A Sustainable Approach to Marine Reclamations Using Local Dredged Marine Soils and Wastes: Soft Soil Improvement, Physical Modelling Study, and Settlement Prediction-control

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ABSTRACT

Housing is currently one of the burning social issues in Hong Kong. There is an urgent need for providing large areas of suitable lands for residential houses and other infrastructures. In 2018, the Hong Kong Government proposed a major reclamation project in Hong Kong waters, i.e., "Lantau Tomorrow" vision, the main concerns of which are the short supply of fill materials, long construction time, and high cost. To tackle these concerns, the authors have proposed to use local dredged Hong Kong Marine Deposits (HKMD) and construction wastes to fill a reclaimed area on the seabed in a major Research Impact Fund project in 2019 with HK$15M funding. The use of local HKMD and construction wastes can significantly save the costs for fill material and shorten the construction time. In this paper, successful reclamation projects using soft soils will be briefly reviewed. The state-of-the-art research findings in PolyU, including the results from two ongoing physical model tests, turning construction wastes into the competent filling materials, and a well-verified new simplified Hypothesis B method for predicting soft soil settlements will be presented. Lastly, the methodologies for controlling the post-construction settlement will be discussed.

Keywords: Marine Reclamations, Soft Soil Improvement, Settlement Prediction-Control

1 Introduction

Hong Kong ranks among the top in the world in terms of population density and the least affordable housing market. The average waiting time to get the government-subsidized houses has been increased to 6 years, equaling the highest record in history. There is an urgent need for providing a large area of suitable lands for residential housing and other infrastructures. In the Chief Executive's 2018 Policy Address, "Lantau Tomorrow" reclamation in Hong Kong waters was proposed with a total 1700 hectares of land for 700,000 to 1,100,000 people. The detailed planning and engineering study for the artificial islands of about 1000 hectares around Kau Yi Chau is in progress. One of the biggest challenges for "Lantau Tomorrow" reclamation is a short supply of fill materials. The construction time and cost are also a concern.

Marine reclamations have a long history in Hong Kong and the technologies used for marine reclamations vary widely. As part of the Hong Kong-Zhuhai-Macau Link Project, two artificial islands were reclaimed using sandfill and vertical drains with sandfill surcharge surrounded by 120 steel pipe piles (22 m in diameter and 50.5 m in length). In the reclamation of Hong Kong Airport Runway 1 and Runway 2, the reclamation was done by dredging and removing all Hong Kong Marine Deposits (HKMD) which caused marine environmental problems. For the new Third Runway, all HKMD have been kept in situ and surrounded by seawalls constructed on Deep Cement Mixed (DCM) soil foundations. Imported sandfills (or crushed stones) have been used to fill the reclaimed area and vertical drains with preloading have been used to improve the HKMD. However, there is a severe shortage of sand for marine reclamations in Hong Kong. The cost of crushed stones is high and takes time.

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Therefore, there is an imperative need to identify an alternative reclamation method, that is economical, efficient, and sustainable, for Hong Kong.

In view of the limited and expensive sand resources, the dredging and filling method using local soft soils has become the most prevalent method for marine reclamation projects in many other coastal cities. The dredged soft soils are from the basin of harbor, lake or river, the sea channel. These soft soils have normally high water content, high compressibility, low permeability, and low strength. In Hong Kong, maintenance dredging of HKMD for harbor and navigation channels is periodically conducted. The dredged HKMD are naturally dewatered and transported to the Mainland for disposal. In our approach, these HKMD can be used for marine reclamation in Hong Kong. In addition, HKMD in seabed in Hong Kong waters, where marine ecology is not much affected, can be dredged and used for reclamations in Hong Kong.

Among many existing soft soil ground improvement techniques, prefabricated vertical drains (PVDs) with vacuum/surcharge preloading have gained their popularity due to their high efficiency, high safety, low contamination and low cost. Yan and Chu (2005) reported a case study of using the combined vacuum and fill surcharge preloading method to improve the recently dredged clay slurry with a thickness of 16 m for a storage yard at Tianjin Port. Wang et al. (2016) presented an improved vacuum preloading method with an air-pressurizing system to accelerate the consolidation of Wenzhou clay slurry through a field pilot test. Another field test in Shenzhen that investigated the feasibility of under water vacuum preloading method in treating soft clay is reported by Kwong et al. (2008). Similar studies were also conducted in other countries, for example, in Singapore (Chu et al. 2000), Australia (Indraratna et al. 2004), Thailand (Artidteang et al. 2011), and Japan (Chai et al. 2012). Though the performance of vacuum preloading with PVDs has been widely validated so far, the limitations of this method can be seen from: (1) severe bending and clogging of PVDs, reducing dewatering efficiency; (2) obvious deterioration of vacuum pressure along with the depth, resulting in insufficient consolidation at deeper positions; and (3) requirement of a working platform for the installation of PVDs and the application of the preloading, which extends the construction duration.

This paper aims to introduce a novel reclamation method for treating local HKMD slurry in a feasible, efficient and economical way. The detailed procedures of this novel method will be presented. In addition, two ongoing physical models, one is a large cylinder model and the other is a large plane strain model, will be described in detail, followed by the demonstration of initial test results and data interpretation. Turning wastes into competent filling and/or strengthening materials for marine reclamation in Hong Kong will be presented briefly. Besides, a well-verified new simplified Hypothesis B method will be introduced as well. Lastly, the methodology for controlling the post-construction settlement will also be presented. All the works are from our recent studies and will contribute to research development, the design and construction of marine reclamations, and make a positive impact on Hong Kong society.

