Anisotropy in the permeability and consolidation characteristics of dredged mud

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ABSTRACT

Dredging and land reclamation is a billion dollar industry that is spread around the coastal regions of Australia. Maintenance dredging is carried out regularly in many major Australian ports and in several cases the dredged mud is pumped ashore into containment paddocks at high water contents. When the dredged mud settles, there can be segregation with possible anisotropy. The sedimentation of the dredged mud is followed by self-weight consolidation which can take a long time. To accelerate the consolidation process, prefabricated vertical drains and surcharge are often used. To analyse the consolidation process of the dredged mud with vertical drains it is necessary to have a proper understanding of the anisotropic characteristics of the dredged mud. The objective of this paper is to summarise the results of laboratory consolidation studies carried out on reconstituted dredged mud specimens that were sedimented from slurry at high water contents in the order of 270%. The horizontal and vertical coefficients of consolidation were determined on separate oedometer tests with appropriate drainage boundaries. At every pressure increment, vertical permeability and volume compressibility were estimated to evaluate the variation of consolidation properties with depth. Strong anisotropy in horizontal and vertical directions was observed and the variations of consolidation properties and compressibility with depth were assessed.

Keywords: Consolidation, sedimentation, anisotropy, segregation, coefficient of consolidation, permeability

1 INTRODUCTION

Land reclamation work into the sea is undertaken when there is limitation in available land area for expanding major sea side developments, such as ports and airports. Maintenance dredging works are carried out in ports worldwide to ensure deep enough navigation channels for the sea going vessels. In order to find an environmentally friendly means for disposing the dredged material and replacing costly reclamation filling materials such as gravel and sand, the dredged mud is reused as filling materials in a number of land reclamation works undertaken worldwide. The land reclamation works carried out in the Central Japan International airport (Satoh and Kitazume 2003) and in the Port of Brisbane (PoB) expansion project, Australia are some of the examples.

The Port of Brisbane is located at the mouth of the Brisbane River at Fisherman Islands, and it is the major port in the state of Queensland, Australia. The port has seen continuous development until recent years to cater to the increasing trade demand. In order to expand the port for accommodating additional facilities to meet the development expected in the next 25 years, the Port of Brisbane has embarked on a land reclamation process adjacent to the existing land mass, which will ultimately see 235 ha of new reclaimed land area, at the completion of the project. The 4.6 km long rock and sand seawall constructed around the perimeter of the site in Moreton Bay bounds the area which is being reclaimed (Ameratunga et al. 2010). The seawall extends up to 1.8 km into Moreton Bay (Figure 1).

Every year around 300,000 m^3 of mud is extracted from the adjacent Brisbane River during maintenance dredging works in the navigation channel and berths. Land reclamation is undertaken by reusing these dredged materials in an environmentally feasible way of disposing of this dredged mud. The reclamation area enclosed within the seawall is partitioned into a number of containment paddocks. Dredged mud is pumped into the containment paddocks in a slurry form of water content of at least 200%, dewatered and allowed to undergo self weight consolidation. The height of dredged mud placement varies from 7 m to 9 m.

Figure 1: Aerial view of land reclamation site at Port of Brisbane, Queensland, Australia

Dredged mud is a weak, fine grained soil with predominantly 40% silt and 50% clay constituents (Ganesalingam et al. 2011). The dredged mud fill is underlain by highly compressible in-situ Holocene clays, with thickness of 9 to 30 m. As both in-situ clays and dredged materials are highly compressible and have low permeability characteristics, they are treated with surcharge loading together vertical drains to accelerate the consolidation process. Selecting appropriate soil properties is essential for reliable prediction of the degree of consolidation and future settlements. Here, both horizontal and vertical consolidation parameters are required when vertical drains are used. The design consolidation parameters used at the site are often estimated from correlations, back calculations and sometimes based on the properties of other similar Queensland marine clays.

When the dredged mud is pumped in a slurry form with water content higher than its liquid limit, potential grain size sorting occurs when the soil particles settle. Larger particles are carried to the bottom and finer particles stay at the top of the soil column. The degree of segregation depends on the settlement pattern of soil particles as well, which is influenced by the initial water content and salt concentration of slurry, as discussed widely in literature (Imai 1980, Imai 1981, Katagiri and Imai 1994, Sridharan and Prakash 2003). In brief, with a higher salt concentration in slurry, the soil particles tend to stay together with each other and results in flocculated or zone settling. If the initial water content of slurry is high with a low salt concentration, particles settle in a dispersed form. The combination of salt concentration and initial water content of slurry would yield one of the following settlement patterns as discussed by Imai (1980): dispersed free settling, flocculate free settling, and zone settling. The homogeneity of the final soil column formed is high for zone settling. The exact boundary values of the initial water content and salt concentration at which a particular settlement type becomes prevalent, depend solely on the mineralogical components of a soil type (Imai 1980). The above studies emphasize that when a soil layer is formed from slurry, the consolidation properties of the final soil column can be either uniform or non uniform throughout the soil layer, depending on the initial conditions of the slurry.

