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Contributions to the experimental investigation and numerical description of soil cyclic behavior

Příspěvky k experimentálnímu zkoumání a numerickému popisu cyklického chování zemin

Summary of the Doctoral thesis
Autoreferát dizertačnej práce

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Abstrakt

Tato disertační práce představuje některé příspěvky k experimentálním důkazům a numerickému popisu cyklického chování půdy. Začíná rozsáhlou experimentální databází malajského kaolinu a zbraslavského písku. V každé databázi bylo provedeno a analyzováno několik monotónních a cyklických zkoušek s ohledem na širokou škálu počátečních podmínek a zatěžovacích charakteristik. Na malajském kaolinu byl zkoumán zejména vliv amplitudy deviatorického napětí, plasticity zeminy, počátečního poměru napětí, odvodněného cyklického předpětí a posloupnosti balíčků cyklů s různými amplitudami deviatorického napětí. Na zbraslavském písku byl zkoumán vliv amplitudy deviatorického napětí a různých typů odvodněného nebo neodvodněného cyklického předpětí. Experimentální databáze budou volně dostupné na webových stránkách soilmodels.com.

Práce dále představuje podrobnou analýzu a srovnání predikčních možností, výhod a omezení některých nejznámějších a nejpokročilejších konstitutivních modelů pro cyklické zatěžování jemnozrnných zemin: Anamnéza, anizotropní hypoplasticita s ISA, SANICLAY-B a model

třípovrchového kinematického zpevnění. Následně jsou podrobně popsána a diskutována charakteristická omezení čtyř pokročilých konstitutivních modelů pro hrubozrnné zeminy: hypoplasticity s mezikrystalovou deformaci, hypoplasticity s ISA, SANISAND a SANISAND-MSf.

Dále je představena vylepšená verze modelu intergranulární deformace podle Niemunise a Herleho (1997). Představuje lepší možnosti při předpovídání deformace a/nebo akumulace pórového tlaku vody při cyklickém zatížení. Nakonec byly simulovány dvě případové studie sestávající z monopilu a trojnožky vystavených několika epizodám cyklického zatížení. Analýzy se zaměřují na mechanismus jejich deformace a schopnosti uvažovaných konstitutivních modelů reprodukovat výsledky odstředivého zatížení.

Abstract

This dissertation presents some contributions to the experimental evidence and numerical description of soil cyclic behavior. It begins with comprehensive experimental databases on Malaysian kaolin and Zbraslav sand. In each database, several monotonic and cyclic tests were performed and analyzed considering a wide range of initial conditions and loading characteristics. In particular, the influence of the deviatoric stress amplitude, soil plasticity, initial stress ratio, drained cyclic preloading and sequence of packages of cycles with different deviatoric stress amplitudes was investigated on Malaysian kaolin. On Zbraslav sand, the influence of the deviatoric stress amplitude and different types of drained or undrained cyclic preloadings was investigated. The results of the experimental databases will be freely available at the soilmodels.com website.

The thesis further presents a detailed analysis and comparison of the prediction capabilities, advantages and limitations of some of the most well-known and advanced constitutive models for cyclic loading on fine-grained soils: Anamnesis, anisotropic hypoplasticity with ISA, SANICLAY-B and three

surface kinematic hardening model. Subsequently, the characteristic limitations of four advanced constitutive models for coarse-grained soils: hypoplasticity with intergranular strain, hypoplasticity with ISA, SANISAND and SANISAND-MSf are detailed and discussed.

An improved version of the intergranular strain model by Niemunis and Herle (1997) is further presented. It presents improved capabilities in the prediction of strains and/or pore water pressure accumulation under cyclic loading. Finally, two case studies consisting of a monopile and a tripod subjected to multiple episodes of cyclic loading were simulated. The analyses focus on their deformation mechanism and the capabilities of the considered constitutive models to reproduce the centrifuge results.

CONTENT

1. Introduction

A wide range of geotechnical structures are subjected to episodes of cyclic loading. Among them, we can for example mention onshore and offshore foundations subjected to environmental loadings, pavements subjected to traffic loading, filling-emptying cycles on silos and water tanks, among many others. In order to develop constitutive models or numerical tools that accurately reproduce the soil behavior on the aforementioned geotechnical problems, a deep understanding on how soil behaves under cyclic loading is necessary. This behavior is, however, not trivial since non-linearity, small strain stiffness, stiffness degradation, cyclic hysteresis and recent stress history play a significant role.

