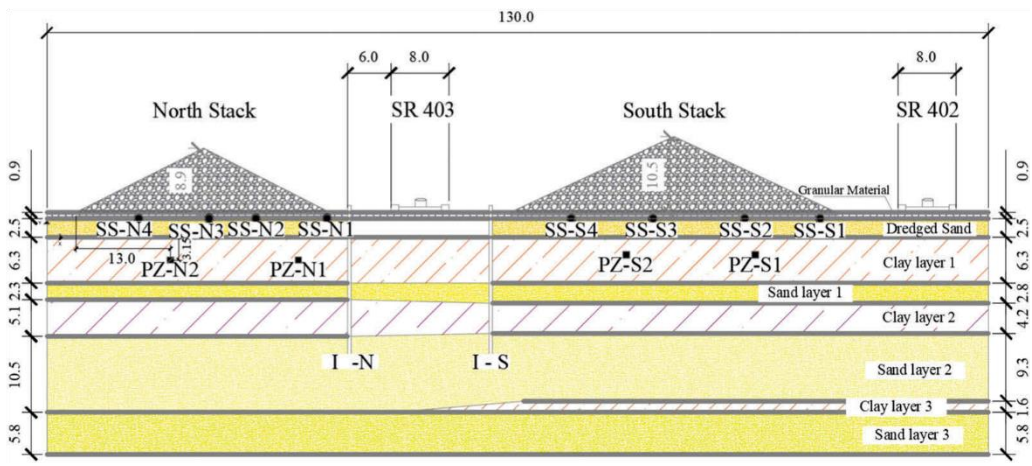


Figure 2. Stockyard outline and location of the studied section.



Using the finite element algorithm PLAXIS 2D, 15 nodal triangular elements were employed to simulate the soil clusters as part of the numerical studies of planar strain. The same numerical model was used to examine the northern and southern stacks, but with differing soil profile.

4.1. Model configuration

The Tan, Tjahyono, and Oo (2008) approach, which was also effectively applied by Hosseinpour et al. (2017a), was utilised to convert the axisymmetric to plane strain of the stone column. In this method, the analogous plane strain wall is used to replace the granular columns, and the column half-width (bc) is established by:

$$bc \text{ ¼ } BRr\underline{c}2_2 \quad (2)$$

R = radius of the unit cell, rc = radius of the column, and B = half of the plane strain effect area.

The following equation, based on the equivalent total area and column pattern, provides the connection between R and B (Barron 1948):

$$R \text{ ¼ } 1{:}13B \qquad (3)$$

For column spacings of 2.20 m and 1.75 m, respectively, the provided column diameter and grid pattern yield plane strain column widths of 0.36 m and 0.44 m. In order to accurately calculate the deformations and stresses, a fine mesh was chosen for the whole model based on the results of a mesh sensitivity analysis. In terms of the boundary fixities, the model was limited to deform vertically along the sides (i.e., roller borders), while remaining totally fixed along the base, as illustrated in Figure 4 along with the finite element mesh. The contact between the working platform and the top soft clay layer, as seen in situ, was chosen as the groundwater table level. The basal geogrid, a thin element with axial stiffness capable of withstanding just tensile force, was modelled using a pre-defined geogrid element provided in PLAXIS. A linear-elastic material with perfect adhesion to the surrounding soil and an axial stiffness of J= 800 kN/m was used to model the geogrid reinforcement. According to previous studies (e.g., Hatami and Bathurst 2005; Tandel, Solanki, and Desai 2012; Hosseinpour, Soriano, and Almeida 2019), parametric studies have demonstrated that the assumption of perfect interface bonding under working stress conditions yields reasonable predictions with respect to measured data.

