Geotechnical properties of reconstituted dredged mud

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ABSTRACT
In the present study, laboratory tests were undertaken in order to assess the index properties and consolidation characteristics of dredged mud which is excavated during the maintenance dredging works carried out in Port of Brisbane, Queensland, Australia. These dredged spoils are reused as the fill materials in the major land reclamation project currently undertaken adjacent to the port for expanding the port facilities. In the laboratory, index properties of dredged mud such as, particle size distribution, Atterberg limits and specific gravity were determined. Oedometer tests were conducted to calculate the consolidation properties of dredged mud. Laboratory results for consolidation properties were compared with the design values used in Port. Reasonable agreement can be noticed between the laboratory and field compression and recompression ratio CR and RR. Field coefficient of consolidation c_v obtained from back calculations is higher than the laboratory results.

RÉSUMÉ
Dans cette étude, des tests ont été effectués en laboratoire dans le but d'évaluer les propriétés et les caractéristiques de consolidation de boues excavées lors de travaux de maintenance dans le port de Brisbane, Queensland, Australie. Ces boues sont réutilisées sur place comme matériau de comblement dans un projet en cours de réalisation pour l'agrandissement des installations portuaires. Des propriétés fondamentales de ces boues, telles que la distribution de la taille des particules, les limites d'Atterberg et la gravité spécifique ont été déterminées au laboratoire. Des tests oedométriques ont également été réalisés afin de calculer les propriétés de consolidation des boues. Les résultats expérimentaux obtenus pour les propriétés de consolidation ont été comparés aux valeurs utilisées pour la construction au niveau du port. Une bonne concordance a été observée entre les valeurs du laboratoire pour le taux de compression et de décompression CR et RR. La valeur du coefficient de consolidation c_v obtenu à l'aide des mesures expérimentales précédentes est plus élevée que celle obtenue le laboratoire.

1 INTRODUCTION

Land reclamation work in sea is undertaken when there is limitation in 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 ways for the sea 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, 2002) 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 main 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, which will 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 sea wall extends up to 1.8 km into Moreton Bay (Fig 1). Every year 300,000 m^3 of mud is extracted from the adjacent Brisbane River during maintenance works in the navigation way for vessels. Land reclamation is undertaken by reusing these dredged materials, as an environmentally feasible way of disposing of this dredged mud.

The reclamation site is underlain by weak and compressible deep Holocene clays with thickness varying from 9 m to 30 m, which will undergo significant settlements due to the post construction loads, such as buildings and traffic. The situation is exacerbated by the placement of reclamation materials, which are significantly weaker than the underlying Holocene clays, with predominantly 40% silt and 50% clay constitutions. 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 soil treatment option. Therefore vertical drains are incorporated to speed up the consolidation process.

The reclamation area enclosed within the sea wall is partitioned into a number of containment paddocks. Dredged mud is pumped into the containment paddocks in a slurry form, dewatered and allowed to undergo self weight consolidation. The height of dredged mud placement varies from 7 m to 9 m. Following capping with a layer of sand, a large surcharge loading is subsequently applied after installation of the vertical drains to speed up the consolidation process. Once the fill and the underlying compressible clays attain a significant portion of consolidation settlement, the surcharge load is removed allowing construction work to commence.

Figure 1: Aerial view of Land reclamation Site at Port of Brisbane, Queensland, Australia

Ground improvement by combined surcharging and vertical drains has to be designed to remove the majority of expected primary settlement and limit the long term post construction settlement. According to the design requirement of the Port of Brisbane, the long term residual settlement should be within the limits of 150-250 mm over a period of 20 years for applied pressures up to 50 - 60 kPa (Ameratunga et al. 2010a). Accurate prediction of the consolidation settlement, both primary and secondary, is therefore of key importance. Choosing appropriate soil properties is essential for reliable prediction of the degree of consolidation achieved; 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.