In order to develop, validate and improve advanced constitutive models for their further application on boundary value simulations, high quality experimental databases on different types of soils are necessary. Such databases should include a wide range of monotonic and cyclic tests on "standard" conditions but also tests with more complex conditions aiming to represent real loading scenarios.

From the numerical point of view, although several constitutive models have been developed, their performance is usually accurate only on a restricted range of initial conditions and loading characteristics. Therefore, a critical review of their performance, advantages and limitations is crucial to have confidence before their application on boundary value simulations.

This dissertation presents comprehensive experimental databases on Malaysian kaolin and Zbraslav sand. The work also includes a detailed review and discussion of the prediction capabilities, advantages and limitations of some of the most well-known and advanced constitutive models for cyclic loading on coarse-grained and fine-grained soils. The thesis further presents an improved version of the intergranular strain model by Niemunis and Herle (1997). The extended model presents improved capabilities in the prediction of strains and/or pore water pressure accumulation under cyclic loading. Finally, two case studies consisting of a monopile and a tripod subjected to multiple episodes of cyclic loading were simulated. The analyses focus on their deformation mechanism and the capabilities of the considered constitutive models to reproduce the centrifuge results.

2. Aims of the study

The work presented in this thesis have been funded by the Czech-China bilateral project LTACH19028 Inter-Excellence (Inter-Action) entitled *Failure mechanism and hazard mitigation of wind turbine structure subjected to extreme environmental loadings*. Therefore, the aim of the thesis follows from the objectives of the project, which are summarized as: I) develop experimental databases in coarse-grained and fine-grained soils to improve the understanding of the soil mechanical behavior under cyclic loading. II) Inspect the capabilities and limitations of some advanced and “popular" constitutive models under cyclic loading. With this, hints are provided to end-users about in which boundary conditions the models are reliable. III) propose extensions of the constitutive models based on the found limitations, and IV) apply some of the constitutive models on boundary value simulations of foundations for offshore wind turbines.

3. Material and methods

In this thesis, the experiments were performed considering two different soils. The first one corresponds to Malaysian kaolin. It was purchased as a dry powder from the company "Kaolin (Malaysia) SDN BHD" and presents a specific gravity $G_s = 2.71$, a liquid limit $LL = 65\%$, a plastic limit $PL = 40\%$, and therefore a plasticity index $PI = 25\%$. According to the position of the kaolin in the Casagrande plasticity diagram, it is classified as a silt with high plasticity (MH). All triaxial samples were prepared by mixing the kaolin dry powder with distilled water having an initial water content of $w = 1.8LL$. The slurry was then deposited in a plexiglas cylinder with diameter $d = 50$ mm. Subsequently, it was pre-consolidated by gradually increasing the axial stress until a maximum value of 100 kPa. After the pre-consolidation, samples were cut out with a height $h = 100$ mm (a high-to-diameter ratio of 2) and mounted into the triaxial device. Samples were sandwiched with porous stones and filter papers. All samples were water-saturated with a back pressure of 500 kPa, which always provided Skempton's *B*-coefficient greater than 0.98. All triaxial tests were performed at a controlled temperature of 20° C. All cyclic triaxial tests were performed with a triaxial cyclic testing system (ELDYN) with capabilities of 5kN - 5 Hz from the company GDS. One of the advantages of using this specific kaolin is that it has been widely implemented in many geotechnical model tests and is the standard fine-grained soil used in many geotechnical centrifuge facilities. Therefore, a database on this material is valuable for the calibration of constitutive models and their further application on boundary value simulations.

The second testing material corresponds to Zbraslav sand. It was collected from a borrow pit in the district of Zbraslav, Prague, Czech Republic. This sand presents the following characteristics: specific gravity $G_s = 2.65$, maximum void ratio $e_{max} = 0.893$, minimum void ratio $e_{min} = 0.52$, mean diameter $D_{50} = 0.531$ mm, uniformity coefficient $C_u = 3.19$ and coefficient of curvature $C_c = 0.98$. According to the Unified Soil Classification System (USCS) the tested sand is classified as poorly graded sand (SP). Zbraslav sand presents almost no fines (< 1%). The sand grains present subangular shape. Samples for triaxial tests were prepared following the procedure described by Wichtmann et al. (2020), in which the dry material was deposited in its loosest state and was further compacted by tapping to achieve the desired initial relative density. Samples were prepared considering a diameter of 50 mm and a height of 100 mm (a high-to-diameter ratio of 2). All samples were sandwiched with porous stones and filter papers.