4.2. Constitutive models and material properties

The Soft Soil Creep model, a Cam-Clay type model, was used to mimic the behaviour of the soft clay layers while taking into account secondary compression during consolidation analysis. Prior to the field load test, laboratory and in-situ studies were used to establish the Cam-Clay specifications. When the stone columns were installed, clay layer 1 was thought to have been smeared, which decreased the zone's coefficients of permeability. As advised by Watts et al. (2000), the geometric connection between the smear zone and the stone-column diameter was chosen to be 5.0. The performance of a field test on a set of 16 stone columns, loaded with iron rails, with extensive instrumentation was studied by (Almeida et al. 2014b) due to uncertainties of some parameter values of the column gravel material and the clay (e.g., the earth pressure coefficient after column installation - K* - and clay permeability). By contrasting numerical outcomes with field measurements, the model was validated. The best fit provided by the parametric analysis was achieved using a column friction angle Ø' = 40° (according to Barksdale and Bachus - FHWA, 1983; Mestat, Magnan, and Dhouib 2006; bouassida, Ellouze, and Hazzar 2008) and K* = 1.25 (similar to Guetif, Bouassida, and Debats 2007; Choobbasti, Zahmatkesh, and Noorzad 2011). Additionally, up to the final column depth, a ratio of the coefficient of permeability between before and after column installation equal to 5.0 was applied. The parameters of the soft clay layers employed in the numerical analysis are listed in Table 2.

For all the granular materials used in the investigation, the elastoplastic Mohr-Coulomb model (Ambily and Gandhi 2007; Six et al. 2012) was used. Along with the parameters used for the initial layer of granular material and ore pellets suggested by well-respected literature (e.g. Terzaghi and Peck 1967; Schmertmann 1978; Lambe and Whitman 1979), Table 3 shows the parameters used for the granular column, hydraulic fill working platform (dredged sand), and sand layers.

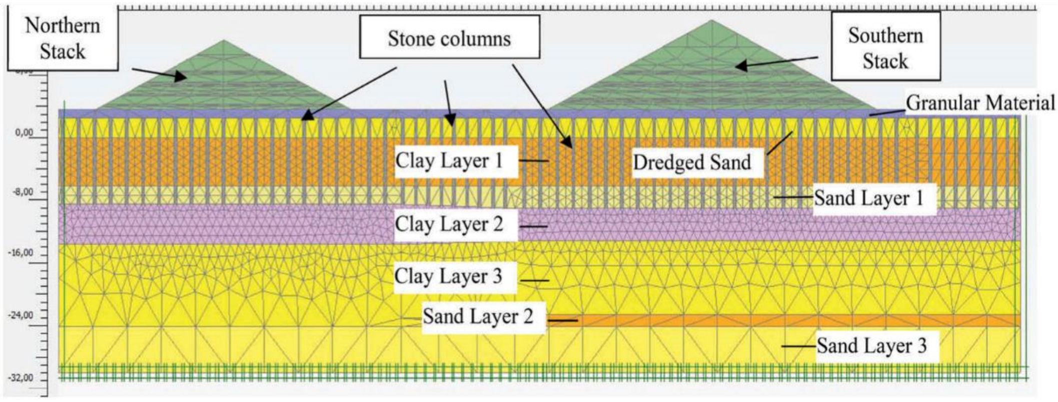


Figure 4. Numerical model showing soil cluster, boundary condition and generated mesh.

