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RESEARCH ARTICLE

Role of Numerical Modelling in Understanding Soft Soil Behaviour Under Construction Load

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ABSTRACT - This study reviewed the application of numerical modelling in soft soil subjected to different construction loads. Various models to simulate clayey soils were discussed which included the Modified Cam Clay (MCC) model and Elastic-viscoplastic (EVP) model. The MCC model was useful for predicting soil settlement and pore water pressure through a 2D FEA model, but the EVP model performed more accurately in predicting soil settlements, lateral displacements and excess pore water pressures. A 3D numerical model using HSM for pile raft foundations accurately predicted the displacement under working loads based on the non-linear soil elasticity and plasticity. Case studies on tunnels constructed on soft soil were presented. Numerical simulations had been found reliable in simulating the settlements and slope stability. The findings emphasized the importance of numerical modelling to enhance design optimization in geotechnical aspects.

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1. INTRODUCTION

Construction loads applied on ground consist of static loads from embankments and foundations of buildings, and dynamic loading from earthquake, traffic, and rapid transit system [1]. Over the past few decades, rapid civilization advancement has led to the reduction of suitable sites for construction. Constructions on soft soils become more prevalent. [2]. Constructions on unimproved soft ground are susceptible to damages, instabilities, and failures due to the nature of soft soils [3, 4]. The physical properties of soft soils include high natural water content, high percentage of organic matter, high compressibility, low shear strength and low bearing capacity [1, 3-5]. In the context of Malaysia, the southern region of Peninsular Malaysia is extensively covered by soft soils with remarkably high-water content of 80% and above [6]. Multiple studies on soft soils have been conducted in Malaysia. For instance, in Pekan, Pahang [7, 8], in Pontian and Tanjung Pelepas, Johor [9] and in Batu Pahat, Johor [10]. Based on their studies, it is concluded that soft soils have low shear strength which are less than 25 kPa. Bearing capacity failure and excessive settlement are often caused by the low undrained shear strength of soft soil [11, 12]. According to the previous studies, it is significant to understand the behaviour of any construction constructed on soft soils. In this study, various methods and roles of understanding the behaviour of soft soils subjected to different type of loadings using numerical modelling will be reviewed and discussed.

2. METHODS AND MATERIALS

2.1 Embankment on Soft Soil Using 2D MCC Model

A fully coupled finite element consolidation analysis model was developed to simulate and compare the results of finite element analysis (FEA) model and field measurements of a test embankment on soft Ariake clay in Saga, Japan [13]. The real embankment for a highway construction on soft clay was under development during their study. Their model was based on 2D plane strain. Figure 1 illustrated the meshing and boundary conditions of the model. Both vertical and horizontal displacements were not allowed at the bottom boundary, while only vertical displacement was permitted at the side boundaries. The embankment and the soil layers underneath were defined using 8-nodes quadrilateral elements. The only difference was that the soil layers' elements had excess pore pressure degree of freedoms (DOF) at the 4 vertex nodes. In the FEA model, the soft clay was described using the Modified Cam Clay (MCC) model [14] and other soils including the embankment were represented by the elastoplastic model. The model was computed using the CRISP program [15].

Excess pore pressures, lateral displacements and settlements of the foundation soils were compared with the site measurements. Due to the influence of strain rate on the undrained shear strength, S_u and the strain-softening characteristics of clayey soils, S_u obtained from laboratory tests may not characterize the actual field conditions [13, 16, 17]. A correction factor for S_u of 0.85 [13] and μ factor from [16] was recommended to be utilized. The FEA model with modified S_u and *OCR* (Class-C2) predicted the excess pore pressures excellently, over-estimated the lateral displacements and under-estimated the settlements, but the overall performance of this model was considered good. The results from