The saturation procedure for triaxial samples is equivalent to the one reported by Wichtmann (2016) and Lade (1972). Initially, dry samples were placed in the triaxial cell and were saturated with CO_2. Then, the back and cell pressures were increased to 10 and 20 kPa, respectively. Under these stresses, samples were saturated with deaerated water (i.e. we let deaerated water flow through the sample under these stresses very carefully, in order not to change the stress states). Subsequently, the back and cell pressures were increased to 500 and 510 kPa, respectively. Finally, we did flushing again under this back pressure (i.e. we carefully let deaerated water flow through the sample). The aforementioned procedure always provided Skempton's coefficient $B \geq 0.99$, which confirms a full water-saturation. After the saturation, samples were isotropically consolidated at the corresponding initial mean effective stress of each test.

Most elemental and finite element simulations were performed with a high capacity 64 thread 128 GB RAM Threadripper computer. All finite element simulations were performed using the freely-available finite element software Tochnog Professional.

4. Results and discussion

4.1. Laboratory tests

Seven undrained cyclic triaxial tests were performed on normally consolidated samples with isotropic consolidation and variation of the deviatoric stress amplitude. A typical result on test UCT4 with deviatoric stress amplitude of 50 kPa is given in Figure 4.1. The results suggest that the liquefaction state ($p = q = 0$) was not reached with the kaolin, see Figure 4.1a. In addition, the kaolin does not exhibit eight-shaped effective stress paths in the final phase of tests with isotropic consolidation, which are are usually observed on fine-grained soils with low activity but "erased" with increasing activity (Hyodo et al. 1999). The material does not show any significant inclination of the effective stress paths. The accumulated pore water pressure, presents three very different accumulation rates, see Figure 4.1b. At the beginning of the test, a high rate of pore water pressure accumulation is observed (stage “A”). Then, the accumulation rate decreases and gets almost constant during stage “B”. Finally, at the end of the test, the accumulation rate increases again during stage “C”. The stress-strain behavior is analyzed in the $q - \varepsilon_1$ space in Figure 4.1c. The results suggest that the vertical strains accumulate at a very small rate until reaching the stage “C” (see Figure 4.1b), in which the double strain amplitude grows quickly with each subsequent cycle until reaching the defined failure criterion, see Figure 4.1d.

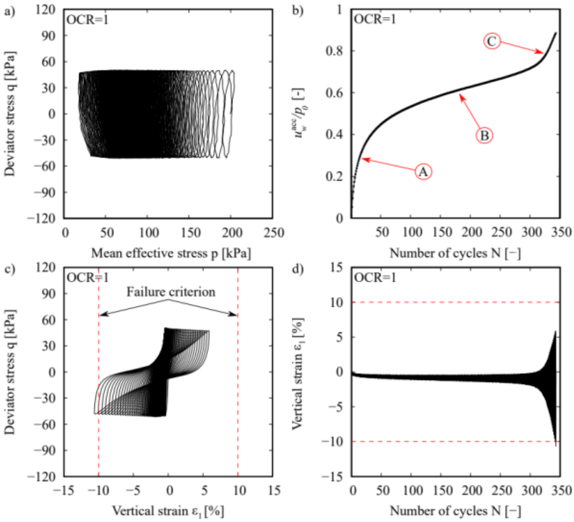

Fig. 4.1 Typical results of an undrained cyclic triaxial test with isotropic consolidationand constant deviatoric stress amplitude of 50 kPa

The validity of Miner's rule under undrained conditions is now evaluated. For that purpose, three undrained cyclic triaxial tests were performed on normally consolidated samples with isotropic consolidation and packages of cycles in different sequences. Each package of cycles considered 100 cycles, for a total of 300 cycles in each test. The magnitudes of the deviatoric stress amplitudes were carefully selected, in order to avoid cyclic mobility. The experimental results of these tests are summarized in Figure 4.2. The results suggest that the order of the packages of cycles plays a major role in the final accumulation of pore water pressure and strains (i.e. Miner's rule is not valid under undrained conditions). Packages with decreasing order of the deviator stress amplitude produce the higher accumulation, while packages with ascending order of loads produce the lowest accumulation.

