Land reclamation on soft clays at Port of Brisbane

Construction d'un terre-plein sur des sols argileux dans le port de Brisbane

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ABSTRACT: Land reclamation work is being carried out for the Port of Brisbane (PoB) Future Port Expansion Project, in the State of Queensland, Australia, which would develop a new reclaimed area of 235 ha. The mud excavated during maintenance dredging operations is pumped into the containment paddocks within the reclamation area. The dredged mud is soft and fine grained in nature, placed in a remoulded dilute slurry form at water content of at least 200%. The reclamation site is underlain by weak Holocene clays of depths up to 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. This paper briefly addresses the maintenance dredging works and the background of the PoB land reclamation project. It mainly focuses on the site conditions, the design of surcharge loading and installation of vertical drains to accelerate the consolidation settlement and minimize the post construction secondary compression. Furthermore, the in-situ and laboratory tests undertaken and the soil parameters obtained from these tests are discussed along with empirical correlations used to estimate appropriate soil parameters.

RÉSUMÉ : Des travaux d'aménagement s'inscrivant dans le cadre du « Future Port Expansion Project » (Projet d'extension du futur port) sont menés dans le Port de Brisbane, dans l'Etat du Queensland en Australie. Ce projet prévoit la création d'une étendue de terre de 235 ha gagnées sur l'eau. Les boues issues des opérations de dragage de maintenance sont pompées vers des bassins au sein même de la zone aménagée. Ces boues naturellement molles et fines forment une suspension diluée contenant au minimum 200% d'eau. Le site repose sur des terres argileuses datant de l'Holocène sur une profondeur allant jusqu'à 30 m. Avec une telle épaisseur d'argile compressible d'origine in-situ ou provenant du dragage, le tassement total dû aux charges de développement lors de la consolidation primaire sera significative. La compression secondaire constituera une composante supplémentaire à prendre en compte pour le tassement. L'article présente brièvement les travaux de dragage de maintenance et le contexte du projet de terre-plein dans le Port de Brisbane. Il se concentre principalement sur les caractéristiques du site, la conception des suppléments de charge et l'installation de drains verticaux pour accélérer la consolidation de l'installation et minimiser la compression secondaire liée à la construction. Les tests menés en laboratoire et sur le site ainsi que les paramètres du sol obtenus à partir de ces tests, et les corrélations empiriques utilisées pour estimer ces paramètres sont également abordés.

KEYWORDS: land reclamation, dredged mud, sedimentation, consolidation, vertical drains

1 INTRODUCTION

Dredging and land reclamation is a billion dollar industry asscociated with the ports throughout the coastal region of Australia. Maintenance dredging is carried out regularly in many major Australian ports and in some cases the dredged mud is reused as filling materials in the land reclamation works undertaken near the coast. The land reclamation works carried out in the Port of Brisbane (PoB) expansion project, Australia is one 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. In order to expand the port to accommodate 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. 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 carried out in the navigation channel and berths. 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 of at least 200 % 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. Since both in-situ clays and dredged mud are highly compressible and have low permeability characteristics, they are treated with preloading together with vertical drains to accelerate the consolidation process. Selecting appropriate soil properties is essential for 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. Detailed review of the design strength, consolidation and compressibility parameters used for the recent dredged mud fill is given. In addition, laboratory tests results are discussed which were conducted on reconstituted dredged mud specimens prepared simulating the sedimentation and consolidation process at the reclamation site.

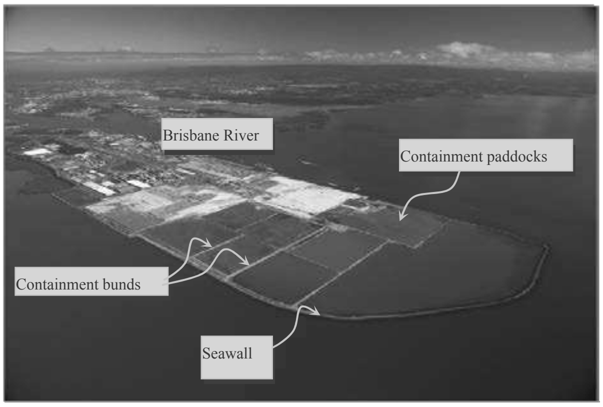

Figure 1: Aerial view of land reclamation site at Port of Brisbane, Queensland, Australia

2 SITE CONDTIONS

The Holocene clay layers include upper and lower Holocene clay layers. The upper Holocene layer consists of sand layers with interspersed soft clays and silts, thus the pore water pressure dissipation and the settlement rate is relatively fast. The lower Holocene clay layer, where the sand and silt layers are relatively few, controls the rate of settlement at the site because of its large compressible thickness. As both the in-situ Holocene clays and dredged mud fill 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 preloading as the only soil treatment option. Therefore vertical drains are incorporated to speed up the consolidation process. Ground improvement by combined preloading and vertical drains is designed to accelerate the majority of expected primary settlement and limit the long term postconstruction settlement. 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 but the settement limit is greater for the deeper Holocene clay areas (Ameratunga et al. 2010a).