In the present study, Index properties of PoB dredged mud samples such as Atterberg limits, particle size distribution and specific gravity were determined in the laboratory. Oedometer tests were conducted in order to assess the consolidation characteristics of dredged mud. The results of laboratory tests and the geotechnical properties of PoB dredged mud used at the site are compared and discussed in detail. Also, geotechnical properties of three other selected soft clays are summarized and compared with those of PoB dredged mud. 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 INDEX PROPERTIES OF DREDGED MUD SAMPLES

Dredged mud samples used in the laboratory tests were obtained from the reclamation site in the Port of Brisbane. The average water content of the samples was around 80%. Index properties of PoB dredged mud such as, particle size distribution, Atterberg limits and specific gravity were determined in the laboratory. The results were compared with the basic properties of another dredged mud sample which was obtained from the Port of Townsville (TSV), also in Queensland, and tested in the laboratory (Table 1). From the Atterberg limits and particle size distribution, the PoB dredged mud can be classified as highly plasticity clayey soil. TSV dredged mud shows likely properties of a highly plasticity silty soil. The wet sieve and hydrometer analysis revealed that the dominating constitutions in the PoB dredged mud are 50% clay and 40% silt; and the mud is classified as fine grained soil. Atterberg limit values 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 average dry density of the dredged mud was 1223 kg/m^3. Based on limited laboratory tests done on the characteristics of dredged clays in the Port, Atterberg limits were in the range of 80-100% (LL) and 55-75% (PI). The index properties are often used in an empirical relationship to assess the strength and consolidation properties of dredged mud, as undisturbed sampling for laboratory tests is difficult in such soft soil with high water content, whilst the tests take much more time. Incorporating PI values in the empirical correlation proposed by Skempton & Henkel (1953) for normally consolidated clay (equation 1), the undrained shear strength (c_u/σ_v') for PoB dredged mud fill was taken as 0.35.

$$c_u / \sigma_v' = 0.11 + 0.0037 * PI \qquad [1]$$

The derived values were found to be in agreement with those obtained from in situ vane shear strength tests conducted using Pilcon vane within the paddocks. According to Mitchell's (2005) equation, for undisturbed clay, the relationship between the friction angle (ϕ') at critical void ratio and PI is given by equation 2.

$$\sin \phi' = 0.8 - 0.094 \times \ln(PI) \qquad [2]$$

However, considering remoulded dredged material which is sensitive to disturbance and based on some previous experience in South East Queensland marine clays, a residual friction angle of 15 degree was selected for design purposes.

Table 1: Characteristics of dredged mud

Characteristics	PoB	TSV
Liquid limit (%)	80-85	73
Plastic Limit (%)	34-37	30
Linear shrinkage (%)	18-19	15
Plasticity Index	44-46	44
Specific Gravity	2.65	2.70
Sand (%)	5-12	14
Silt (%)	38-47	85
Clay (%)	41-51	<1

3 OEDOMETER TESTS

Disturbed dredged mud sample obtained from the Port of Brisbane reclamation site was remoulded and reconstituted in the laboratory and specimens were extruded for the oedometer tests. The 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 a water content of around 450%. In a cylindrical tube of diameter 150 mm and height 585 mm, a porous plate covered by a filter paper was placed at the bottom. The 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 (Figure 2). The soil column was allowed to consolidate under each vertical stress increment for 3-7 days, before the next weight was added. The column was loaded up to a maximum vertical stress of 6500 g over a duration of 8 weeks. The final thickness of the column at the completion of consolidation was around 100 mm and the final average water content of the sediment was 112% in average. Four specimens (namely A, B, C and D) were cut from the settlement column using 63mm diameter, 20 mm height oedometer cutting rings and transferred into the oedometer apparatus for the consolidation tests. Specimens A and B were cut from the top of the final soil column, both at the same level, similarly, specimens C and D were cut from the bottom of the soil column. The average initial water contents of each specimen were 110.1% (A), 112.4% (B), 116.4% (C) and 111.6% (D). In each oedometer, the loading path followed is as follows: The specimens were successively loaded with 6 kPa, 11.5 kPa, 22 kPa, 40 kPa, 78 kPa and 156 kPa of vertical stress, then unloaded to 78 kPa - 40 kPa and again reloaded from 40 kPa to 78kPa, 156 kPa up to a maximum vertical stress of 310 kPa. A load increment ratio of 1.0 was maintained throughout the loading, unloading and reloading stages. Taylor's square root of time method was used to check if the end of primary consolidation had reached under each vertical stress increment, and if so, the next stress increment was added. At the vertical stress of 78 kPa during the loading stage, specimens were allowed to undergo some degree of secondary compression.