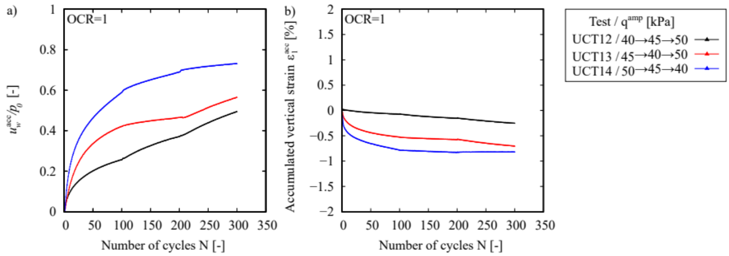

Fig. 4.2 Typical results of an undrained cyclic triaxial test with isotropic consolidationand variable deviatoric stress amplitude

A typical undrained cyclic triaxial test on Zbraslav sand is presented in Figure 4.3. Some observations regarding the experimental results are worthy to remark: the final stage of tests with isotropic consolidation (cyclic mobility phase) exhibit butterfly-shaped effective stress paths. Three remarkably different accumulation rates of the relaxation of the mean effective stress are observed, see Figure 4.3c,d. Initially, a fast accumulation of the pore water pressure is evidenced (stage “A”). Later, during stage “B” the pore water pressure accumulates at a lower rate. At the end of the test, the accumulation rate increases again during stage “C”. The analysis of the stress-strain behavior is presented in Figure 4.3e. The results also indicate that accumulation of axial strains grows at a very small rate until reaching the stage “C”, after which the axial strain amplitude quickly grows with each subsequent cycle until reaching the defined failure criterion of $|\varepsilon_1| = 10\%$, see Figure 4.3b,d,f.

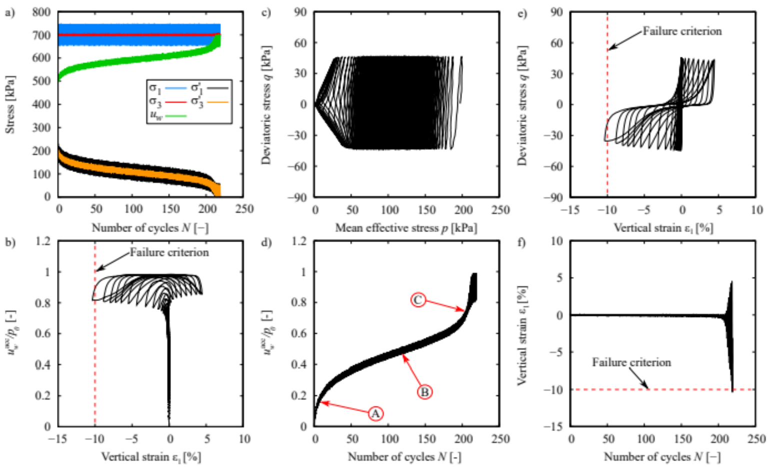

Fig. 4.3 Typical results of an undrained cyclic triaxial test with isotropic consolidation and constant deviatoric stress amplitude of 45 kPa

4.2. Analysis of several constitutive models and improvement of the IS model

An analysis of the performance, advantages and limitations of four advanced constitutive models for fine-grained soils was performed. Some typical simulation results are presented in Figure 4, in which the accumulation of pore water pressure under undrained cyclic loading is analyzed. It is clear that only the CAM and AHP+ISA models are able to properly predict the pore water pressure accumulation due to the rotation of the flow surface and the evolution of function χ, respectively. Pore water pressure generation of the critical state-based elasto plastic models (SANICLAY-B and A3-SKH models) predict negligible plastic volumetric strains not only at the critical state defined by mean effective stress, void ratio and stress ratio, but even if stress ratio is lower than the one corresponding to critical state. Therefore, pore water pressure accumulation stops as soon as mean effective stress reaches critical state value (for current void ratio). This behavior limits the applicability of these models on problems with cyclic loading, whereby the accumulation of excess pore water pressure is essential.