2 New Reclamation Method Using Dredged Marine Soils

Figure 1: Tian Kun dredging and blow-filling ship

To solve the problem of the short supply of lands, the authors have advocated using local dredged marine soils or construction wastes with a combined ground improvement method to achieve a super-fast, large-area and low-cost marine reclamation in Hong Kong or even Guangdong-Hong Kong-Macau Greater Bay Area. The super-fast construction is contributed in four ways: (i) fast dredging and blow-filling local marine deposits, (ii) use of local general soil fill or even construction wastes instead of using imported sand, (iii) fast consolidation by using horizontal and vertical drains with vacuum preloading rather than using imported sand as surcharge; and (iv) fast installation of steel (or FRP/FRP concrete) pipe pile walls rather than the slow method using concrete blocks. Chinese Mainland has had the newest dredging blow-filling ship Tian Kun (Figure 1) which has a blow filling speed of 6000 m^3/hour, operating 24 hours per day. Take the 1700 hectares of "Lantau Tomorrow" reclamation as an example. If we assume that the average water depth is 15 m and the final filled land is +6 m above sea level, the total volume of the fills needed, considering added fill for 2 m settlement compensation, will be 3.91×10^8 m^3. If 20 Tian Kun ships are employed for the reclamation, the time required will be 1.39 years. If 20 conventional 4000-ton ships are utilized to move sand from a site in Mainland, say, the time required will be 133.9 years.

"Large-area" and "low-cost" of the new method are contributed in two ways: (i) blow filling a large area, say, a few square kilometers is easy and (ii) since no imported sand is needed, the economical benefit of the new method of using local free marine soils or general soil fills/construction wastes for reclamation is obvious. The estimated cost saving for imported sand only will be HK$60 billion dollars for the 1700 hectares of "Lantau Tomorrow" reclamation.

3 Combined Ground Improvement Method for HKMD Slurry

The local dredged marine soils are mainly HKMD. In order to facilitate the blow-filling process, dredged HKMD will be mixed with water to reach a certain slurry state. Due to the high water content, high compressibility, and extremely low bearing capacity of blow-filled HKMD slurry, ground improvement techniques have to be integrated with the blow-filling process to speed up the consolidation and enhance the shear strength.

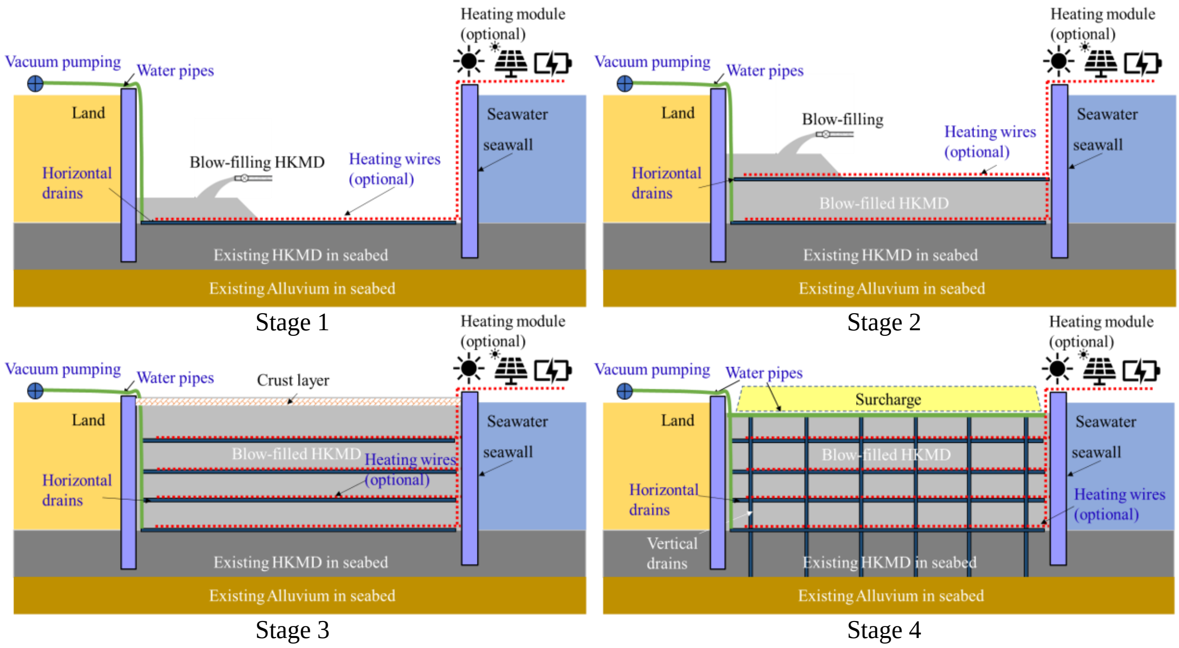


Figure 2: Illustration of a combined ground improvement method for HKMD slurry in stages

A ground improvement method combining horizontal and vertical drains with vacuum and/or surcharge preloading is recommended by the authors to improve the HKMD slurry in stages, as illustrated in Figure 2.

In Stage 1, a layer of prefabricated horizontal drains (PHDs) is placed on the existing seabed before blow-filling HKMD slurry. Vacuum pressure is applied to the PHDs after significant self-weight consolidation of the HKMD slurry. In Stage 2, the second layer of PHDs is placed on the blow-filled HKMD slurry followed by filling the second layer of HKMD slurry. The newly filled HKMD slurry is subjected to a self-weight consolidation followed by a vacuum preloading, while the HKMD slurry filled in Stage 1 is subjected to both the vacuum preloading and the vertical surcharge from the self-weight of the newly filled HKMD slurry. In Stage 3, a multi-layer HKMD ground is formed by blow-filling and improved by PHDs with vacuum preloading technique. Once a crust layer with sufficient strength is formed on the surface of the HKMD ground, PVDs are installed in Stage 4 to further improve the HKMD ground with the help of vacuum and/or surcharge preloading. The surcharge preloading can be applied by filling construction wastes. Optional heating techniques can be used to tackle marine soils with very low permeability or significant viscosity.

For a PVDs-installation crawler with a working load of 80 kPa and a working area of around 6 m^2, a crust layer possessing an average undrained shear strength of 35 kPa and a thickness of 0.5~1 m is most likely to meet the requirement of FOS>3. For a light crawler with a working loading of 30 kPa and a working area of around 6 m^2, a crust layer with an average undrained shear strength of 15 kPa and a thickness of 0.5~1 m is most likely to satisfy the requirement of FOS>3. Crust layers can be formed by sun-dry or mixing the surface layer with cementitious binders (presented in Section 5). It should be noted that the abovementioned ground improvement methods are proposed with the purpose of fast reclamation. Post-construction settlements shall be controlled by other measures, which are discussed in Section 7.