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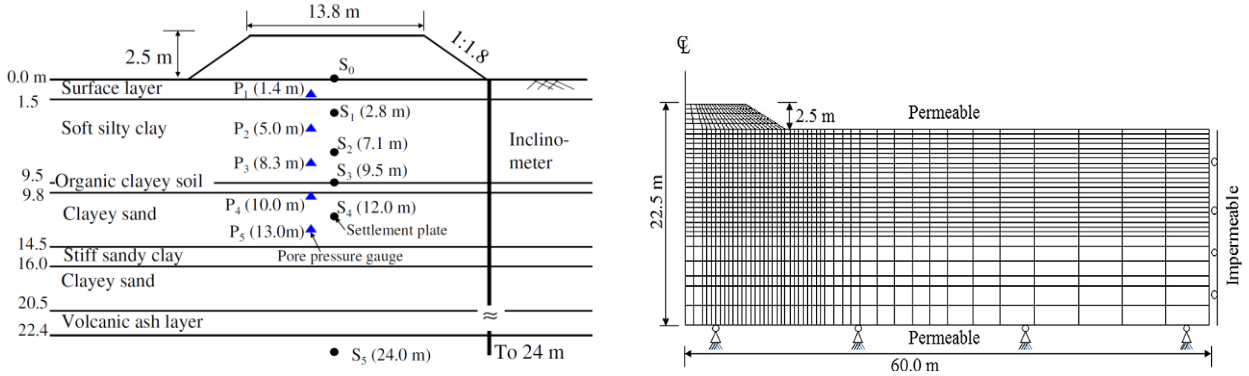
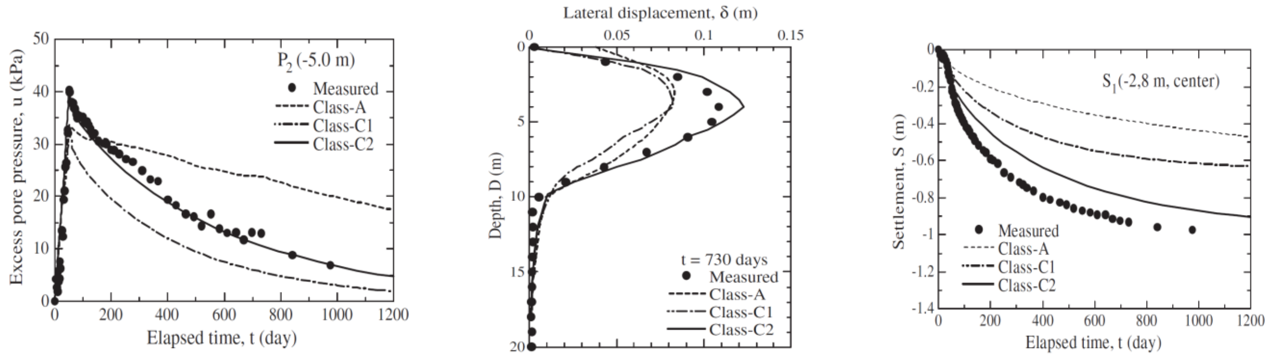
[13] are illustrated in Figure 2 below. In order to enhance the accuracy and reliability of FEA model in predicting soft soils under embankment, it was concluded that the model parameters must be able to represent the soils' physical properties, soil investigation must be conducted at the specific site location and the strength parameter, *M* for MCC model should be acquired from laboratory tests with suitable effective stress paths [13].



(a) Actual test embankment's cross section (b) Meshing and the boundary conditions of FEA model

Figure 1. Properties of the embankment on site and FEA model [13]



(a) Excess pore pressure at 5 m depth (b) Lateral displacement after 2 years (c) Settlement at 2.5 m

Figure 2. Comparison between measured field data and MCC FEA predictions [13]

2.2 Embankment on Soft Soil Using 2D EVP Model

A MCC model and an elastic-viscoplastic (EVP) soil model were implemented in the finite element program named SIGMA/W in Geostudio 2007 to simulate and compare the response of soft estuarine soil beneath an embankment which had undergone staged construction [18]. The EVP model shows similarity in some aspects to the model used by MCC [19]. The EVP model proposed by [18] was based on the idea that a complete description of the "state" of the soil was required to determine the deformation response of soil [20]. The "state" of the soil was defined as both the specific volume and stress levels of the soil [18]. The meshing of the FEA model was illustrated in Figure 3 below. Eight-node quadrilateral elements and plane strain conditions were assumed in this model. The unit weight of the embankment was increased to simulate the staged construction process. The soil parameters for the FEA model were obtained from the laboratory works reported in [21].

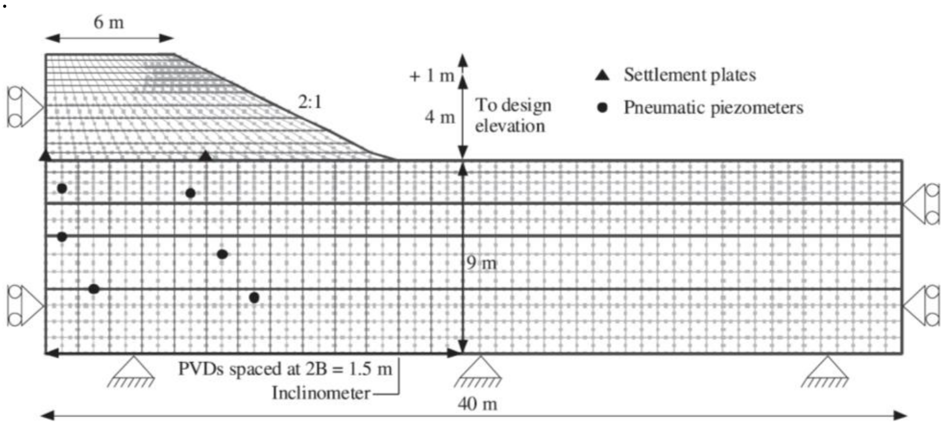
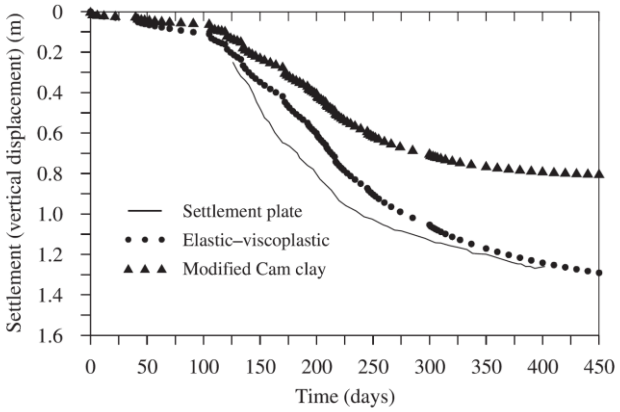
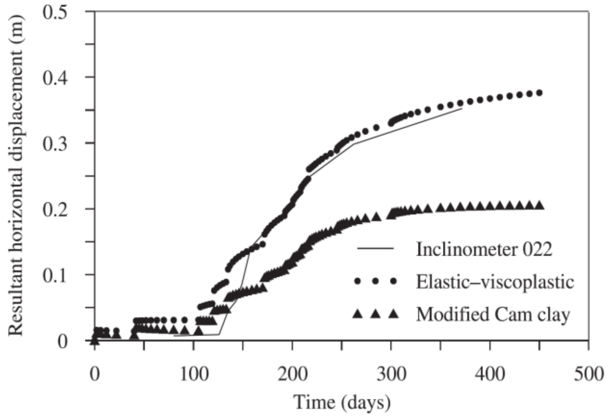