Figure 2: Soil settlement column

3.1 Results

For each vertical stress increment, the coefficient of consolidation c_v, coefficient of volume compressibility m_v, and permeability k were estimated and their variation with effective vertical stress (σ'_v) was plotted as shown in Figures 3, 4 and 5. Taylor's square root of time method was used in estimating the coefficient of consolidation c_v. As seen in the figures, the consolidation properties of dredged mud significantly change with effective vertical stress. During the earlier stress increments (σ'_v = 6 kPa, 11.5 kPa), the c_v values of all specimens decrease largely with effective vertical pressure and then increase gradually (Fig 3). During the unloading stage, the c_v values peak under the vertical effective stress of 78 kPa, then drops down again at the effective stress of 40 kPa. The c_v values again increase largely during the reloading stage (σ'_v= 78 to 155 kPa) and then again decrease at the maximum vertical effective stress of 310 kPa.

In the field, a value in the order of 1-2 m^2/yr is generally adopted for c_v based on back calculation of parameters of all trials as well as development areas at the site (Boyle et al. 2009). The horizontal coefficient of consolidation c_h is considered to be twice of the c_v value. In figure 3, over the σ'_v range of 30-160 kPa, during the loading stage, a peak value of 0.35 m^2/yr is observed for c_v in specimen A, out of all four specimens. Although during the unloading and reloading stages peak values of 2.3 m^2/yr (specimen D) and 0.9 m^2/yr (specimen A) are observed respectively. Overall the laboratory c_v values are less than the design c_v values estimated from back calculation. Dredged mud possibly contains sandy or silty materials, which advantages in speeding up the consolidation process is significant in entire fill rather than in a laboratory specimen, hence the back calculated field c_v values could be higher than the laboratory results.

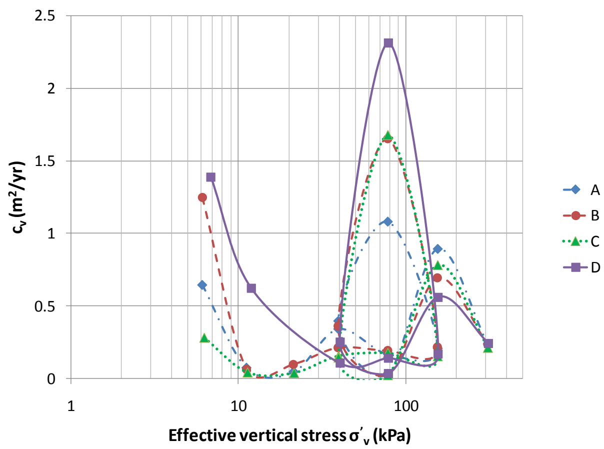

Figure 3: Coefficient of consolidation (c_v) Vs Effective vertical stress (σ'_v)

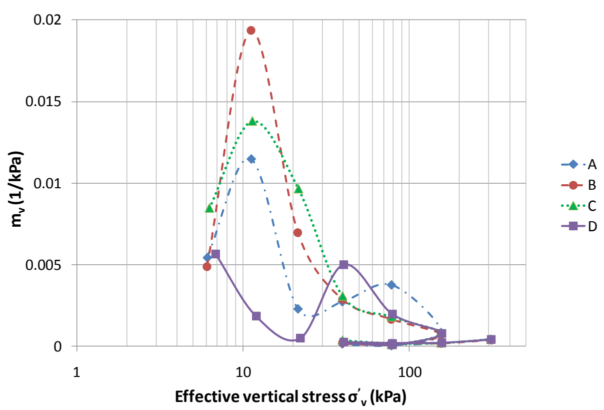

Figure 4: Coefficient of volume compressibility (m_v) Vs Effective vertical stress (σ'_v)