Maximum vertical stress exerted under the development loads (i.e buildings, traffic) can vary over the site between 15 kPa and 60 kPa depending on the different purposes the land would be used. In addition, the total thickness of compressible clays is variable over the site. The above two factors decide the amount of preloading to be applied at the ground level. Preloading is applied by both sand capping and vacuum preloading. Thickness of sand capping layer varies from 6 m to 9 m across the site. Initially, a number of vacuum trials was conducted at several test sites within the reclamation area itself in order to assess the effectiveness of wick drains to be used as a ground improvement measure (Ameratunga et al. 2010b)

The subsoil layers at the reclamation site is subjected to a preloading higher than the expected post construction design load, so that the underlying soil will generally be in an over consolidated state under design loads. In the over consolidated stage (recompression range), the settlements in both the primary consolidation and secondary compression range are significantly less than in the normally consolidated stage, which will be discussed later.

2.1 *Design Parameters of dredged mud fill*

The design strength and consolidation parameters of dredged mud fill used at the site are estimated from both in-situ and laboratory tests. In the absence of the above, correlations with physical properties of the clays are used for the preliminary assessment of properties. Atterberg limit values of dredged mud are in the range of 80-85% (Liquid limit- LL), 34-37% (Plastic limit- PL), 18-19% (Linear shrinkage- LS) and 44-46% (Plasticity Index- PI). The major constituents in the PoB dredged mud are 50% clay and 40% silt. From the Atterberg limits and particle size distribution, the PoB dredged mud can be classified as high plasticity clayey soil.

The undrained shear strength of recent dredged fill is evaluated from in situ vane shear and piezocone (CPTu) dissipation tests. In some instances, the shear strength parameters are estimated from the following empirical correlations incorporating PI values.

$$c_u / \sigma_v' = 0.11 + 0.0037 * PI \tag{1}$$

$$\text{Sin } \phi' = 0.8 - 0.094 * \ln(PI) \tag{2}$$

where c_u and ϕ' are the undrained shear strength and drained friction angle respectively.

Piezocone dissipation test results are also used to estimate the consolidation properties such as coefficient of consolidation c_v or c_h and approximate permeability. The c_v values calculated from the in situ tests are verified from the back calculations using Asaoka's method from field monitoring. A c_v value of 1 m^2/yr is used for the dredged mud fill and the coefficient of consolidation in the vertical and horizontal directions (c_v and c_h) are assumed to be equal.

At the PoB reclamation site, there are insufficient records yet for long time period settlements, thus the secondary compression parameters are estimated only from the laboratory tests and correlations. The subsoil is subjected to preloading higher than the expected post construction design load. As a result, the underlying soil will generally be in an over consolidated stage under 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. 2010b; Alonso et al. 2000; Wong, 2007). For the reduction of $C_{\alpha e}$ with the *OCR* the following exponential law has been adopted (Eq.3).

$$C_{\alpha e}(OC) / C_{\alpha e}(NC) = [(1-m)/e^{(OCR-1)n}] + m \tag{3}$$

m is taken as 0.1, which is equivalent to ratio of C_r/C_c (Mesri,1991) and n is equal to 6. At the PoB reclamation site the underlying soil is generally over consolidated to an *OCR* ratio of 1.1-1.2. An average value of 0.008 was adopted for design $C_{\alpha e}$.

The design compression ratio *CR*, given by $C_c/(1+e_o)$ (e_o - Initial void ratio), is taken as 0.2 to 0.3 based on laboratory tests. Recompression ratio *RR* (= $C_r/1+e_o$) is generally taken as 0.1 times the compression ratio.

3 LABORATORY TESTS

The sedimentation and consolidation of dredged mud were simulated in the laboratory using the dredged mud samples obtained from the PoB reclamation site. The objective of the laboratory tests is to evaluate some of the consolidation parameters (c_v and c_h) and compressibility properties (*CR* and *RR*) of reconstituted dredged mud sediment and make comparison with the design values. In addition, potential anisotropy that can exist between the horizontal and vertical coefficients of consolidation and permeability was investigated. Series of oedometer tests were conducted in the present laboratory studies.

The mud was initially sieved through a 2.36 mm sieve to eliminate all the broken shells and 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 to radial consolidation tests (denoted by 'R'). The procedure for the radial consolidation tests is explained below briefly.

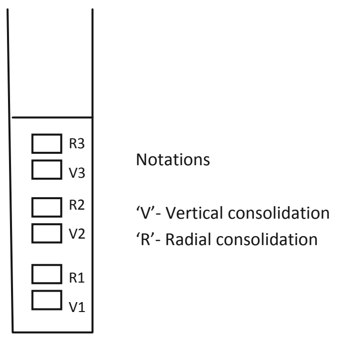

Figure 2: Specimen locations for oedometer tests

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 top cap, 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 (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. 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 given in McKinlay (1961) for radial consolidation with a peripheral drain.