4 Two Types of Ongoing Physical Model Tests

4.1 Testing Material

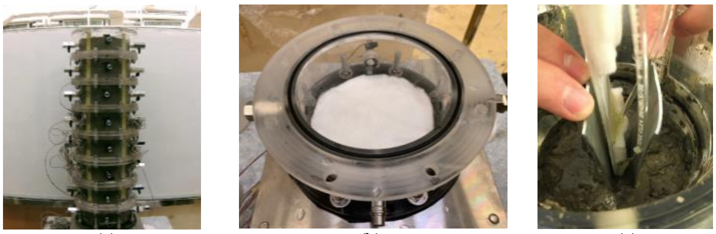
The soil used in the physical model tests was HKMD, which is mud-like soil dredged from a construction site in Tuen Mun, Hong Kong. In accordance with the British standard BS 1377:2016, basic properties were determined and listed in Table 1. Particle size distribution was determined by the wet sieving method and hydrometer method and showed that the soil mainly consists of clay (36%), silt (33%) and sand (31%). It should be noted that the initial water content was increased to 220% in the following physical model tests.

4.2 Cylindrical Physical Model Tests

Table 1: Basic properties of the HKMD

Property	Value
Natural water content (%)	86.31
Liquid limit (%)	63.88
Plastic limit (%)	28.23
Plasticity index	25.22
Specific gravity	2.63
Void ratio	2.27
Salinity (%)	3.30

Figure 3(a) presents the photos of the cylindrical physical model, which consists of several short Perspex tubes called sub-columns. Each sub-column is 100 mm in height and 170 mm in internal diameter. Eight sub-columns can be assembled together to have an 800 mm initial height for each test. There are three holes opened at the middle height of each sub-column. Two of them serve as sampling ports with valves and the third one is for pore pressure monitoring. During the test, soil samples were extracted from sampling ports at different heights and different time points so that the water content profile during the test can be captured. Meanwhile, pore pressure was monitored via the pressure transducers installed on the sidewall. Settlement of the soil surface was observed by a digital camera due to the transparent feature of the Perspex material. There were two outlet channels in the bottom plate. One was for measuring the pressure at the bottom, and the other one connected the vacuum system to apply vacuum loading through the bottom. Two pieces of geotextiles were placed on the bottom plate and a porous stone was put



(a) (b) (c)

Figure 3: Photos of a cylindrical physical model: (a) the segmented settling columns, (b) geotextile laid on the bottom as PHD, and (c) PVD installation

between the geotextiles to form a sandwich permeable layer as a horizontal drain (Figure 3(b)). The sandwich layer was used instead of the prefabricated drain (PD) so that the vacuum loading can be uniformly applied on the whole cross-section of soils. In this model test, only one horizontal drainage was placed at the bottom. In fact, multiple PHDs can be installed at different soil depths, for example, in our large physical model test and the field trial. Two model tests were designed and completed: namely, Test 1 and Test 2. Test 1 was a control test of vacuum preloading with PHD, and multi-staged vacuum pressure from 20 kPa to 80 kPa was applied in order to reduce clogging effect by lower initial vacuum pressure. Both tests started with 4 days of self-weight consolidation. The total period of applying vacuum pressure in Test 1 was 19 days. Test 2 was the test combining PVD and PHD, with 14 days for vacuum application on PHD only and another 14 days for vacuum pressure on both PHD and PVD. The installation of PVD is shown in Figure 3(c). The initial height of soil slurry was 800mm.

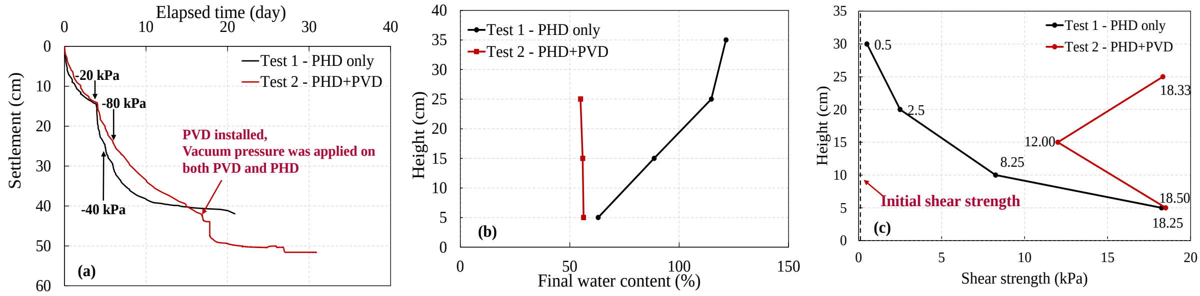
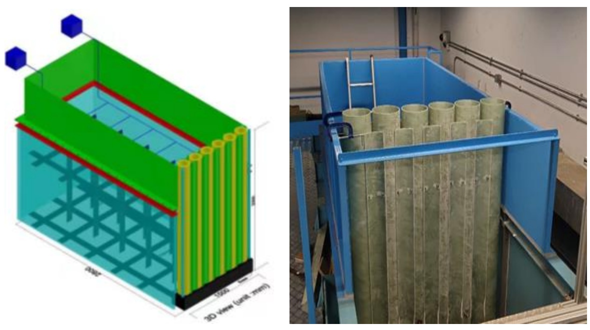


Figure 4: Plots of cylindrical physical model tests: (a) settlement versus elapsed time, (b) final water content versus soil height, and (c) shear strength versus soil height

As can be seen in Figure 4(a), the HKMD slurry settled significantly in the first 4 days due to self-weight consolidation and followed by a sudden drop induced by -20 kPa vacuum pressure at the bottom. -40 kPa and -80 kPa were applied after 5 and 6 days, respectively. It can be observed that in Test 1 the settlement tended to level off after -80 kPa was held for 9 days. And the settlement in Test 2 generally kept increasing with a decreasing rate. While, for Test 2, an obvious sudden drop on settlement curve was observed immediately after the PVD was inserted with -20 kPa applied for 1 day. Another large sudden increase occurred when vacuum pressure was further increased to -80 kPa on both PHD and PVD. The settlement, then, started to stabilize quite quickly after 3 days with no obvious further increase. The final settlements for Test 1 and Test 2 were 0.46 m and 0.53 m, respectively. The final water content along soil height at the end of each test was shown in Figure 4(b). As expected, the final water contents at different heights in Test 2 were lower than those in Test 1, with an even distribution for Test 2 and decreasing trend with decreasing height for Test 1. Figure 4(c) shows the shear strength along the soil height measured by a hand-held vane shear device. The shear strength in Test 1 increased with decreasing soil height and kept more or less stable along with soil height in Test 2, which corresponds to the change of water content in Figure 4(b). After the combined PHD and PVD with vacuum preloading, the shear strength generally increased from almost zero to around 18 kPa at the top and bottom boundary and to 12 kPa at the middle height of soil. The test results imply that the dewatering capacity can be largely improved using the vacuum preloading method with both PHD and PVD, compared to the case with PHD only.