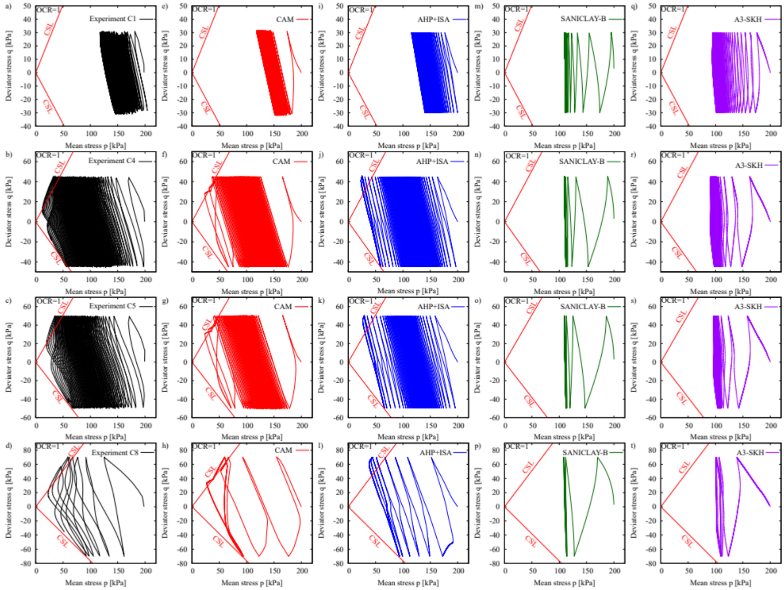

Fig. 4.4 Simulations of undrained cyclic triaxial tests C1-C8 on normally consolidated samples of Karlsruhe kaolin with isotropic consolidation and variation of the deviatoric stress amplitude

Seven characteristic limitations of constitutive models for coarse-grained soils were carefully detailed an discussed. For exmplification purposes, two hypoplastic and two bounding surface plasticity models were considered. The reasons underlying this incoorrect performance were pointed out and suggestions on how to address these isuees were discussed.

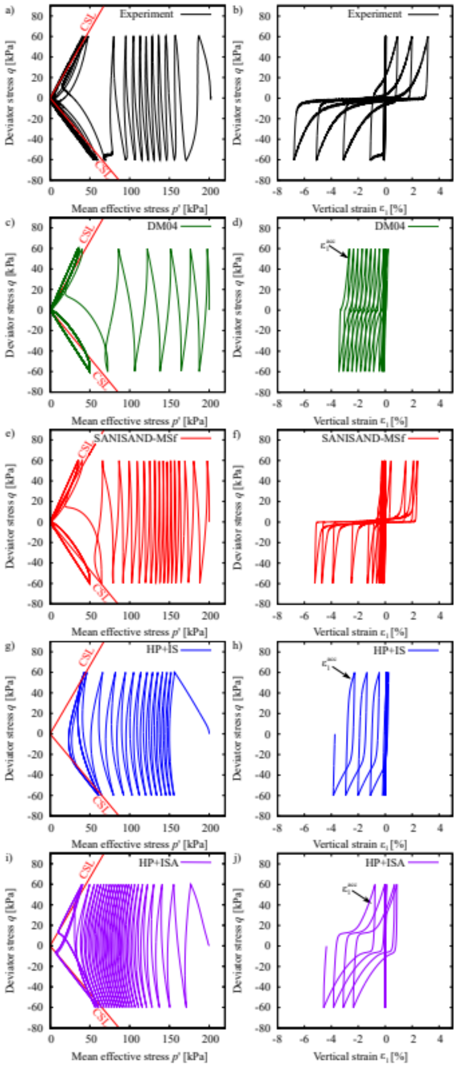

Fig. 4.5 Undrained cyclic triaxial test on Karlsruhe fine sand. Medium density sample with isotropic consolidation and stress cycles

As presented in Figures 4.1 and 4.3, the accumulation rate on undrained cyclic tests with isotorpic consolidation presents three different stages. Unfortunately, neither the basic intergranular strain model by Niemunis and Here (1996) nor the modified version by Wegener and Herle (2014) are able to qualitatively reproduce this behavior. A simple modification was proposed to reproduce this behavior by turn parameter χ into a function that depends on the new parameters χ_0 and χ_{max}. The results with the proposed modification are presented in Figure 4.6 in color green and suggest a substantial improvement in the prediction of the pore water pressure accumulation under cyclic loading.