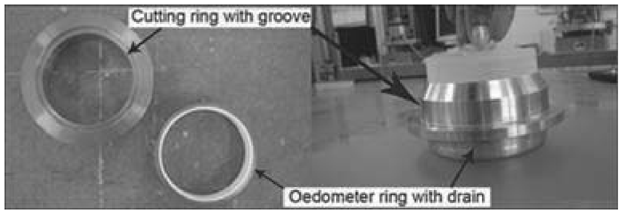

Figure 3: Specimen preparation for radial consolidation test

3.1 Results and discussion

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.

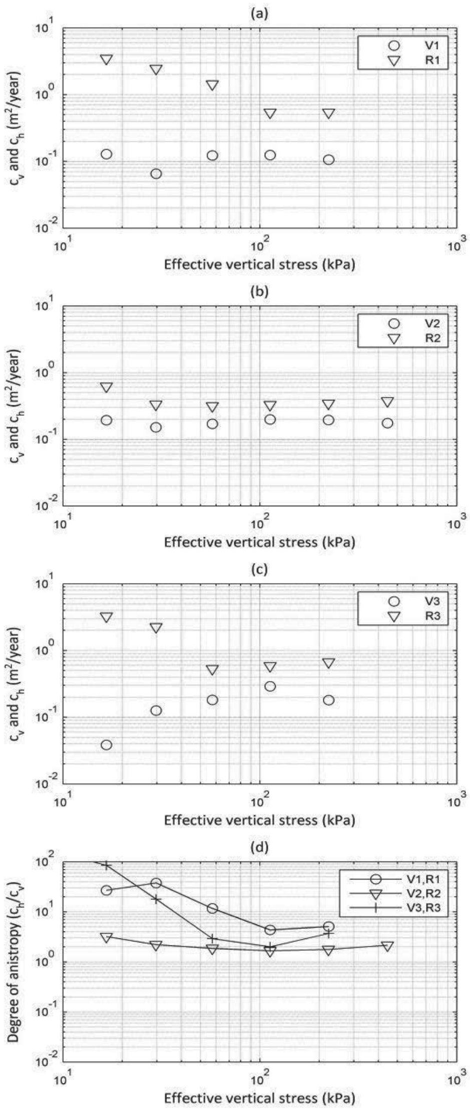

Figure 4: Comparison of c_v and c_h for specimens (a) V1, R1 (b) V2, R2 (c) V3, R3 (d) Degree of anisotropy

As clearly observed, the 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 (σ'_v < 20 kPa), the ratio c_h/c_v varies from 2 to as much as 100. The average degree of anisotropy in permeability (k_h/k_v) for the various stress levels is given in Table 1. The ratio k_h/k_v lies between 1 to 4. The horizontal permeability and coefficient of consolidation are higher than those in the vertical direction.

In remolded young clay sediment, the permeability and coefficient of consolidation are expected to be isotropic at low stress levels, since the particles are arranged in a random way, and there will be less fabric anisotropy (Clennell et al. 1999; Lai and Olson 1998). However, similar observations of higher c_h and k_h values than c_v and k_v have been reported elsewhere for remolded normally consolidated clays at low stress levels (Sridharan et al. 1996; Robinson 2009). In Fig. 4(d), at the 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.

Table 1: Anisotropy in permeability (k_h /k_v)

σ'_v (kPa)	k_h/k_v
50-100	2.0
100-200	3.5
200-300	1.1

For a vertical stress levels 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).

The compression ratio (*CR*) and recompression ratio (*RR*) are given in Table 2 for all the six specimens. The *CR* values of specimens varies between 0.15- 0.36 and the *RR* values lie in the range of 0.02-0.035. These values agree well with the design *CR* and *RR* values discussed in the previous section.

Table 2: Compression and recompression ratio of specimens (*CR* & *RR*)

Specimen	*CR*	*RR*
V1	0.228	0.031
R1	0.378	0.035
V2	0.153	0.019
R2	0.219	0.062
V3	0.204	0.029
R3	0.159	0.020

4 CONCLUSIONS

In the paper, a review of the Port of Brisbane land reclamation works is given including the site conditions, design of vertical drains and soil parameters. 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 results show that a large degree of anisotropy can exist between the horizontal and vertical coefficients of consolidation and permeability in young clay sediment. The c_v values obtained from the laboratory tests were found to be 5 to 10 times smaller than the field values. The compression ratio *CR* and recompression ratio *RR* are in good agreement with the design values.

5 ACKNOWLEDGEMENTS

The writers would like to acknowledge Warren O'Donnell (Senior Engineering Technician, Geomechanics Laboratory, School of Engineering & Physical Sciences James Cook University, Townsville, Australia) for his assistance in conducting the 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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7 NOTATIONS

C_c - Compression index
c_h - Horizontal/ Radial coefficient of consolidation
CR - Compression ratio
C_r - Recompression index
c_u - Undrained shear strength
c_v - Vertical coefficient of consolidation
$C_{\alpha e}$ - Coefficient of secondary compression
e_0 - Initial void ratio
k_h - Horizontal permeability
k_v - Vertical permeability
LL - Liquid limit
LS - Linear shrinkage
NC - Normally consolidated
OC - Over consolidated
OCR Over consolidation ratio
PI - Plasticity index
PL - Plastic limit
RR - Recompression index
σ'_v - Effective vertical stress
$\varnothing'$ - Drained friction angle