In order to verify this, an experimental study was conducted using dredged mud obtained from the PoB, as explained in detail in the following section. Both vertical and horizontal coefficients of consolidation were evaluated with oedometer tests to confirm the degree of anisotropy induced. The depth variation of consolidation properties and volume compressibility were also analysed.

The letter symbols used in the following sections are defined at their first appearance, and the full list is given at the end of the paper.

2 SETTLEMENT COLUMN TEST

Dredged mud samples obtained from the Port of Brisbane reclamation site was used for the experimental studies. Wet sieve and hydrometer analysis revealed that the dominating constituents in the dredged mud are 50% clay and 40% silt and the mud is classified as fine grained. Atterberg limits are in the range of 80-85% (Liquid limit- LL), 34-37% (Plastic limit- PL) and the Linear Shrinkage (LS) 18-19%. Plasticity Index (PI) was about 44-46. Dredged mud sample was initially sieved through a 2.36 mm sieve to eliminate all the broken shells and debris. They were remoulded in a slurry form at water content of around 270%. Sea water obtained from Townsville (in Queensland) was used to mix the slurry. Salt concentration of sea water was estimated to be approximately 370 N/m^3 by drying 1 L volume of water and finding the weight of salt crystals collected.

In a cylindrical tube of diameter 107 mm and height 810 mm a porous plate covered by a filter paper was placed at the bottom. Well mixed slurry was poured in to the tube up to its entire height and initially allowed to undergo self weight consolidation. A week later, another filter paper and porous top cap were placed on top of the soil column. When the soil column accomplished most of its self weight consolidation settlement, it was sequentially loaded with small weights in the range of 500 to 3000 g. The soil column was allowed to consolidate under each vertical stress increment for couple of days before the next weight was added. The soil column was loaded up to a maximum vertical stress of 21 kPa over duration of 8 weeks. The final thickness of the column at the completion of consolidation was around 230 mm.

Four oedometer specimens of 76 mm diameter, 20 mm height, two each from the bottom (namely 1A and 1B) and top (namely 2A and 2B) of the final soil column were extruded. The locations of the samples are shown in the schematic diagram in Figure 2.

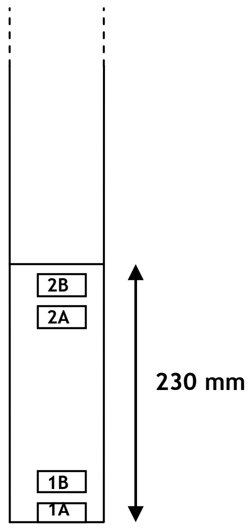

Figure 2: Specimen extrusion for oedometer tests

Specimens 1A and 2A were used for standard one dimensional consolidation tests. Specimens 1B and 2B were tested for radial consolidation with an outer peripheral drain. The material used for outer peripheral drain was 1.58 mm in thickness. The strip of the drain was aligned along the inner periphery of the oedometer ring. A special cutting ring of diameter of around 72.84 mm was used to cut specimens 1B and 2B and then they were carefully transferred into the ring with drain. The porous bottom and top caps used for standard vertical consolidation tests were replaced with two impermeable caps, for radial consolidation tests.

The specimens were loaded in the oedometer approximately between a vertical stress range of 9 kPa to 440 kPa (9 kPa, 17 kPa, 30 kPa, 59 kPa, 118 kPa and 220 kPa and 440 kPa). A load increment ratio of around 1.0 was adopted throughout the loading stage.

2.1 Results

For each vertical stress increment, vertical and horizontal coefficients of consolidation c_v and c_h were estimated for relevant specimens. Taylor's square root of time method was used to estimate c_v. For finding c_h, the curve fitting method suggested by McKinlay (1961) was used. The settlement vs. time plot, where the time values raised to the power of 0.465 (t^0.465), was drawn. The rest of the curve fitting method is similar to the Taylor's square root of time method, except, a slope factor of 1.22 was used instead of 1.15 to find the t_{90} point. c_v and c_h values for samples 1A,1B and 2A,2B were compared In Figures 3(a) and 3(b) respectively. Effective vertical stress (σ'_v) is given in log scale in all the following figures.