4.3. Boundary value simulations

This section presents the back-calculation of the centrifuge tests performed by Lai et al. (2020). The centrifuge tests consisted of large diameter monopiles, typical of offshore foundations, embedded in a soft normally consolidated kaolin and subjected to either monotonic or cyclic lateral loading. The geometric layout of the model is illustrated in Figure 4.7. A comparison between experimental and numerical force-displacement curves for each of the cyclic loading episodes is presented in Figure 4.8. The numerical results suggest a satisfactory prediction of the normalized lateral displacements. However, a small underestimation is revealed in the second and third stages, where the experimental results show a small increase in stiffness while the model predicts an almost identical stiffness regardless of the loading stage.

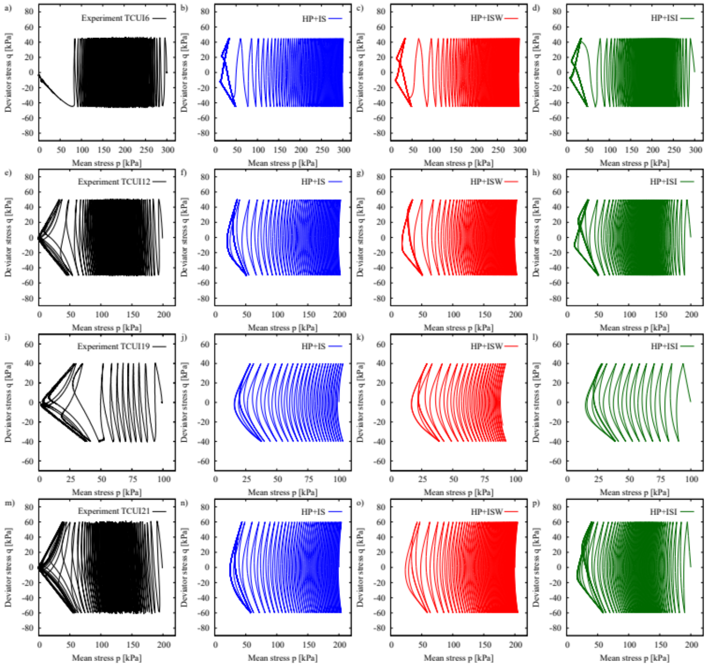

Fig. 4.6 Simulations of undrained cyclic triaxial tests TCUI6,12,19,21 with isotropic consolidation and variation of the initial density and deviatoric stress amplitude

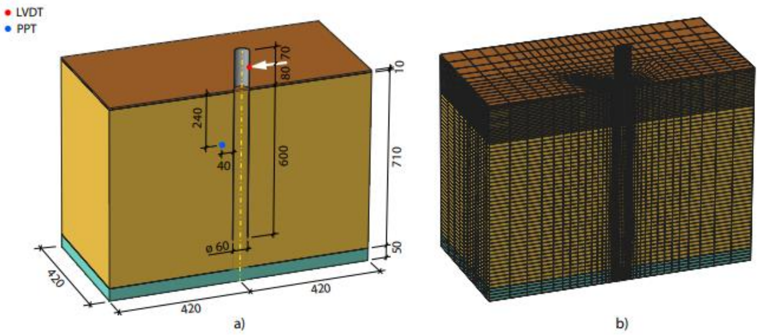

Fig. 4.7 Geometry of the FEM model: a) main parts, dimensions and locations of the transducers, b) mesh discretization (in model scale, unit: mm).