Figure 3. Meshing and properties of the FEA model, and instrumentation [18]

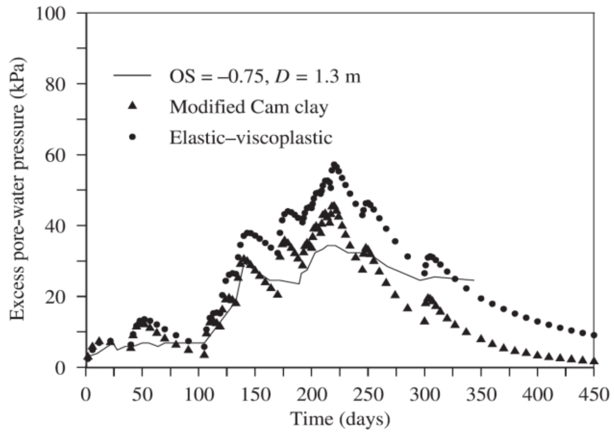
According to the results in [18], the EVP model performed better than the MCC model in predicting the surface settlement below the embankment, the horizontal displacement and the excess pore water pressure which are shown in Figure 4 below. Based on Figure 4(a), the MCC model started to diverge significantly from the actual measurements at day 108. This was because the elastic deformation transformed to viscoplastic starting at that point. Even though, the EVP model estimated the settlement more accurately than the MCC model, it still underestimated the actual surface settlement. The plastic strains generated by the EVP model were greater than the MCC model. In respect of horizontal displacement, the EVP model precisely captured the maximum horizontal displacement as shown in Figure 4(b). Conversely, the MCC model underestimated the horizontal displacement remarkably because it was difficult to use elastic-plastic model to predict the horizontal deformation of soils [22-24]. It was known that the precise simulation of excess pore water pressure was difficult due to multiple reasons [25]. The MCC model underestimated the excess pore pressure at most locations. However, the prediction by MCC model at a depth of 5.8 m beneath the embankment was accurate. Overall, the EVP model mapped the actual measurements from piezometers at most of the locations and depths. [18] highlighted that the characteristics of viscoplastic or time-dependent plastic became apparent between day 50 and day 100. Lastly, it was concluded that the EVP model performed better than the MCC model in simulating the deformation response of soft soils. The drawbacks of the EVP model were longer computing times, high computation cost and greater complexity [18].



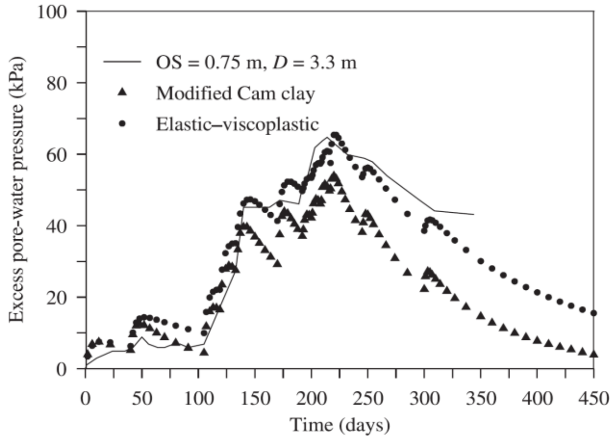
(a) Surface settlement at the centre of embankment



(b) Maximum horizontal displacement at the toe of embankment



(c) Excess pore pressure at 1.3 m depth below the centre of embankment



(d) Excess pore pressure at 3.3 m depth below the centre of embankment

Figure 4. Comparison of surface settlement, horizontal displacement and excess pore water pressure [18]

A ‘super embankment’ or ‘super levee’ is an excellent-quality embankment where no collapse can be permitted [26]. It was constructed along the Yodo River in Toroshima, Japan [27]. However, settlement and cracks were observed after 10 years of construction. Subsequently, investigation on this issue was carried out using the 2D FEA incorporating the EVP model introduced by [28] to simulate the behaviour of clay soils [27]. Fully saturated conditions were assumed to study the long-term consolidation behaviour of soft clay. The consolidation analysis consisted of two phases. The first phase analysed the original ground while the second phase assessed the improved ground. The EVP model proposed by [28] was a development of the rate-dependent model for saturated clay firstly developed by [29], which integrate the Cam-clay model [30] and viscoplasticity for the elasto-viscoplastic approach [31]. The model by [29] was enhanced by [28], considering the shrinkage of over-consolidated boundary surface and static yield surface. The detailed formulations of the modified EVP model implemented by [27] can be found in [27]. Eight-node and four-node quadrilateral isoparametric elements were used to model the displacements and pore water pressure. The dimensions and boundary conditions of the FEA model are shown in Figure 5 below. As shown in Figure 5, vertical displacements were allowed at the side boundaries while the bottom boundaries restrained both vertical and horizontal displacements.