5. Results and discussion

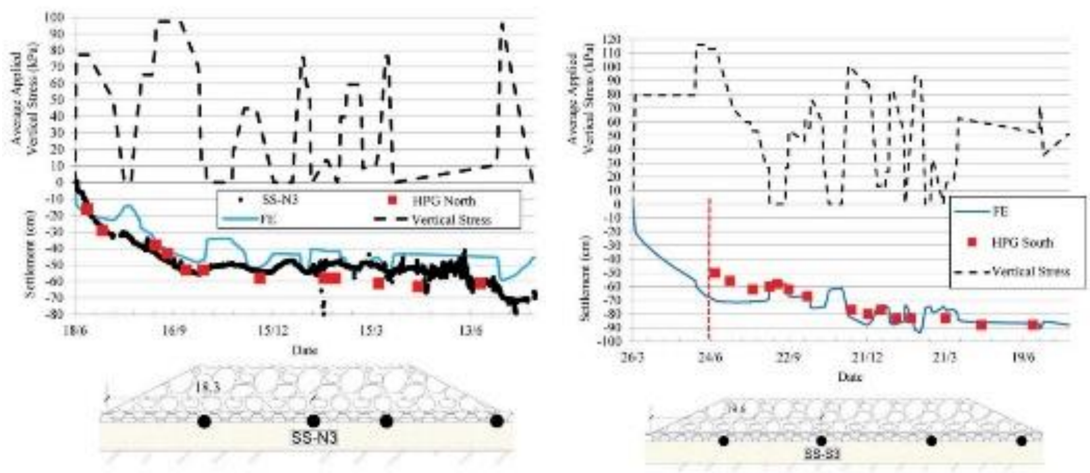
5.1. Vertical displacements

Here, the field data from the settlement sensors and the profilometers, with their locations shown in Figure 3, are contrasted with the settlements calculated by the numerical analyses.The settlements measured by the northern profilometer (HPG) and settlement sensor SS-N3 (northern section) are shown in Figure 5(a), together with the outcomes of the numerical analysis (FE - Finite Element). The pellet stack in the north is also displayed to show the average vertical stresses it applies. The findings demonstrate that the numerical analysis accurately anticipated the settlement as determined by the profilometer and settlement sensors.

Figure 5(b) also displays the settlement data for the southern profilometer, measured at the SS-S3 position, together with the outcomes of the numerical analysis. The sensor data are not displayed because to some technical issues with the southern settlement sensors, and the profilometer measurements started only 90 days later (vertical dashed line in Figure 5(b)).

As can be observed, the size and trend of the settlements over time are reasonably consistent between the findings of the numerical analysis and the data supplied by the profilometer in the southern stockyard. This tendency was seen throughout all of the profilometer readings, confirming the usefulness of the numerical model for predicting ground settling in composite materials. The computational analysis also revealed heave displacements caused by the unloading of the ore stacks (i.e., a reduction in the imposed vertical tension), which are also evident in the measured data by the profilometer and settlement sensors shown in Figure 5. This tendency indicates that the field response and the applied numerical model are generally consistent. Figure 6 compares the settlements from the north and south profilometers with the outcomes of the numerical study. It has been shown that the outcomes of the numerical analysis accurately anticipate the direction of the measured settlement in the southern and northern portions, as well as the heave displacements that occur during the unloading of pellet stacks. Particularly for the profilometer in the southern portion, the numerical findings were extremely similar to the field data.

The anisotropy and heterogeneity of the clay layers, as well as stack heights, which were not accurately represented in the geomechanical model, may be the cause of the disparity in the magnitude of the projected and measured settlements in the northern section.



(a) (b)

Figure 5. Development of measured and predicted settlements versus time: (a) northern stack; (b) southern stack.