(a) (b)

Figure 5: (a) Schematic drawing and (b) photo of plane strain physical model

4.3 Plane Strain Physical Model Tests

The plane strain physical model is aimed to simulate the real case of reclamation projects, including the dredging process, self-consolidation process, fast consolidation process by vacuum preloading with prefabricated band drains. This large model test was built in a water tank with the dimension of 1.5 m in width, 2.5 m in length, and 2.3 m in height. The details of the model are shown in Figure 5. The filled soil was surrounded by steel plate wall and FRP pipe wall, which consists of 6 FRP pipes with a length of 2.3 m, a diameter of 0.21 m, and a thickness of 0.005 m. The vacuum pressure was applied through PVDs and PHDs.

The test procedure is as follows:

Step 1: Place PHD grid first at the bottom, then pump HKMD slurry into the tank, and apply vacuum pressure on PHD grid layer.

Step 2: After consolidation, place another PHD grid layer at the soil surface in Step 1, then pump in the second layer of slurry, then apply vacuum pressure on both PHD grid layers.

Step 3: After consolidation, place the third layer of PHD grid layer at the soil surface in Step 2, then pump in the third layer of slurry, then apply vacuum pressure on three PHD grid layers together.

Step 4: After consolidation, install PVDs at predetermined positions and cover the soil with a geomembrane, then apply vacuum pressure on all PHD grid layers and PVDs.

The development of the settlement of the bottom layer of soil in Step 1 with time is shown in Figure 6(a). It can be observed that a sharp increase in settlement occurred after the vacuum preloading was applied. The height of the bottom layer of HKMD was decreased from 1 m to 0.634 m. Figure 6(b) presents a general decreasing trend of water content with the decrease of the soil height at all time points and the average water content in the physical model decreased from 181% to 101% during the period of applying vacuum pressure on the PHD layer at the bottom. Special emphasis should be paid to the fact that the difference of water content between the soil at the bottom and the surface increased with time, which indicates that a better dewatering effect can be achieved for the soil with a closer distance to the PHDs.

After the second layer of soil was pump-filled on top of the bottom layer, vacuum pressure was applied on both PHD layers. The development of settlement with time is shown in Figure 7(a). A sudden increase in the settlement of the bottom layer of soil can be observed because the second layer of soil served as a surcharge on the bottom layer of soil and the second layer of PHD with vacuum pressure also accelerated the dewatering of the bottom layer of soil. Comparisons of water content and shear strength are shown in Figure 7(b) and Figure 7(c), respectively. A general decreasing trend of water content and a general increasing trend of shear strength along with decreasing soil height can be observed. An obvious inflection point at a height of 47cm (the height of the second layer of PHDs) can be observed in both figures, proving that a better improvement performance can be obtained at a closer distance with the PHDs. It can be seen that the average water content of two layers of soil was decreased from 155% to 76% with the final water content of 60% at the bottom and the final shear strength of 27 kPa at the bottom. In the following test schedule, the third layer of soil will be placed on

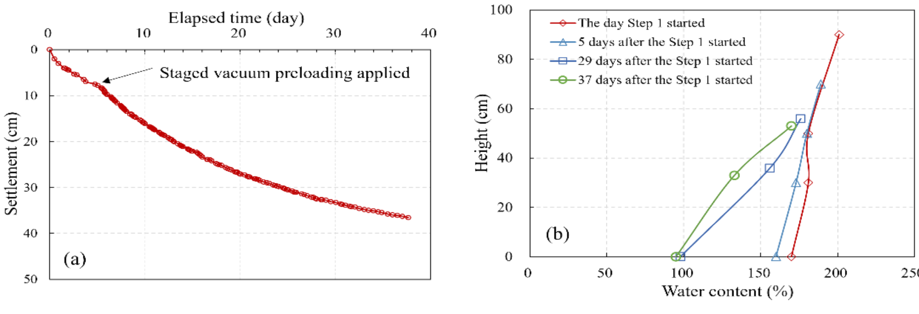


Figure 6: (a) Settlement versus time under staged vacuum preloading and (b) water content along soil height at different times after vacuum pressure applied

top and treated by PHDs. After the stabilization of settlement of soil due to PHDs, PVDs will then be inserted and applied with vacuum pressure.

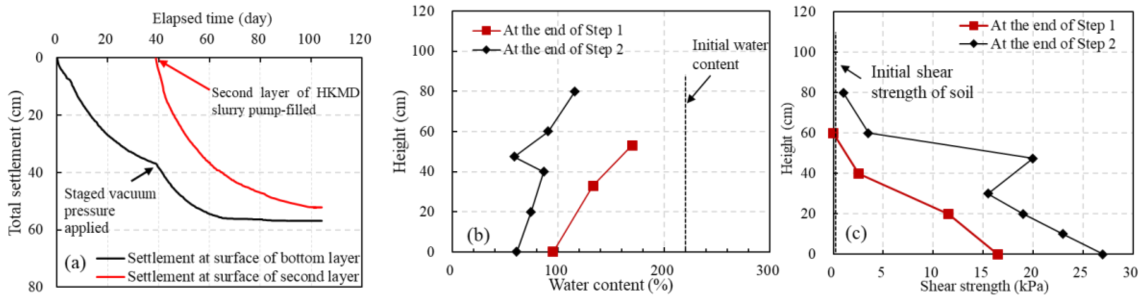


Figure 7: Plots of (a) the settlement versus time of two layers of soil in Step 2, (b) the water contents along soil height after Step 1 and Step 2, and (c) the shear strength along soil height after Step 1 and Step 2