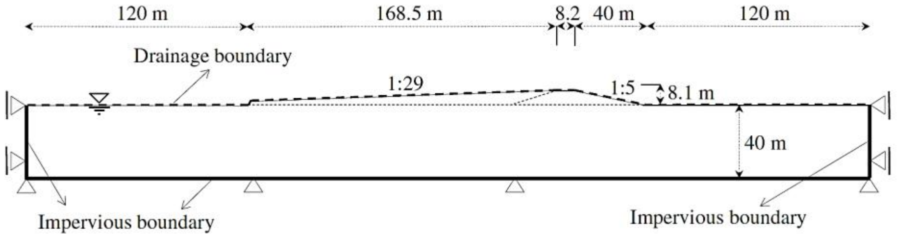
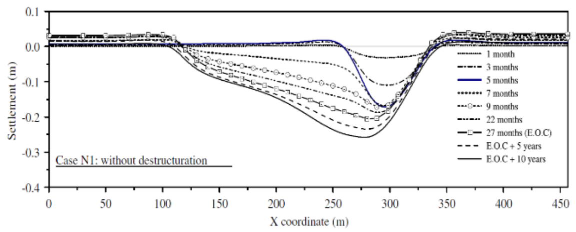
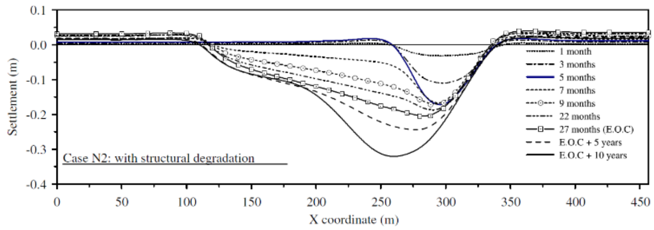


Figure 5. Dimensions and boundary conditions of the FEA model [27]

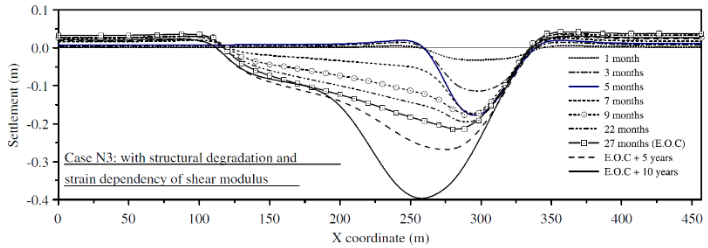
The long-term consolidation behaviour of soft soil due to the effect of the structural degradation and the strain dependency of elastic shear modulus were studied [27]. Structural degradation is related to the strain softening and viscoplastic strain while strain dependency of elastic shear modulus is related to the normalized shear modulus reduction function, G_0. G_0 is associated with the experimental constant, r [32], the strain-dependent parameter, α, void ratio [33] and others. The soil parameters for the EVP model were obtained from laboratory test. Undrained triaxial tests with various strain rates were conducted to acquire the viscoplastic parameters as stated in [28]. Three different cases were considered to assess the effects of soft clay destructuration in embankment consolidation. Case N1 did not consider the structural degradation or the strain dependency. Case N2 considered only structural degradation, while Case N3 considered both as shown in Figure 6 below. Greater settlements were noticed in during the loading process (before E.O.C) in Case N3 due to the strain dependency, while Case N2 and N3 showed significant increases in the settlement in the consolidation period due to strain softening in soft soils [27].



(a) Case N1



(b) Case N2



(c) Case N3

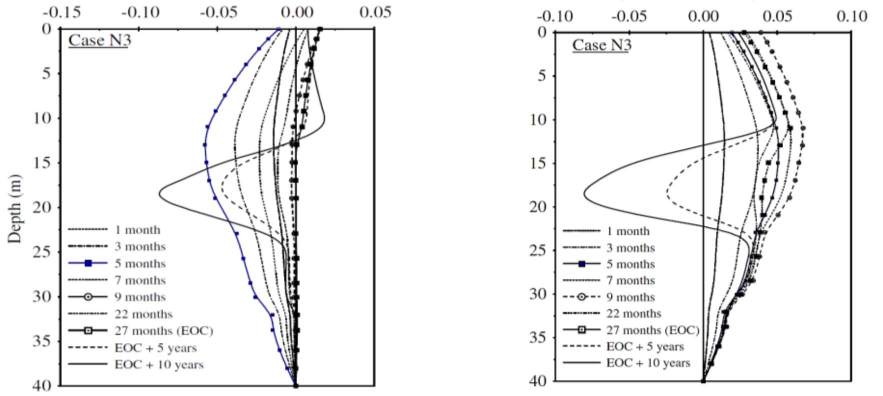
Figure 6. Ground level settlement of the ground [27]

