Port of Brisbane Land Reclamation Project and the Use of Reconstituted Dredged Mud

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ABSTRACT: This paper addresses the Port of Brisbane (PoB) land reclamation project, in the state of Queensland, Australia, where maintenance dredged materials are reused to fill the reclamation site. Engineering challenges associated with stabilising and minimising the post construction settlement of the very soft dredged mud fill and in situ Holocene clays are described. Laboratory tests were undertaken to study the consolidation and compressibility properties of reconstituted dredged mud with particular focus on the degree of anisotropy between the horizontal and vertical coefficient of consolidation. The results discussed in this paper show the strong anisotropy observed between the horizontal and vertical consolidation properties.

1 INTRODUCTION

Port of Brisbane in the State of Queensland, Australia has embarked on a land reclamation process adjacent to the existing land mass, aiming to acquire 235 ha of reclaimed area to accommodate new port facilities over the next 25 years. The 4.6 km long rock and sand seawall constructed around the perimeter of the site bounds the area which is being reclaimed (Ameratunga et al. 2010a). The seawall extends up to 1.8 km into Moreton Bay (Fig.1). Annually around 300,000 m^3 of mud is extracted from the adjacent Brisbane River during the maintenance dredging works. Land reclamation is undertaken by reusing these dredged materials in an environmentally friendly manner, as a way of disposing the dredged mud. The reclamation area is partitioned into a number of containment paddocks. Dredged mud is pumped into the containment paddocks in a slurry form of water content 200 - 300 % and allowed to undergo self-weight consolidation. The height of the dredged mud placement varies from 7 m to 9 m.

Dredged mud is a weak, fine grained soil with predominantly 40% silt and 50% clay constituents. The dredged mud fill is underlain by highly compressible in-situ Holocene clays, with thickness varying from 9 m to as much as 30 m. With such a large compressible thickness of both dredged fill and the underlying in situ Holocene clays, the total settlement under development loads during the primary consolidation will be significant. The secondary compression will be another considerable component of settlement to deal with. Considering the low permeability characteristics of the clays, the site is treated with preloading and vertical drains to accelerate the consolidation process and minimize the post construction secondary compression. Selecting appropriate soil properties is essential for the reliable prediction of the degree of consolidation and future settlements. Hence, both horizontal and vertical consolidation parameters are required when vertical drains are used.

The paper outlines the land reclamation works with the design of preloading and vertical drains in order to stabilize the underlying weak soils by consolidation. Laboratory consolidation test results are discussed which were conducted on reconstituted dredged mud specimens prepared simulating the sedimentation and consolidation process at the reclamation site. Both vertical and radial consolidation tests were carried out and the degree of anisotropy between the horizontal and vertical coefficient of consolidation was evaluated.

2 SITE CONDITIONS

The upper Holocene clays generally consist of sand layers with interspersed soft clays and silts, which accelerate the pore water pressure dissipation. The lower Holocene clay layer, on the other hand, controls the rate of settlement at the site, because of its greater thickness and the absence of sand layers to accelerate pore water pressure dissipation. As both in-situ clays and dredged materials are highly compressible, settlement due to filling alone could be as high as 2 m even before any service loads are imposed. It is predicted that it would take as much as 50 years for the area to be consolidated considering surcharging as the only treatment option. Ground improvement by combined preloading and vertical drains is designed to accelerate the majority of expected primary settlement and limit the long term post construction settlement. The maximum vertical stress exerted under the development loads can vary over the site (15 kPa to 60 kPa) depending on the different purposes the land is used for. According to the design requirement of the Port of Brisbane, the long term residual settlement should not exceed 150 mm over a period of 20 years for applied pressures up to 50 - 60 kPa in areas where the Holocene clay thickness is less. The settlement limit is greater for the deeper Holocene clay areas (Boyle et al. 2009)

Fig. 1 Aerial view of land reclamation site at Port of Brisbane, Queensland, Australia

3 DESIGN OF PRE LOADING AND VERTICAL DRAIN SYSTEM

Considering the past case studies from Australia which witnessed the underperformance of vertical drains or ‘wick drains’ in accelerating the ground settlement of soft soils, Port of Brisbane Corporation (PBC) wanted to conduct several wick drain trials within the PoB reclamation site itself, before embarking on a full scale treatment (Boyle et al. 2009 and Ameratunga et al. 2010a). In highly sensitive soft soils, the installation of wick drains causes disturbance to the soil around the perimeter of the drains, which is referred as ‘smear’. This results in reduction in strength and permeability of the soil in the disturbed area. For an early completion of consolidation process, placing the drains at closer spacing can be considered as an option. However the installation disturbance can be more in the soil. Therefore arriving at optimum drain spacing for the Holocene clays and dredged fill was one of the intentions of conducting the wick drain trials. The wick drain trials were implemented with number of wick drain type, filter type, spacing and configuration or grid patterns of the drains to pick the best suited ones for the varying subsurface conditions across the reclamation site. At some locations the vertical drains has to be driven as deep as 30 m to consolidate the entire thickness of the Holocene clay layer.