5 Use of Waste into Filling Materials for Marine Reclamation

HK now produces approximately 1200 tonnes/day (2000 tonnes in 2030) of dewatered sewage sludge. The sludge will then be converted into Incinerated Sewage Sludge Ash (ISSA) which has to be disposed of at landfills, which will be fully filled within 5 years. ISSA is a by-product from the burning dewatered sewage sludge in furnaces in Hong Kong. The use of a combination of ISSA with traditional binders including cement or lime on soil improvement has been studied and showed that ISSA has great potential to be utilized in geotechnical application. Therefore, a "win-win" strategy of turning large volumes of dredged HKMD into fill materials using ISSA and certain alkali activators is expected to be of high economic and social benefits. This sustainable fill material can be used to form a competent crust layer quickly on top of HKMD slurry treated by PHD vacuum preloading for the easy access of personnel and light machinery to insert PVD.

The authors have adopted lime-activated ISSA as a binder, which is mixed into HKMD slurry to decrease the water content and increase the strength. In order to further increase the strength, another industrial by-product, i.e., ground-granulated blast-furnace slag (GGBS) is also added into binder material with different mixing ratios to ISSA. So far, two series of unconfined compression (UC) tests have been conducted: (1) dry mass of binder (ISSA only) to dry mass of HKMD is equal to 30%, and the activator ratio (AR) (a dry mass ratio of lime to binder) is 20%, 30%, and 40%, respectively; (2) dry mass of binder (ISSA and GGBS) to dry mass of HKMD is equal to 30%, the mass ratio between ISSA to GGBS is 0, 1, 2, 3, and 5 respectively, and the addition of lime is kept at 30% in dry mass to the binder. The initial water content of all specimens is 200%, defined as the ratio of water mass to dry mass of soil. All the mixed specimens were cured under an environmental chamber with humidity of 90% and a temperature of 24 ℃. The specimen dimension is 50mm in diameter and 100mm in height.

Figure 8(a) shows the unconfined compressive strength (UCS) of ISSA-based specimens, i.e., the first series of tests. The UCS with an AR of 20% showed no obvious change with the increase of curing time, which implies that the low content of lime is incapable of providing a sufficient alkali environment for activating ISSA. When AR was increased to 30% and 40%,

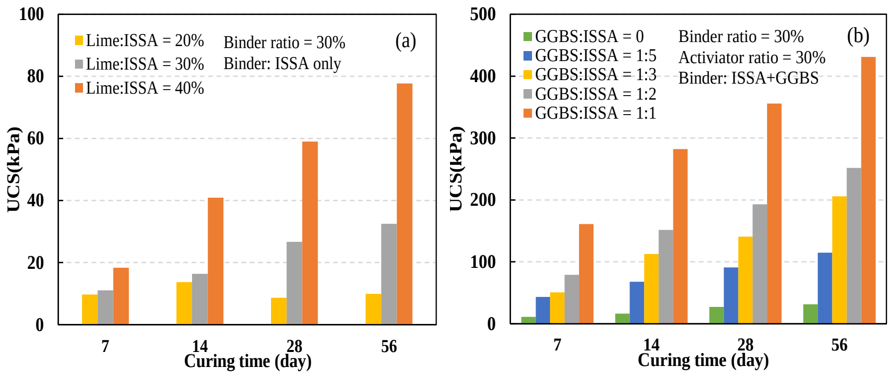


Figure 8: UCS of (a) ISSA-based and (b) ISSA+GGBS-based specimens at 7, 14, 28, 56 days

the effect of curing time became more significant. The hydration of lime will decrease the water content by a large extent and provide hydration products to bond soil particles. This explains the reason why only 10% change of AR would lead to a double in UCS when AR changed from 30% to 40%. As discussed in Section 3, with AR of 30%, the treated specimen achieved a strength level of soft clay, which generally fulfills the strength requirement for a light crawler to insert PVD with a FOS>3 (UCS>30 kPa). While with 40%, the specimen achieved a level of medium stiff clay, which is competent to sustain a crawler with working load of 80 kPa to meet the requirement of FOS>3 (UCS>75 kPa). In order to further increase the strength, GGBS can be added by different ratios to ISSA as binder. As can be seen in Figure 8(b), the UCS keeps increasing with the addition of GGBS and the increasing curing time. The obvious increase in UCS implies that there would be a great potential of alkali-activated ISSA+GGBS treated HKMD to be used in road pavement applications, provided that further studies can be conducted on the long-term reliability influenced by environmental change and degradation behavior under cyclic loading due to moving vehicle. As the following work, a mechanical-chemical combined method, which incorporates binder stabilization and vacuum preloading with PHD at the same time, will be further implemented in PolyU to propose an efficient and sustainable way in the marine reclamation project.

6 New Simplified Hypothesis B Method

It is well known that creep compression shall be considered in both "primary" consolidation and "secondary" consolidation periods. For an accurate settlement calculation, a rigorous Hypothesis B method coupling the dissipation of excess pore water pressure and viscous deformation of soil skeleton has been used based on a proper Elastic Visco-Plastic (EVP) constitutive model (Yin and Graham 1996 and 1999). However, this method needs to solve a set of nonlinear partial differential equations which is not easy to be used by engineers. Yin and his coworkers have done many works to simplify the rigorous Hypothesis B method by partially de-couple the dissipation of excess pore water pressure and have proposed a simplified method for multi-layered soils with/without drains under complicated loadings (Yin and Feng 2017; Feng et al. 2020). Recently, a general simplified Hypothesis B method (Yin et al. 2022) was proposed and verified.