The m_v values increase largely for smaller values of σ'_v, (σ'_v < 12 kPa) and then reduces gradually, as observed in specimens A, B and C (Figure 4). Specimen D shows a sinusoidal variation of m_v with σ'_v. Over the loading stage the maximum m_v value of 1.95 x 10^{-2} (1/kPa) is observed at σ'_v=11.5 kPa for specimen B, and the minimum is 4 x 10^{-4} (1/kPa) at σ'_v= 22 kPa for specimen D. Majority of m_v values are more than 10^{-3} (1/kPa) over the loading stage from 6 kPa-156 kPa. The m_v values of all specimens are equal throughout unloading and reloading stage, in the order of 5 x 10^{-4} (1/kPa). Permeability k of specimens are in the order of 0.05 x10^{-9} to 2.5 x 10^{-9} (m/s) initially at σ'_v = 6 kPa, and declines abruptly at the next immediate load increment (Figure 5). The k values of specimens vary from 3 x 10^{-10} to 0.5x10^{-10} (m/s) over the σ'_v range of 12 to 156 kPa. During the loading and unloading stages the permeability values of all specimens are similar and below 10^{-10} (m/s).

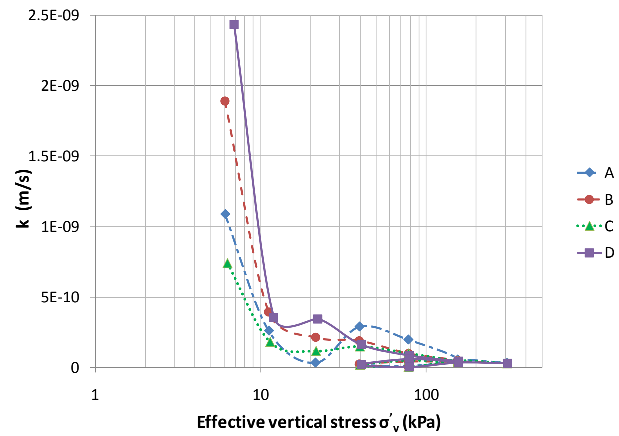

Figure 5: Coefficient of permeability (k) Vs Effective vertical stress (σ'_v)

The relation of void ratio at the end of primary consolidation (e_p) with σ'_v was plotted during the loading, unloading and reloading stages (Figure 6). The compression index C_c and recompression index C_r, were estimated over an approximate stress range of 20 – 156 kPa and 40-156 kPa, respectively. The results are given in Table 2. In the field, based on some laboratory tests, a value between 0.2-0.3 was estimated for compression ratio CR, given by $C_c/(1+e_o)$ (e_o - Initial void ratio). Considering very soft normally consolidated, highly plastic remoulded clay of moisture content greater than 100, a design value of 0.3 was adopted for CR. Recompression ratio RR (=$C_r/1+e_o$) is generally taken as 0.1 times the compression ratio. The results obtained for CR and RR in the laboratory in the range of the design values.

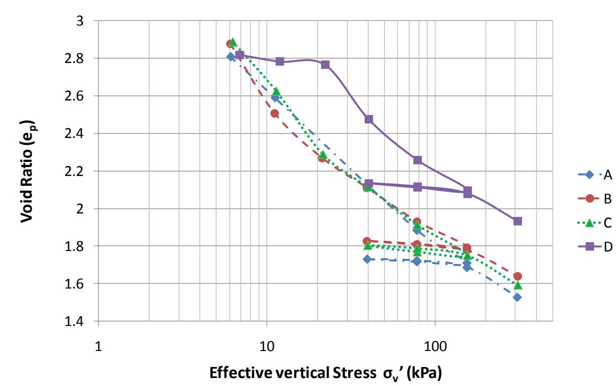

Figure 6: Void ratio (e_p) Vs Effective vertical stress (σ'_v)

Table 2: Compression ratio (CR) and recompression ratio (RR) of samples

	A	B	C	D
Compression Index (C_c)	0.825	0.564	0.657	0.560
Recompression index (C_r)	0.056	0.0617	0.095	0.09
Compression ratio (CR)	0.210	0.142	0.161	0.142
Recompression ratio (RR)	0.014	0.015	0.023	0.023