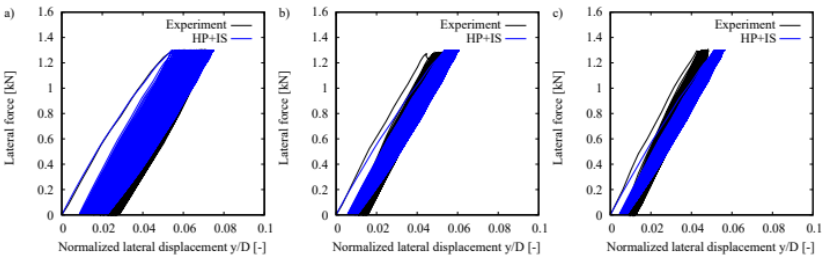

Fig. 4.8 Cyclic load-displacement curves of the monopile during the stages of cyclic loading: a) first episode, b) second episode, c) third episode

5. Conclusions

This cumulative dissertation presented some contributions to the experimental investigation and numerical description of soil cyclic behavior. Initially, an experimental database on a high-plasticity Malaysian kaolin under monotonic and cyclic loading was presented. The results of undrained cyclic triaxial tests suggested a remarkable influence of the deviatoric stress amplitude and the initial stress ratio in the accumulation rates of pore water pressure and strains, and therefore, in the number of cycles to reach failure conditions. Undrained cyclic tests performed considering packages of cycles in different sequences suggested that Miner's rule is not valid under undrained conditions. A modified Stewart's approach was proposed to analytically estimate the accumulated pore water pressure and strains on the tests with variable loading magnitudes based on single-amplitude tests.

An experimental database on Zbraslav sand under monotonic and cyclic loading was presented. The results of undrained cyclic triaxial tests with isotropic consolidation suggest a remarkable influence of the deviatoric stress amplitude in the accumulation rates of pore water pressure and strains. In addition, their final phase is characterized by butterfly-shaped effective stress paths accompanied with a progressive increase of the axial strain amplitude with each subsequent cycle.

A detailed review of the prediction capabilities, advantages and limitations of four advanced constitutive models for fine-grained soils, namely: CAM, AHP+ISA, SANICLAY-B and A3-SKH was presented. The inspected constitutive models performed well under undrained monotonic loading. However, many issues were found on the simulations of cyclic tests. The accumulated pore water pressure was well reproduced by the AHP+ISA and CAM models. The pore water pressure accumulation with the SANICLAY-B and A3-SKH stops as soon as mean effective stress reaches critical state value. This issue limits the prediction capabilities of these models on boundary value problems whereby the prediction of pore water pressure is of interest. Variation of the initial conditions such as the initial stress ratio, OCR and strain-cycles suggest a less accurate performance with all models.

Seven characteristic limitations of models for cyclic loading for sands were discussed. They were exemplified with the simulations of two models from the bounding surface plasticity family and two models from the hypoplastic family. The limitations are summarized as: I) overshooting and undershooting, II) one-way ratcheting in strain accumulation on undrained cyclic triaxial tests, III) inadequate modelling of cyclic liquefaction strength curves, IV) problems to reach the liquefaction attractor on simulations on very dense samples, V) unrealistic plastic strain accumulation when dealing with cyclic loading with closed stress loops of small amplitude, VI) inadequate oedometric loading stiffness, VII) inadequate reproduction of drained preloading affects.

A simple modification to the original intergranular strain model by Niemunis and Herle (1997) was proposed. The modification consists on an additional mechanism to reduce the strain accumulation rate on paths with repetitive cycles of small strain amplitudes. The improvement allows to adapt its response at cycles of larger strain amplitudes, where the strain accumulation rate is expected to increase. The modified model was able to predict with accuracy: a) the number of cycles required to reach failure conditions, b) the evolution with number of cycles of the normalized accumulated pore water pressure under undrained conditions, and c) the evolution with number of cycles of the accumulated strains under drained conditions.

Finite element simulations of the monopile tests reported by Lai et al. (2020) were performed. The considered model was able to accurately reproduce the magnitude of the ultimate lateral monotonic loading. In addition, the cumulative peak and residual displacements of the monopile under cyclic loading were also well predicted. The cyclic lateral force-displacement response was reasonably well predicted. The centrifuge tests suggested an increase of the unloading stiffness ratio after each subsequent stage of reconsolidation which was not reproduced by the model.

On the other hand, finite element simulations were performed to provide an insight into the deformation mechanism of tripod foundations subjected to cyclic loading. The results suggest that the foundation initially rotates according to the direction of the applied load. Then, after several cycles, the uneven progressive change in soil state leads to a significant change in stress state around both buckets. Finally, when the cumulative settlement of the pulled bucket overcome those for the pushed one, the whole foundation change rotation to the one against loading direction producing the so-called "self-healing" effect.

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