The lateral displacements at the left and right-side toe are illustrated in Figure 7. At the left-side toe, the displacement shifted to the right, followed by movements to the right and eventually shifted back to the right following the dissipation of pore water pressure. At the right-side toe, the displacement had different behaviour compared to the left-side toe. This might be contributed by the different construction process. It was indicated that the large volumetric strain below the embankment led to the lateral displacements at the toe. The three cases showed similar excess pore water pressure variations during the construction stage. However, considerable differences appeared during the long-term consolidation. The excess pore pressure simulated in Case N1 experienced a reduction, while Case N2 and N3 increased due to the assignment of structural degradation parameter as discussed in [28]. An increase in pore pressure was caused by the increase in viscoplastic strain, strain localization and strain softening [27]. [27] concluded that the structural degradation had excessive effect on the pore water pressure after construction process and the strain dependency influenced the deformation of soil during construction and consolidation.

3. RESULTS AND DISCUSSION

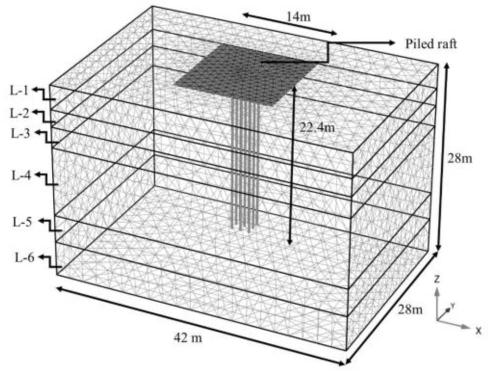
3.1 Foundation on Soft Soil Using 3D Numerical Model

Pile raft foundations in soft soils were studied by [34]. A 3D FEA model using Hardening Soil Model (HSM) was modelled using Plaxis 3D software. HSM was used because it gave precise displacements at working load conditions [35] and it was based on the theories of non-linear elasticity and plasticity. Furthermore, a centrifuge model developed by [36] was used to compare the results from the FEA model. The soft clay soils at Bogota, Columbia were represented using kaolin soil mixtures as proposed by [36]. The kaolin soft mixtures were prepared based on the procedures highlighted in [37]. Oedometer and triaxial tests were conducted to acquire soil parameters required. Figure 8(a) illustrated the geometry of the pile raft foundation and the soil layers while Figure 8(b) showed the soil parameters for modelling. Layer L-1 represented the crust, with over consolidated high plasticity clay while L-2 to L-6 represented the soft clay layers with high plasticity saturated clay.



(a) Left-side toe (b) Right-side toe

Figure 7. Lateral displacements at the toe of the embankment during consolidation [27]

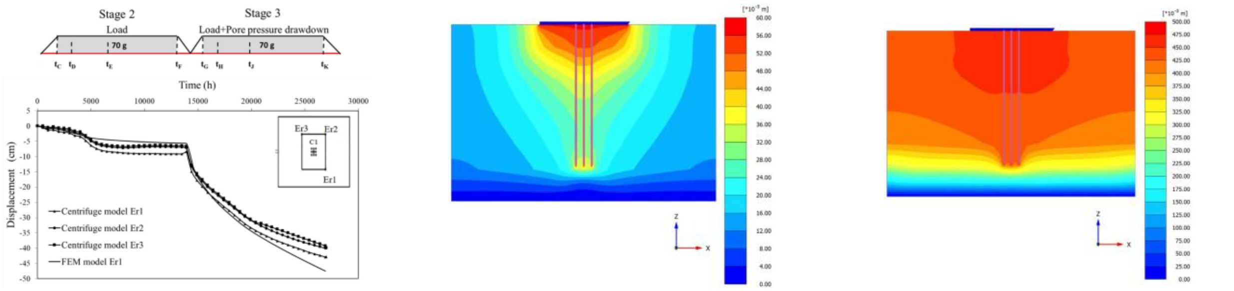


Element	Parameter	Value
Plate (Raft)	Unit weight	25 kN/m³
	Thickness	1.147 m
	Young's modulus	35 GPa
	Width	14 m
	Length	14 m
Embedded beams (Piles)	Unit weight	25 kN/m³
	Diameter	0.63 m
	Young's modulus	30 GPa
	Length	22.4 m
	Axial skin resistance	11.38 kN/m
	Base resistance	205 kN

(a) Geometry and meshing of the 3D FEA model (b) Soil parameters for FEA model

Figure 8. 3D FEA model and the soil parameters [34]