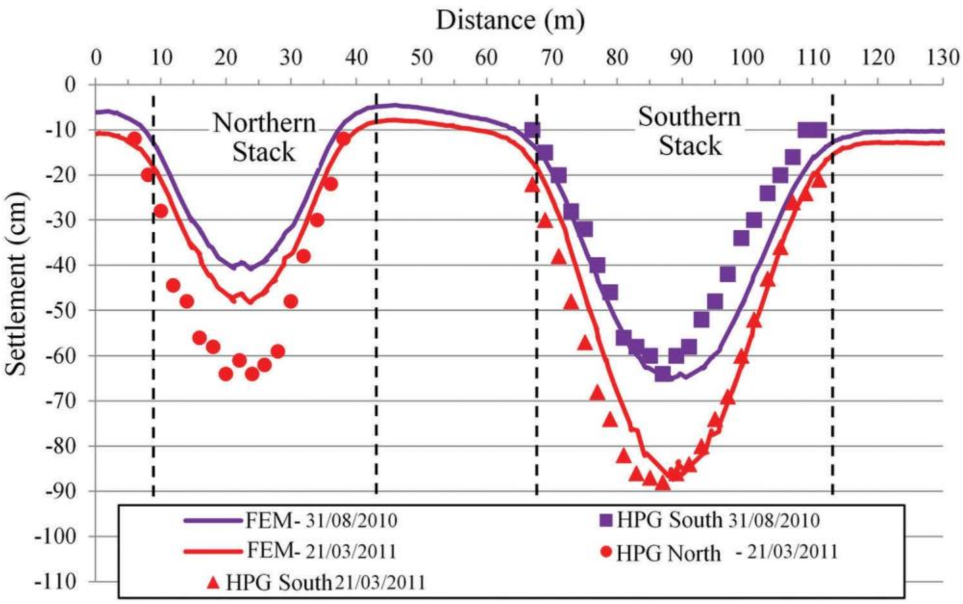


Figure 6. Comparison of measured and predicted settlement along the section studied.

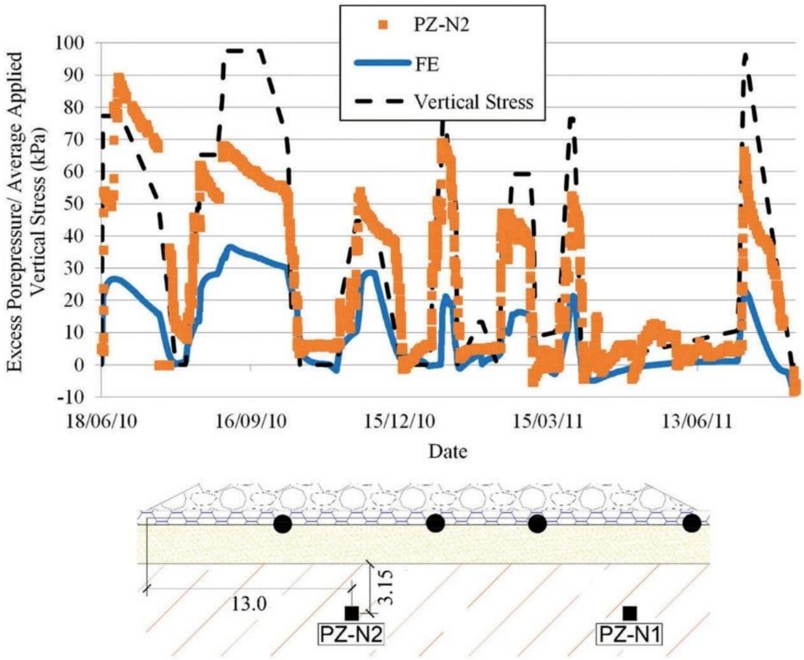


Figure 7. Comparison of measured and predicted excess pore pressure in the middle of clay Layer 1 – PZ-N2.