In many cases, the consolidation of soils is very close to or can be approximated as a one-dimensional (1D) problem, that is, strain and water flow occur in vertical direction only as assumed in Terzaghi's 1D consolidation theory. In this case, a simple method for consolidation analyses of clayey soils exhibiting viscous behaviour is available to use. The simple method for a single-layer case is presented below.

$$S = S_{primary} + S_{creep} = \\ = \begin{cases} US_{pf} + \alpha U^{\beta} S_{creep,f} & for \quad t_0 \le t \le t_{EOP,field} \\ US_{pf} + [\alpha U^{\beta} S_{creep,f} + (1-\alpha U^{\beta}) S_{creep,d}] & for \quad t \ge t_{EOP,field} \end{cases} \quad \textbf{(1)}$$

Here S is total settlement. $S_{primary} = US_{pf}$ is "primary" consolidation settlement. U is average degree of consolidation. S_{pf} is settlement at the end of the "primary" consolidation. $S_{creep} = \alpha U^{\beta} S_{creep,f}$ or $S_{creep} = [\alpha U^{\beta} S_{creep,f} + (1-\alpha U^{\beta}) S_{creep,d}]$ is viscous (creep) settlement. $S_{creep,f}$ is creep settlement under the final total vertical effective stress assuming the excess porewater pressure zero. $S_{creep,d}$ is creep settlement delayed to occur by $t_{EOP,field}$, that is, $t \ge t_{EOP,field}$ for $S_{creep,d}$. t_0 is a creep parameter and is equal to 1 day since all points in the compression curve in Figure 9 have the load duration of 1 day in a conventional staged oedometer test. $t_{EOP,field}$ is the time at the End-of-Primary consolidation for the field condition, calculated using $U = 98\%$. α and β are two constants with $\alpha = 0.8$ and $\beta = 0.3$

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recommended for all clayey soils. Small ranges of $0.75 \le \alpha \le 0.85$ and $0.2 \le \beta \le 0.4$ are possible and can be verified by using the rigorous Hypothesis B method.

In the above equation, S_{pf} is the final primary consolidation settlement and is caused by an applied load. S_{pf} depends on the relative magnitudes of the initial vertical effective stress acting on the soil and the effective preconsolidation pressure, and can be estimated as follows, based on oedometer test condition or one-dimensional (1D) strain condition:

For $\sigma'_{v0} = \sigma'_p < \sigma'_{v0} + \Delta\sigma'_v$: $S_{pf} = H_s(\frac{C_c}{1+e_0}\log\frac{\sigma'_{v0}+\Delta\sigma'_v}{\sigma'_{v0}})$ (2a)

For $\sigma'_{v0} < \sigma'_p < \sigma'_{v0} + \Delta\sigma'_v$: $S_{pf} = H_s(\frac{C_r}{1+e_0}\log\frac{\sigma'_p}{\sigma'_{v0}} + \frac{C_c}{1+e_0}\log\frac{\sigma'_{v0}+\Delta\sigma'_v}{\sigma'_p})$ (2b)

For $\sigma'_{v0} < \sigma'_{v0} + \Delta\sigma'_v < \sigma'_p$: $S_{pf} = H_s(\frac{C_r}{1+e_0}\log\frac{\sigma'_{v0}+\Delta\sigma'_v}{\sigma'_{v0}})$ (2c)

Here σ'_{v0} is initial vertical effective stress in the soil layer. e_0 is initial void ratio in the soil layer. σ'_p is effective preconsolidation pressure, which is the maximum vertical effective stress that has acted on the soil layer in the past and can be determined from laboratory oedometer tests. $\Delta\sigma'_v$ is the change in vertical effective stress due to the fill and future imposed load on the soil layer. H_s is thickness of the soil layer to be considered. $C_c/(1+e_0) = CR$ is compression ratio, equal to the slope of the virgin compression portion of the $\varepsilon_v - \log\sigma'_v$ plot as shown in Figure 9 and C_c is compression index which can be estimated from laboratory oedometer tests. $C_r/(1+e_0) = RR$ is recompression ratio, equal to the average slope of the recompression portion of the $\varepsilon_v - \log\sigma'_v$ plotted in Figure 9 and C_r is recompression index which can be estimated from laboratory oedometer tests, and ε_v is vertical strain caused by $\Delta\sigma'_v$. Note that the curve of $\varepsilon_v - \log\sigma'_v$ plotted in Figure 9 is normally measured from an oedometer test under multiple staged load increments with one day duration under each load increment.

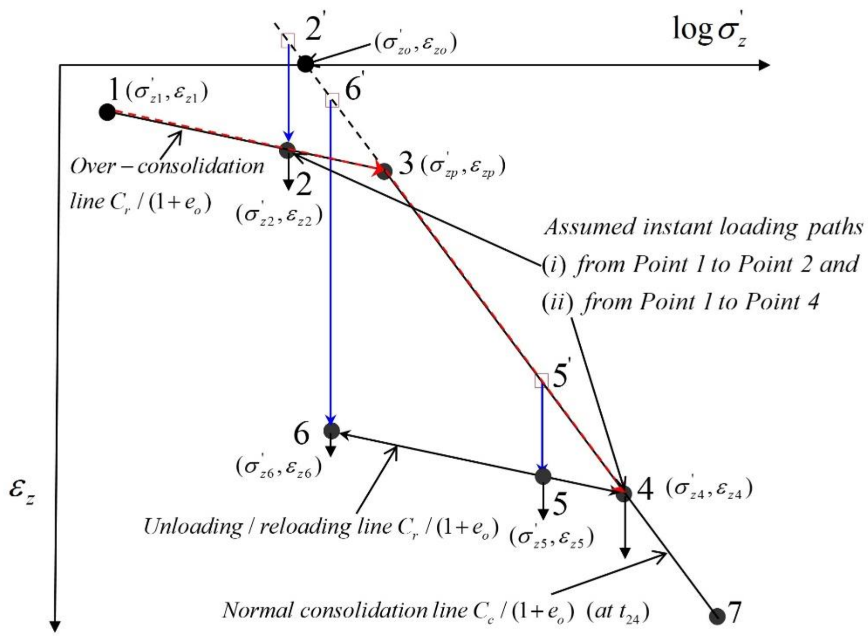

Figure 9: Relationship of strain and log (effective stress) with different consolidation states (after Yin *et al.* 2022)