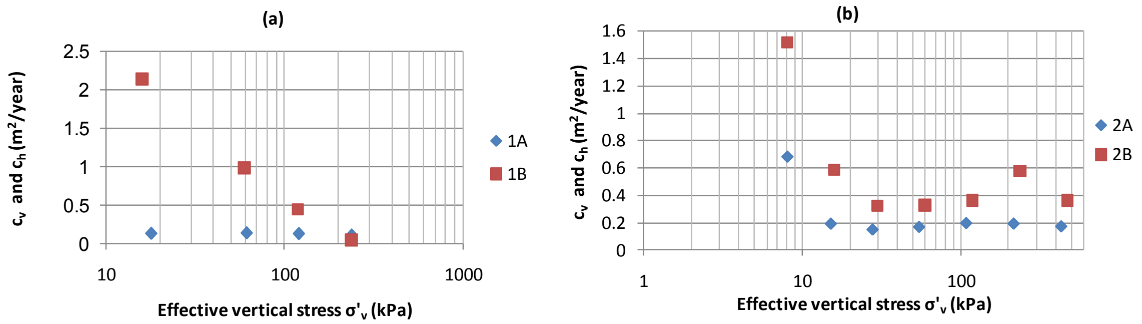

Figure 3: Comparison of c_v and c_h for specimens at bottom (a) and top (b)

As observed, horizontal coefficient of consolidation c_h is always higher than the vertical coefficient and strong anisotropy can be noticed. At the vertical stress level of 17 kPa, c_h is almost 16 times the c_v value in Figure 3(a) and 3 times higher in Figure 3(b). The degree of anisotropy (c_h/c_v) is higher at the initial low vertical stress levels and tends to reduce as the vertical stress increases, except for few discrepancies. At σ'_v of around 120 kPa, the degree of anisotropy is still about 2, both in Figures 3(a) and 3(b). The reason for the c_h to be higher than c_v could be because of the orientation the soil particles attain when they settle from slurry. This can be further ascertained by Scanning Electron Microscopy (SEM) photographs of specimens. Overall c_v and c_h reduce with the increasing σ'_v. Over the vertical stress range of 17-240 kPa, c_h varies from 2 to 0.04 m^2/year for specimen 1B. c_h values of specimen 2B drops from 1.5 to 0.36 m^2/year in between of 8 to 470 kPa. Variation of c_v with σ'_v is not as significant as the variation of c_h. For a σ'_v variation between 17 kPa to 120 kPa, c_v values lies in the range of 0.132-0.129 m^2/year for specimen 1A and 0.193-0.197 m^2/year for specimen 2A.

In order to observe any depth variation of consolidation properties, c_v values for specimens extruded from the bottom (1A) and top (2A) of the column were compared in Figure 4(a). Similarly c_h values for specimens (1B) and (2B) were compared in Figure 4(b).

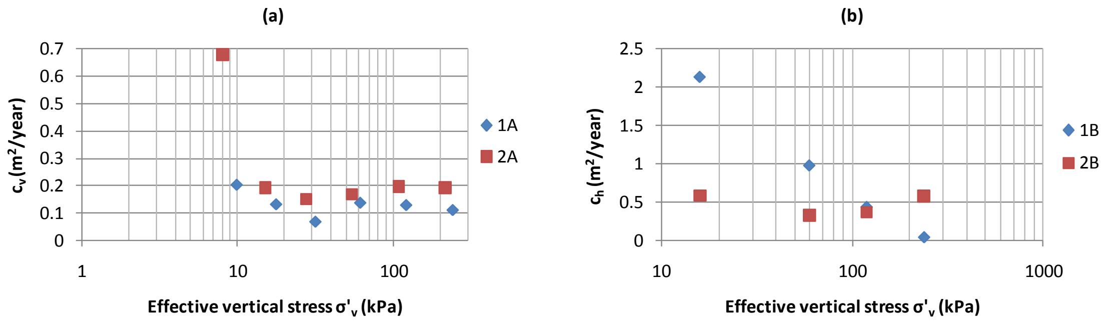

Figure 4: Depth variation of c_v (a) and c_h (b)

At any vertical stress increment, c_v at the top (specimen 2A) is higher than at the bottom (specimen 1A). Except at $\sigma'_v \sim 10$ kPa, at all vertical stress increments, c_v of specimen 2A remains less than 2 times of specimen 1A. Unlike in Figure 4(a), c_h values at the bottom (specimen 1B) is higher than at the top (specimen 2B), especially for the lower σ'_v values. The variation is large as well, which reduces with the increment in σ'_v, except the discrepancy at σ'_v = 120 kPa. To further explain the segregation with depth, the vertical permeability values k were computed from equation 1, using the volume compressibility m_v and c_v values estimated.

$$k = c_v . m_v . \gamma_w \quad (1)$$

where, γ_w is the unit weight of water. Variation of k and m_v with σ'_v are plotted in Figures 5(a) and 5(b) respectively, for specimens 1A and 2A.