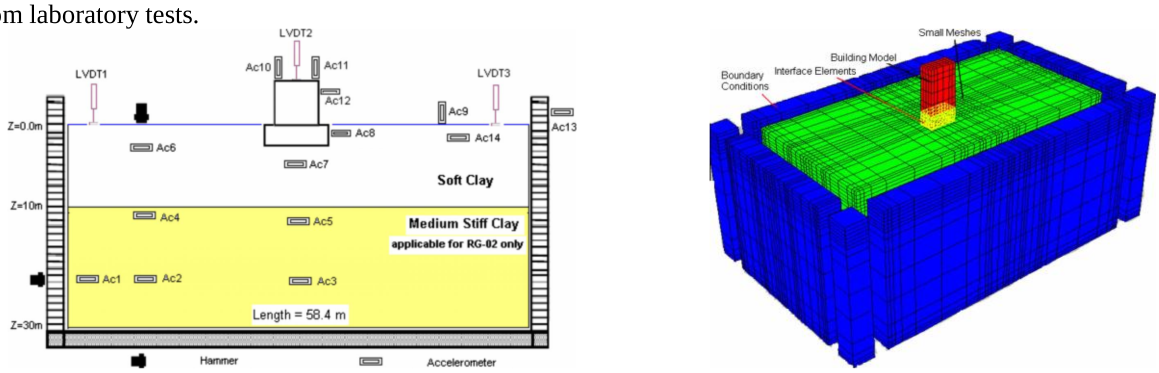
Based on the results in [34], the displacements measured from the centrifuge model were slightly lower than the FEA model. However, both the centrifuge and FEA model had very close paths and therefore represented the phenomena quite accurately as shown in Figure 9(a) below. Nevertheless, the FEA model over-estimated the displacement at position Er1. The results of vertical displacements obtained from the FEA model were presented in Figure 9(b) and 9(c). The settlements at the end of Phase 6 (consolidation with load and pore pressure reduction) were remarkably higher than the settlements at the end of Phase 2 (consolidation with load only). It was concluded that the combination of loadings from the foundation and pore water pressure drawdown could result in this dissimilar situation [34].



(a) Comparison of displacement between centrifuge and FEA model (b) Total displacement at the end of Phase 2 from FEA model (c) Total displacement at the end of Phase 6 from FEA model

Figure 9. Results obtained from the study by [34]

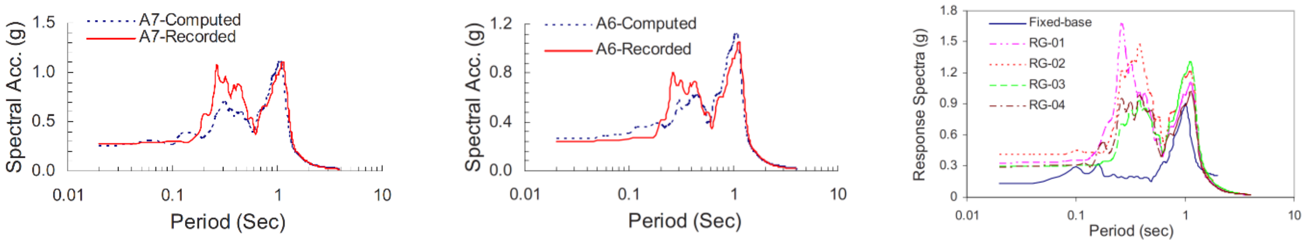
The FLAC3D program [38] which was a 3D finite difference program to study the behaviour of soils, was adopted by [39] to simulate the seismic response of rigid foundation on soft soil. For instance, peak acceleration, response spectra and amplification factors were discussed. Soil-structure interaction (SSI) and non-linear site response (SR) were related to the seismic response of structures on soft soil [40-42]. Hence, it is important to study the non-linear behaviour of soil on the structures' response [43]. The study by [39] compared the results obtained from 3D numerical model and the centrifuged model of foundation in soft soil developed by [44]. The centrifuged models in Figure 10(a) were subjected to shaking events and the results were recorded. The meshing of the 3D model was shown in Figure 10(b) below. Non-linear behaviour was simulated based on Mohr-Coulomb model in the 3D model and the related parameters were determined from laboratory tests.



(a) Centrifuge model (b) Finite difference 3D model

Figure 10. Geometry of the centrifuge and numerical model [39]

The results of ground acceleration and spectra acceleration from the numerical model had good agreement with the centrifuge model. Nonetheless, as observed from Figure 11(a) and (b), the computed spectra slightly deviated from the recorded data. Furthermore, it was found that the amplification factor of a soil profile with an upper soft layer was higher than a uniform soil profile. Hence, it was significant to consider the soil layering during the assessment of seismic response. Lastly, Figure 11(c) illustrated the SSI of different soil profiles. In our case, only RG-01 (soft clay) will be reviewed. It was clearly shown that the soft soil has the highest spectra during about period of 0.2 s, which was much higher than the fixed-base structure. As highlighted by [45], the effects of SSI were pronounced for stiff structures on soil.



(a) Response spectra beneath the structure (b) Response spectra near the ground surface (c) Response spectra of various soil profiles

Figure 11. Geometry of the centrifuge and numerical model [39]