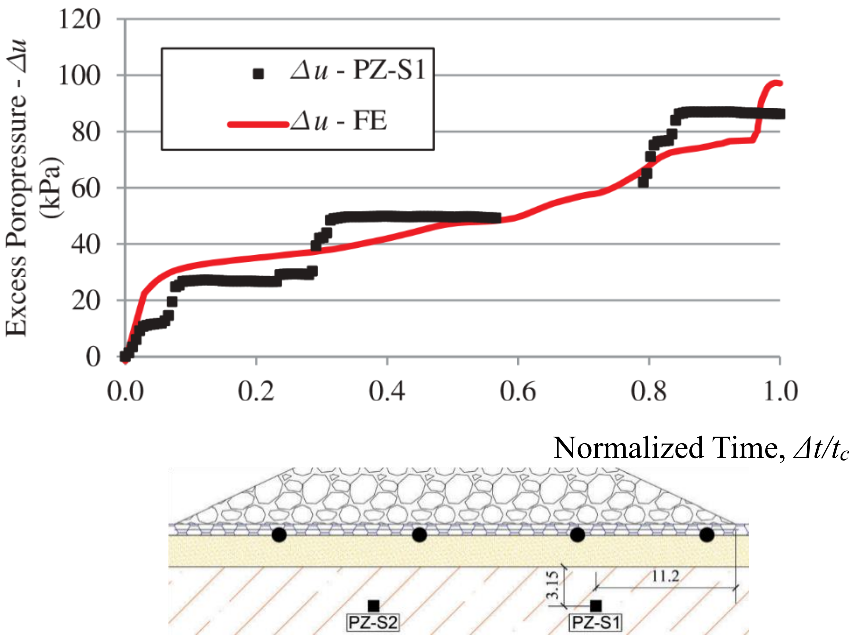


Figure 8. Excess pore pressure due to a quick load: comparison of measured and predicted results.

5.2. Excess pore pressures

In Figure 7, the average values of applied vertical stress are presented with the excess pore pressures determined by PZ-N2 numerical analysis (FE). This confirms that the hypothesis used to determine the height of the stack of pellets and average applied vertical stress was accurate. The surplus pore pressure was measured by the piezometer PZ-N2 and closely varied with the average applied vertical stress. The use of an average vertical stress throughout the whole length of the stack in the load calculation resulted in a disparity between the average applied vertical stress and the measurement. However, there are variations in the height of the stack in the field, particularly when stacks are loaded and unloaded, which may or may not have taken place close.

The anticipated curve of extra pore pressure has a similar pattern to the field data, as seen in Figure 7. The peak excess pore pressures determined by FE analysis are, however, lower than those determined by the more central PZ-N2 piezometer. This variation can result from the minimum time of Due to the data being accessible from the Stockyard, 1 day was used in the numerical analysis for each load stage. At this point, While quick loading in the field can produce stacks of pellets in a couple of hours, there is limited time for the dissipation of excess pore pressure during stack creation. The dissipation of excess pore pressure at the loading stage has already happened in the numerical analysis.

Based on the data from the piezometers, specific loading phases with the true loading timings were designed in order to build the highest excess pore pressure during the loading phases. After these phases, however, attempts to carry on with the numerical studies failed as a result of numerical convergences. For easier data comparability, the time period employed in these numerical studies and the actual field data (t) were normalised by the construction time (tc). Figure 8 displays the outcomes at the PZ-S1 site in the southern stack, together with instrumentation data collected at the precise instant the southern stack attained its highest point. The data from PZ-S1 and the FE findings exhibit good agreement, demonstrating that numerical analysis can reproduce peak values of excess pore pressure.

6. Conclusions

This study examined the behaviour of a foundation for a coal/ore stockyard that was separated into north and south sections and reinforced with stone columns. A complementary numerical analysis was carried out, and the outcomes were compared to the field readings the apparatus had produced. The following is a summary of the key findings:

- Overall, the finite element analysis findings were in good agreement with the field measurements, especially when it came to vertical deformation and the development of excess pore pressure during loading and unloading phases. It can be said that the geotechnical characteristics of the soft clayey strata discovered during the site study were fairly trustworthy when employed in numerical analysis to forecast how the ground reinforced with stone columns would behave when subjected to stage loading.
- The settlement measured by the profilometer and the point settlement sensors positioned beneath the load stacks were found to be in good agreement. However, the profilometer revealed that the unloading of pellet stacks, which the numerical analysis also correctly anticipated.
- The average values of the applied load were found to be quite near to the development of excess pore pressure as seen by piezometers during the loading phases. This behaviour shows a nearly quasi-undrained situation in any load application stage and the accuracy of the hypothesis that was utilised to determine stack height. The numerical analysis's findings also showed that the two-dimensional plane strain model, which turned the columns into walls, was able to accurately anticipate the behaviour of the examined portion.

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