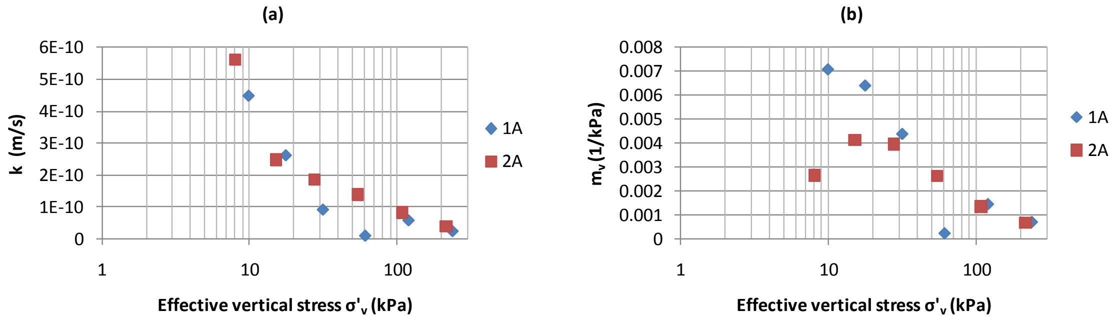

Figure 5: Depth variation of k (a) and m_v (b)

As expected k and m_v values generally reduce with the increment of σ'_v. Vertical permeability k is at least two times higher for specimen 2A than 1A, for σ'_v range in between 20 kPa – 60 kPa. This is similar to the c_v trend as discussed from Figure 4(a). From Figure 5(b), specimen 1A is highly compressible than 2A over σ'_v of 9 to 20 kPa. Again the depth variations of m_v and k become insignificant at high vertical stress levels (> 100 kPa).

From the results considerable depth variation of consolidation properties and compressibility is demonstrated. When the soil particles settle from slurry it is usual that the heavier coarse particles fall through and rest at the bottom while the finer particles stay at top. This should have produced higher c_h, c_v and k values for the specimen extruded from the bottom. But the results are in contrast for c_v and vertical permeability k. The reason for this is not clear yet. But the author believes this is mainly to do with the orientation the soil particles attain when they settle; which could differ with the depth.

3 CONCLUSION

When the dredged mud is mixed in a slurry with water content significantly higher than its LL, and allowed to settle, the resulting sedimentation and self-weight consolidation process would induce anisotropy in consolidation properties and segregation of particles. This commonly occurs in land reclamation works using the maintenance dredged spoils, as in the Port of Brisbane (PoB) land reclamation project. This phenomenon was verified with a laboratory experimental study. PoB dredged mud was remoulded at a water content of around 270%, poured into a cylindrical column and reconstituted. At the completion of self-weight consolidation within the sedimentation column, oedometer specimens were extruded from the top and bottom of the final soil column and tested for vertical and horizontal consolidation properties c_v and c_h. The results show the presence of strong anisotropy between c_h and c_v, probably due to the orientation of particles attained when they settle within the slurry. The variation of consolidation properties with depth, induced by any potential particle segregation or orientation, was confirmed with consolidation properties c_v and c_h, vertical permeability k and volume compressibility m_v of specimens extruded from the top and bottom of soil column. c_h was found to be the highest at the bottom, even though, c_v and permeability k were smaller at the bottom than at top. The degree of anisotropy and depth variation of consolidation properties is large at the lower vertical stress increments, which become less significant with higher stress increments. Anisotropy in consolidation properties and their variation with depth can be inevitable in practical terms. Estimating the coefficient of consolidation with back calculations and in-situ piezocone tests can take these effects to some degree, rather than using empirical correlations.

4 ACKNOWLEDGEMENTS

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APPENDIX A: Notifications

c_h	Coefficient of horizontal consolidation
c_v	Coefficient of vertical consolidation
k	Permeability
LL	Liquid limit
LS	Linear Shrinkage
m_v	Coefficient of volume compressibility
PoB	Port of Brisbane
PI	Plasticity Index
PL	Plastic limit
t	time
σ'_v	Effective vertical stress