3.2 Comparison of Numerical Soil Models

Table 1. A summary comparison of the main numerical models applied to soft soils

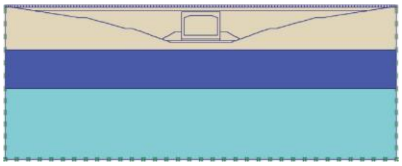
Model	Accuracy	Computational Cost	Application Scope
Modified Cam Clay (MCC)	Moderate- good for predicting settlement and pore pressure, but tends to underestimate lateral displacement	Low-relatively efficient in 2D FEA	Suitable for embankment and consolidation studies where soil behavior is primarily elastic-plastic
Elastic-Viscoplastic (EVP)	High-better at capturing time-dependent behavior, settlements, and lateral displacement	High-requires longer computing time and complex formulation	Best for staged construction, creep, and long-term consolidation of soft clays
Hardening Soil Model (HSM)	High-accurate in predicting 3D load-displacement behavior	Moderate-higher than MCC but less than EVP	Well-suited for foundations (e.g., pile rafts) where non-linear elasticity and plasticity are important
FLA3D (Finite Difference 3D)	High-captures seismic response, soil-structure interaction, and non-linear soil behavior	High-computationally demanding, requires advanced calibration	Applied to seismic loading, soil-structure interaction, and complex 3D soil response analyses

3.3 Tunnel on Soft Soil Using 2D Numerical Model

In the case study conducted by [46], a 2D numerical model was developed using PlaxisV8.5 software to simulate the Yongjiang immersed tunnel in Mainland China, which was the first tunnel constructed on very soft clay. The slope stability and settlements of the soft soils during construction and long-term were predicted using the model. The case study was conducted because differential settlement had affected various immersed tunnels on soft ground and caused other problems in the past [47-49]. According to field data, the maximum joint settlement was 86 mm after 16 years of service [50]. In the 2D model, the soil layers were modelled as presented in Figure 12(a) below while Figure 12(b) showed the overview of the 2D model. The entire construction stages were modelled.

Stratum	Thickness (m)	density ($g{*}cm^{-3}$)	Yang's modulus (Mpa)	Cohension (kPa)	Internal friction angle (°)
Muck layer	12	1.81	9.7	19	13
Mucky clay	11	1.72	13.5	23	16.1
Medium-sized sand	15	1.87	50	8	30

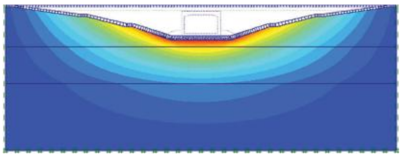
(a) Soil parameters for the subsoil



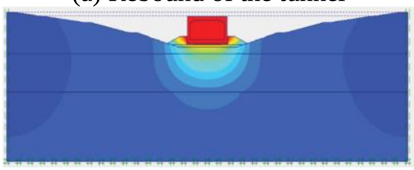
(b) Cross section of 2D numerical model

Figure 12. Soil parameters and modelling on the immersed tunnel [46]

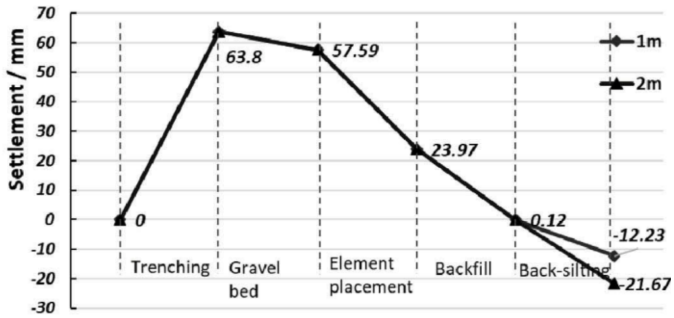
The results of settlement obtained from the 2D model were illustrated in Figure 13(a), (b) and (c) below. At the end of excavation process, the settlement rebounded to + 63.8 mm and increased progressively when gravel bed, backfill and back-silting were placed. The maximum settlement due to all the backfill materials was -57.57 mm and the absolute settlement was + 6.32 mm. The vast difference between the field and numerical result was principally influenced by the reduction of bed stiffness. Siltation and element immersion were suspected to be the main factors of bed stiffness reduction [46]. A 2D FEA model based on plane strain analysis was developed using Plaxis program to simulate the settlements and equivalent soil springs of the immersed road tunnel in Busan-Geoje, South Korea [51]. The immersed tunnel was founded on soft marine clay and required suitable soil improvements. For instance, cement deep mixing (CDM) [52] and sand compaction piles (SCP) [53] as shown in Figure 14(a) and (b) respectively. The soil parameters were obtained from laboratory tests and ground investigations [54]. In the FEA model, the material properties for the finite elements were altered to simulate the CDM and SCP. Strength reduction analysis [55] were conducted to compute the safety factors at different construction stages.



(a) Rebound of the tunnel

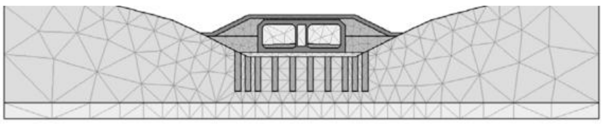


(b) Settlement of the tunnel

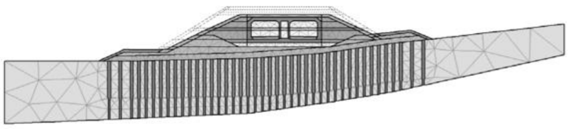


(c) Vertical displacement of the tunnel

Figure 13. Settlements of the immersed tunnel obtained from numerical model [46]