Vacuum consolidation system was considered more appropriate than the conventional preloading by embankments, especially for areas closer to the edge of the sea wall, where the slope stability of the sea wall may be an issue. Proper instrumentation comprising of piezometers, extensometers and deep settlement plates were installed to monitor the time change of excess pore water pressure and settlement of the subsurface layers. The efficiency of the wick drain and vacuum consolidation system was assessed from the time taken to accomplish the consolidation. The expected post construction settlement was checked if it meets the design criteria. From the series of wick drain trials together with vacuum consolidation, it was concluded that the wick drains performed satisfactorily well in treating the dredged fill and Holocene clays.

3.1 *Secondary compression*

The secondary compression settlement expected is significant because of the large compressible thickness of composite Holocene layers and dredged mud fill. To deal with this, the applied preloading should be able to remove some of the secondary compression settlement in addition to the primary consolidation settlement. The subsoil is subjected to a preloading higher than the expected post construction design load, thus, the underlying soil will be in an over consolidated state under the design loads. The coefficient of secondary compression $C_{\alpha e}$ depends on the over consolidation ratio (OCR), and it drops quickly with a small increment in the OCR ratio (Ameratunga et al. 2010a; Ameratunga et al. 2010b; Alonso et al. 2000; and Wong, 2007). The following exponential law given in Eq. 1 is adopted for the reduction of $C_{\alpha e}$ with OCR.

$$C_{\alpha e(OC)}/C_{\alpha e(NC)} = [(1-m)/e^{(OCR-1)n}]+m \qquad (1)$$

m is taken as 0.1, which is equivalent to the ratio of C_r/C_c (Mesri 1991) and n is equal to 6. At the PoB reclamation site, the underlying soil is over consolidated to an OCR ratio between 1.1 and 1.2.

4 LABORATORY TESTS

Particle size analysis revealed that the dredged mud sample contains about 10 % sand, 35 % silt and 55% clay. Liquid Limit, plastic limit and plasticity index lie in the range of 80 - 85 %, 34 – 37 % and 44 – 46 % respectively. X-Ray Diffraction (XRD) tests revealed that the major constituents of PoB dredged mud are Quartz, Kaoline and Illite.

Dredged mud sample was initially sieved through a 2.36 mm sieve to eliminate the debris and then mixed with sea water at a water content of 270 % in a slurry form. Sea water obtained from Townsville (in Queensland) was used to mix the slurry (Salt concentration 370 N/m^3). The dredged mud slurry was placed in a cylindrical tube of 100 mm diameter and 800 mm height and allowed to undergo sedimentation. When the dredged mud 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 two days before the next weight was added. Pore water dissipation was allowed through the porous caps placed at the top and bottom of the dredged mud column. The soil column was loaded up to a maximum vertical stress of 21 kPa over a duration of 8 weeks. The final thickness of the column at the completion of consolidation was around 300 mm.

From the final sediment, specimens were extruded for the oedometer tests. Six oedometer specimens of 76 mm diameter, 20 mm height, were extruded at three different depth levels as shown in Fig.2. Three specimens were subjected to standard vertical consolidation tests (denoted by 'V') and three were tested for radial consolidation tests (denoted by 'R'). The procedure for the radial consolidation tests is explained below briefly.

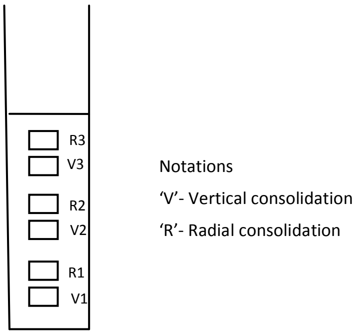

Fig. 2 Specimen locations for oedometer tests

Specimens R1, R2 and R3 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 drain was aligned along the inner periphery of the oedometer ring. A special cutting ring of diameter of 72.84 mm was used to cut specimens. The cutting ring had a circular flange at its bottom. A groove was carved along the inner periphery of the flange, which had a thickness equal to the thickness of the bottom edge of oedometer ring plus peripheral drain. The oedometer ring was placed tightly in the groove, to make it align properly with the cutting ring (Fig.3). The specimen in the cutting ring was then carefully transferred to the oedometer ring using a topcap, without causing any disturbance. The porous bottom and top caps used for standard vertical consolidation tests were replaced with two impermeable caps, for radial consolidation tests. All the specimens were loaded in the oedometer apparatus approximately between a vertical stress range of 9 kPa to 440 kPa. A load increment ratio of around 1.0 was adopted throughout the loading stage. From the settlement – time data of the specimens under each load increment, the vertical and radial coefficients of consolidation c_v and c_h were estimated. Taylor's square root of time method was used for estimating c_v. c_h was obtained from the curve fitting procedure proposed in McKinlay (1961) for radial consolidation with a peripheral drain.