Referring to Figure 9 and considering one soil layer with thickness H_s, Point 1 is considered to be the original starting condition with initial stress and strain $(\sigma^{'}_{v1}, \varepsilon_{v1})$ in the field. Here $\sigma^{'}_{v1}$ has the same meaning as $\sigma^{'}_{v0}$ in the above equations. Due to gravity, the initial vertical effective stress increases with depth. In this case, the initial stress $\sigma^{'}_{v1}$ may be calculated at the mid-height $H_s/2$ of this soil layer. The initial strain ε_{v1} is normally taken as zero. Point 1 to Point 3 in Figure 9 is in an Over-consolidation (OC) line with slope $C_r/(1+e_0)$. The average line slope of the expansion-recompression loop in Figure 9 has the same slope $C_r/(1+e_0)$ as that of this OC line. Point 3 is considered the pre-consolidation pressure point $(\sigma^{'}_p, \varepsilon_p)$ in the soil profile, also at the mid-depth $H_s/2$ of the layer. The preconsolidation pressure $\sigma^{'}_p$ is normally measured in an oedometer test with one-day duration for each staged load on a soil sample taken from the mid-depth. The strain ε_p can be calculated as $\varepsilon_p = [C_r/(1+e_0)]\log(\sigma^{'}_p / \sigma^{'}_{v1})$. The line from Point 3 to Point 4 in the figure is a Normal Consolidation (NC) line with slope $C_c/(1+e_0)$. This is measured from the same oedometer test with one-day duration for each staged loading. From Point 4 to Point 6 is an unloading/reloading line with same slope $C_r/(1+e_0)$ as the over-consolidated line from Point 1 to Point 3.

The above equation can also be used for the case with vertical drains. In this case, $U = 1-(1-U_v)(1-U_r)$ where U_v and U_r are average degree of consolidation for vertical and radial directions, respectively. The values of $\alpha = 0.8$ and $\beta = 0.3$ are obtained from the best-fitting the calculated settlement curve to the settlement curves computed by the fully coupled consolidation analyses and use $U = 98\%$ to calculate $t_{EOP,field}$. The U is related to a time factor, for example, $T_v = tc_v / d^2$ for 1D consolidation. The parameter d is the maximum drainage distance. If the layer with thickness H_s has top and bottom drainages, then $d = H_s/2$. Many charts and equations are available for using the value of T_v to find U. For ramp loading, Terzaghi's correction method can be applied for finding the corrected U. The c_v used in the expression for T_v is related to hydraulic conductivity k and the coefficient of volume compressibility m_v by $c_v = k/(m_v \gamma_w)$. The value of m_v is not a constant. For each stress increment $\Delta\sigma^{'}_v$, the value of m_v can be calculated from strain increment $\Delta\varepsilon_v$ using $m_v = \Delta\varepsilon_v / \Delta\sigma^{'}_v$. For example, if the load increment is from Point 1 to Point 4, the stress increment is $\Delta\sigma^{'}_v = \sigma^{'}_{v4} - \sigma^{'}_{v1}$ and the strain increment $\Delta\varepsilon_v = [C_r/(1+e_0)]\log(\sigma^{'}_p/\sigma^{'}_{v1}) + [C_c/(1+e_0)]\log(\sigma^{'}_{v4}/\sigma^{'}_p)$. In this simple method, m_v is back-calculated using $\Delta\varepsilon_v$ and $\Delta\sigma^{'}_v$ in order to obtain U.

Both $S_{creep,f}$ and $S_{creep,d}$ are calculated as follows for different final stress-strain points.

(i) If the final stress level is in the NC range (for example Point 4 in Figure 9), the final creep settlement is:

$$S_{creep,f} = \frac{C_\alpha}{1+e_0}\log(\frac{t}{t_o}) \times H_s \qquad for \quad t \geq t_o = 1\,\text{day} \tag{3}$$

where C_α is the conventional "coefficient of secondary compression" defined as $C_\alpha = -\Delta e / \Delta \log \sigma^{'}_v$ from an oedometer test on a soil specimen in an NC state. The time *t* is the duration of the applied load of interest, in which the corresponding total settlement is to be calculated. Note that t_o in the equation is soil-related parameter and has the value of one day because the compression relation in Figure 9 has a one-day duration for each load increment. The delayed creep settlement is:

$$S_{creep,d} = \frac{C_\alpha}{1+e_0}\log(\frac{t}{t_{EOP,field}})\times H_s \qquad for \quad t \geq t_{EOP,field} \tag{4}$$

(ii) If the final loading stress level is in an OC range (for example, Points 2, 5, 6), the final creep settlement is:

$$S_{creep,f} = \frac{C_\alpha}{1+e_0}\log(\frac{t+t_{eOC}}{t_o+t_{eOC}})\times H_s \qquad for \quad t \geq t_o = 1\text{ day} \tag{5}$$

where t_{eOC} is the "equivalent time" for the final point in an OC state. The t_{eOC} is calculated from:

$$t_{eOC} = t_o \times 10^{\left[(\varepsilon_{OC}-\varepsilon_p)\frac{V}{C_\alpha}\right]}(\frac{\sigma'_{OC}}{\sigma'_p})^{-\frac{C_c}{C_\alpha}} - t_o \tag{6}$$

where $V=(1+e_0)$ is specific volume. For example, for Point 2 with stress and strain point in Figure 9 $(\sigma'_{v2}, \varepsilon_{v2})$, the time $t_{eOC}=t_{e2}$ can be calculated using the above equation by letting $\varepsilon_{OC}=\varepsilon_{v2}, \sigma'_{OC}=\sigma'_{v2}$, that is, $t_{e2} = t_o \times 10^{\left[(\varepsilon_{v2}-\varepsilon_p)\frac{V}{C_\alpha}\right]}(\frac{\sigma'_{v2}}{\sigma'_p})^{-\frac{C_c}{C_\alpha}} - t_o$. If the NC line in Figure 9 is extended to Point $2'$ and above, Point $2'$ has the same stress σ'_{v2}. The physical meaning of $t_{eOC}=t_{e2}$ is that the time for creep compression from Point $2'$ to Point 2 is equal to t_{e2}. For Point 5 $(\sigma'_{v5}, \varepsilon_{v5})$, the time $t_{eOC}=t_{e5}$ can be calculated by letting $\varepsilon_{OC}=\varepsilon_{v5}, \sigma'_{OC}=\sigma'_{v5}$; for Point 6 $(\sigma'_{v6}, \varepsilon_{v6})$, $t_{eOC}=t_{e6}$ can be calculated letting $\varepsilon_{OC}=\varepsilon_{v6}, \sigma'_{OC}=\sigma'_{v6}$. It should be noted that the same value of C_α from NC state can be used to calculate the creep settlement of the same soil layer in the OC state. In fact, the creep settlement of the same soil layer in OC state is smaller than that in an NC state. This smaller creep settlement is considered by t_{eOC} in the above equation for $S_{creep,f}$. The $S_{creep,d}$ for the final point at an OC state (for example Points 2, 5, 6) is calculated using:

$$S_{creep,d} = \frac{C_\alpha}{1+e_0}\log(\frac{t+t_{eOC}}{t_{EOP,field}+t_{eOC}})\times H_s \qquad for \quad t \geq t_{EOP,field} \tag{7}$$

where t_{eOC} is calculated in the same way as that explained before.

In summary, all parameters used in this simple method are listed in the table below.

Table 2: Basic parameters used in the simple method

C_r	C_c	σ'_p (kPa)	C_α	$t_o = 1$ day	e_0	$\alpha = 0.8$	$\beta = 0.3$	k (m/day)

It is helpful to note that ε_p is calculated using σ'_p and C_r. The m_v is back-calculated using $\Delta\varepsilon_v$ and $\Delta\sigma'_v$ using C_r and C_c; and $c_v = k/(m_v\gamma_w)$ is calculated using k and m_v. The U in the equation of this simple method can be obtained using existing charts and equations.

Generally speaking, the equation of the simple method is used for a 1D straining condition simulated in a oedometer test. If vertical drains are installed in soils with uniform spacing, the consolidation for each equivalent cylindrical cell is in both vertical and radial directions. However, the vertical compressions for the soil layers with such vertical drains can be approximated by 1D straining. In cases of true 2D and 3D cases, such as a rectangular or strip foundation, the settlement calculated using the simple method is still useful, but shall be multiplied by a settlement correction coefficient. If the effective stresses in a soil layer are not uniform, for example, the vertical effective stress increasing with depth, this soil layer should be divided into a number of sub-layers for calculating S_f, $S_{creep,f}$, and $S_{creep,d}$. Similarly, if there are multiple soil layers with non-uniform effective stresses and soil parameters, these layers should be divided into sub-layers (Feng et al. 2020; Yin and Zhu 2020; Yin et al. 2022). A general simple method has been developed based on the simplified Hypothesis B method in Eq. (1) to the cases of multiple layers with or without vertical drains under complicated loading conditions (Yin et al. 2022) with verifications and applications.

7 Post-Construction Settlement Control

Post-construction settlements of dredged Hong Kong Marine Deposits (HKMD) can be calculated or predicted using the simplified Hypothesis B method (or the newly extended general simple method) or fully coupled consolidation analysis with a suitable Elastic Visco-Plastic (EVP) model. The control of the post-construction settlement can be achieved in two ways.

i. Make the dredged HKMD over-consolidated using PVDs with vacuum preloading with additional water/fill surcharge:

As shown in Figure 9, assuming the initial stress-strain point in a soil layer is at Point 1. Under the action of additional design loading pressure, the state point will move from Point 1 to Point 5'. Since Point 5' is on NC line, the creep settlement will be large. The post-construction settlement will be large since the creep settlement contributes significantly, provided that the long-term settlement is of interest. However, if we can use vacuum preloading with additional water/fill surcharge, we can make the HKMD over-consolidated. For example, the state point will move from Point 1 to Point 4, then unload to Point 5 under the same design loading pressure. Point 5 is on OC line, so that the creep settlement is much smaller, resulting in much smaller post-construction settlement. The simplified Hypothesis B method can calculate such creep settlements/post-construction settlements on OC line.

ii. Improvement of the dredged HKMD by deep cementing mixing (DCM) technique:

Generally, DCM can reduce the viscosity, increase the stiffness and shear strength of the HKMD so that the post-construction settlement can be reduced (Yin and Lai 1998; Yin and Fang 2006; Ho et al. 2021). In practice, the DCM technique is implemented by forming columns, clusters, walls, blocks, etc. (Kitazume and Terashi 2013). For example, DCM columns exhibit good efficiency in controlling settlements and transferring load from surrounding soil to DCM columns. The load transfer can induce an unloading process on surrounding soil, which makes the soil into an overconsolidated state with a smaller creep strain rate (Wu et al. 2020).

8 Conclusions

Systematic research works are ongoing in PolyU from element tests and physical model tests to theoretical modelling and engineering applications, on the development of a sustainable reclamation method using local dredged marine deposits. The works will contribute to research development, the design and construction of marine reclamations, and make a positive impact on Hong Kong society. The main conclusions can be drawn as follows:

(a) The marine reclamation method using local dredged marine deposits has been approved as a cost-saving and time-reduction effective method in Hong Kong and other coastal cities, with more test and field evidences coming up on the effectiveness of this method using dredged Hong Kong Marine Deposits (HKMD).

(b) A new soft soil improvement method combining horizontal and vertical drains with vacuum and/or surcharge preloading is proposed to strengthen the dredged HKMD slurry for marine reclamation works.

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(c) Two types of physical model tests are ongoing in PolyU's laboratory in order to simulate the whole process of implementing the new soft soil improvement method. Both of the tests well proved that the new improvement method is capable of decreasing water content and increasing soil strength effectively.

(d) Stabilizing HKMD slurry using alkali-activated industrial waste, like ISSA and/or GGBS, has a great potential for forming the crust layer on the top of vacuum preloading-treated slurry in an efficient and sustainable way.

(e) A new simplified Hypothesis B method is proposed. This method has been verified to be accurate and easy to use by engineers. This method can be used to predict the settlement of HKMD or other soft soils in layers and installed with/without drains subjected to staged loading unloading/reloading and even vacuum loading.

(f) The methodology of controlling the post-construction settlement by making the HKMD overconsolidated through vacuum preloading with additional water/fill surcharge or deep cement mixing method is also presented and discussed.

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