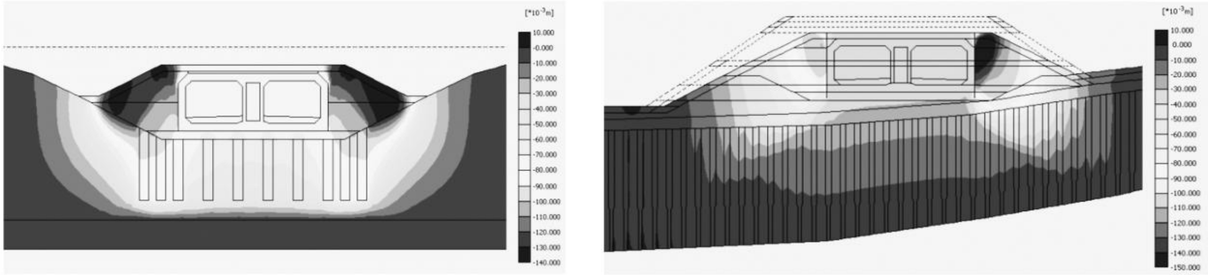
(a) FEA model of trench section with CDM



(b) FEA model of embankment section with SCP

Figure 14. Cross section of the immersed road tunnel [51]

The final settlement simulated by the model was 9 cm in the trench section and 5 cm in the embankment section. It was found that the settlement values were reasonable according to sensitivity studies and many more. The factor of safety (FOS) of the slope of the trench was 2.1. The FOS of the embankment was 1.9 at the full consolidation stage. The slope stability of the tunnel section under wave impact were studied by [56, 57]. It was concluded that the 2D FEA model could predict the settlement and slope stability of improved soft soil excellently [51].



(a) Settlement at trench section (b) Settlement at embankment section

Figure 15. Settlements of the immersed road tunnel from FEA model [51]

3.4 Soil Investigation and Parameter Calibration

Accurate numerical modelling of soft soils depends strongly on the reliability of the soil parameters used. These parameters must be obtained from comprehensive soil investigation programs, which include both field testing and laboratory testing, followed by calibration to reflect in-situ conditions Cone Penetration Test (CPT), Standard Penetration Test (SPT), Pressuremeter Test (PMT), and Vane Shear Test (VST) are widely used. CPT provides continuous soil resistance profiles that are useful for estimating undrained shear strength and stiffness. SPT offers correlations with strength and density, while PMT measures soil deformability. For very soft soils, the vane shear test is effective in determining undrained shear strength. Oedometer tests provide compressibility and consolidation parameters (Cc, Cs, pc’). Triaxial test (UU, CU, CD) supply shear strength parameters (c, φ, *su*) and stiffness. Advanced tests such as resonant column and bender element test are used to obtain small-strain shear modulus (Gmax) and dynamic parameters, which are essential in seismic modelling. Parameter Selection for Models

i. Modified Cam Clay (MCC): Requires compression index, swelling index, preconsolidation pressure, and critical state parameter M, obtained from oedometer and triaxial tests.
ii. Elastic-Viscoplastic (EVP): In addition to MCC parameters, time-dependent parameters such as viscosity and creep rate must be derived from long*erm oedometer and triaxial creep tests.
iii. Hardening Soil Model (HSM): Needs stiffness moduli (E50, Eoed, Eur), reference pressure, and failure ratio, generally obtained from triaxial and odometer tests.
iv. FLAC3D: Typically requires basic shear strength (c, φ), elastic modulus, Poisson’s ratio, and damping factors. These are derived from both laboratory and field test, with dynamic properties calibrated using resonant column tests.

Laboratory test values often need adjustment before use in models due to sample disturbance, strain-rate effects, and scale differences. Calibration involves applying correction factors (e.g., Bjerrums’s correction for undrained shear strength), conduction trial simulations, and matching model outputs against field monitoring data (e.g., settlements plates, piezometers, inclinometers). This iterative process ensures that the chosen parameters represent actual in-situ conditions. The accuracy of numerical simulations depends not only on the constitutive model (MCC, EVP, HSM, FLAC3D) but also on the quality of soil investigation and calibration. MCC models are highly sensitive to critical state parameters; EVP models require creep parameters that are difficult to obtain; HSM models rely on empirical stiffness correlations; and FLAC3D models depend on reliable dynamic properties for seismic applications. Therefore, comprehensive soil investigation combined with careful calibration is essential to reduce discrepancies between numerical predictions and observed field performance.

4. CONCLUSIONS

The numerical simulations of the behaviour of soft soil subjected to embankment loads, foundations and immersed tunnels have been presented. The numerical predictions were conducted using 2D and 3D FEA model using different software. The mechanical behaviour of soft soils was described using MCC model, EVP model and HSM. Comparisons of the vertical settlements, lateral deformations, excess pore water pressure, slope stability and seismic response of soft soils between the field data and numerical simulations were conducted. The numerical models were able to capture the pattern of the soils’ response. However, under-estimation and over-estimation still occurred. Based on a comprehensive review of studies conducted over the past few decades, the following suggestions are provided to improve the reliability and accuracy of numerical models.

a. Extensive soil investigation and laboratory test must be carried out to obtain the accurate soil parameters for the numerical model.
b. The parameters obtained from the testing must be calibrated to represent the characteristics of the soil in the model by using appropriate correction factors.
c. SSI is an important aspect to be considered in the numerical study of seismic response of rigid structures on soft soil.

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