Fig. 3 Specimen preparation for radial consolidation test

5 RESULTS

Figs. 4(a), (b) and (c) show the comparison of c_v and c_h for pairs V1-R1, V2-R2 and V3-R3 respectively at different effective vertical stresses σ_v'. The degree of anisotropy, given by (c_h/c_v) is plotted against σ_v' in Fig. 4(d) for the three pairs of specimens.

As shown, horizontal coefficient of consolidation is higher than that in the vertical direction at all three depths. The ratio c_h/c_v generally decreases with the increase in σ_v'. At low σ_v' ($\sigma_v' < 20$ kPa), the ratio c_h/c_v varies from 2 to as much as 70. In Fig. 4(d), at moderate stress levels (50 – 60 kPa), the degree of anisotropy lies in the range of 2 to 10. Based on the above observation, the assumption of equal design c_v and c_h values for the recent dredged mud fill may have to be reviewed.

The above finding contradicts with the common expectation of a remolded young clay sediment showing isotropic permeability and coefficient of consolidation at low stress levels, because of random particle arrangement, i.e less fabric anisotropy (Clennell et al. 1999; Lai and Olson 1998). Nevertheless, few studies identified similar observation of higher c_h and k_h values than c_v and k_v for remolded normally consolidated clays at low stress levels (Sridharan et al. 1996; Robinson 2009).

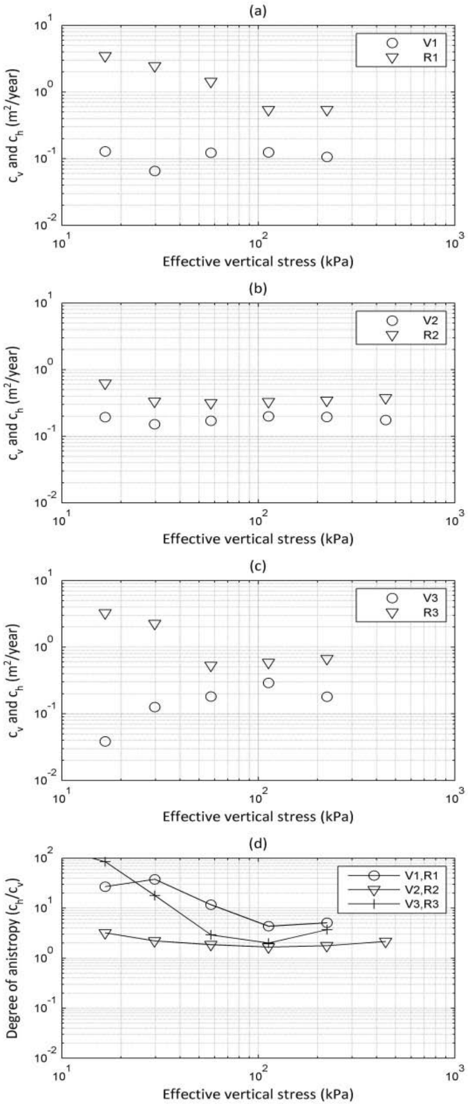

Fig. 4 Comparison of c_v and c_h for specimens (a) V1, R1 (b) V2, R2 (c) V3, R3 (d) Degree of anisotropy

For a vertical stress level between 50 – 60 kPa, c_v values of specimens V1, V2 and V3 varies between 0.1 – 0.2 m^2/year (Fig. 4). When compared to the design c_v obtained from in situ tests, this is about 5 to 10 times smaller. It has been reported that the laboratory tests generally result in lower c_v values than in situ test values for southeast Queensland clays (Ameratunga et al. 2010a).

6 CONCLUSIONS

This paper summarizes the Port of Brisbane land reclamation works, with particular focus on the stabilization of dredged mud fill and in situ clays by means of consolidation. Design of vertical drains and pre loading is discussed. The sedimentation and consolidation process of the dredged mud at the reclamation site was simulated in the laboratory. Standard vertical and radial consolidation tests were conducted on the reconstituted dredged mud specimens. The findings show that significant degree of anisotropy (2 – 10) can exist between the horizontal and vertical coefficient of consolidation in a recent PoB dredged mud sediment, under the post construction loads (50 – 60 kPa).

ACKNOWLEDGMENTS

The writers would like to acknowledge Warren O'Donnell (Senior Engineering Technician, Geomechanics Laboratory, James Cook University) for his assistance with laboratory experiments. The cash and in-kind support provided by the Australian Research Council, Port of Brisbane Pty Ltd and Coffey Geotechnics are gratefully acknowledged.

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