According to the Port of Brisbane design requirements, the long term secondary compression under post construction loads is expected to be within the limits of 150 mm-250 mm for a period of 20 years under the in-service loads up to 50 - 60 kPa. The applied surcharge load is determined mainly taking into account the design loads, thickness of underlying clay layer to be consolidated and OCR required; thus the thickness of sand capping layer varies from 6 to 9 m across the site.In the field, the creep strain, $C_{\alpha e}$ (=$C_\alpha/(1+e_0)$), was observed to be in the range between 0.004 and 0.012, under the vertical stress from 50 kPa to 100 kPa (at normally consolidated stage), based on some laboratory tests performed on samples extruded at different depths in the dredged mud fill. An average value of 0.008 was adopted for design $C_{\alpha e}$. The site is surcharged prior to the construction works commenced, and thus, the underlying soil layer remains in an overconsolidated stage under the post construction loads. The coefficient of secondary compression C_α depends on the over consolidation ratio (OCR), and it drops quickly with a small increment in the OCR ratio, as reported in the literatures (Ameratunga et al. 2010b; Alonso et al. 2000; Wong, 2007; Wong, 2006).For the reduction of $C_{\alpha e}$ with the OCR the following exponential law has been adopted (Equation 3).

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

m is taken as 0.1, which is equivalent to ratio of C_r/C_c (Mesri,1991) and n is equal to 6. In the present study, in order to assess the secondary compression behaviour of the dredged mud samples, under the vertical stress increment of 78 kPa during the loading stage, the oedometer specimens were allowed to undergo compression beyond the end of primary consolidation. The C_α values were 0.112, 0.113 and 0.097 mm per log cycle time for specimens A, B and D respectively. The $C_\alpha/(1+e_0)$ values were 0.0286, 0.0284 and 0.0245. The laboratory secondary compression coefficients are higher than the design values except in specimen C, where the secondary compression curve was observed to be almost flat. More oedometer tests are necessary for better assessing the secondary compression behaviour of dredged mud samples.

4 GEOTECHNICAL PROPERTIES OF OTHER SOFT SEDIMENTS

Available geotechnical properties of three other selected soft clay materials worldwide are summarized below. First one are the bottom sediments of Maumee River, Ohio in USA, where annual maintenance dredging works undertaken as part of the deepening process of waterways (Salem and Krizek, 1976). Samples were obtained in a slurry form and tested in the slurry consolidometer. The other two soft clays are Changi marine clay, in Singapore and Bangkok clay, undisturbed samples of which were obtained adjacent to Asian Institute of Technology (AIT), Bangkok. Ohio clay showed Atterberg limits values in the range of 61-76% (LL), 21-50% (PL) and 21-48% (PI). The average clay content was 41%. Average c_v values of Ohio dredged clay varied between 0.3-0.4 (m^2/yr), while the maximum c_v values observed at various tests were in the range of 0.55-0.83 (m^2/yr), for a vertical stress range of 28-220 kPa. Permeability k values were observed to be below 4×10^{-10} (m/s) for vertical stress under 220 kPa. The m_v values were in the range of 12×10^{-3} to 1.74×10^{-4} (1/kPa) over the vertical stress range of 20-164 kPa. Undisturbed samples of Bangkok clay were obtained over a depth of 9m, where the in situ effective stress varied from 0 to 80 kPa and the clay layer was in over consolidated stage with OCR of around 1.5 (Bergado et al. 1990). LL and PL were 50-110% and 20-40% respectively. Specific gravity was around 2.65-2.70. Consolidation tests revealed that the C_c values varied from 0.6-1.0 over the entire depth. c_v and c_h were in between 0.3-1.5 m^2/yr and 1- 5.5 m^2/yr respectively, but the stress range over which the values were estimated are not given. Field vane shear strength varied in between 15-40 kPa over the depth. The vertical pressure in Changai marine clay layer varies up to 200 kPa over a depth of 30m, and the soil layer is in over consolidated stage (OCR=1.2). The c_v and c_h values of undisturbed Changi marine clay, without potential inclusion of sand or silt materials, were estimated to be 1.0 m^2/yr and 2.0 m^2/yr under the consolidation pressure of 200 kPa (Choa et al. 1979). Compression index CR values were in between 0.2-0.4 for a vertical stress range of 100- 200 kPa. Undrained shear strength values, obtained from vane shear strength tests vary between 5-10 kPa over the depth from 0 up to 20m. The LL and PL were in the range of 70-90% and 20-40% respectively.

The clay content and consolidation properties c_v, m_v and k of disturbed Ohio clay are comparable with the properties of remoulded and reconstituted PoB dredged mud obtained from the present laboratory studies. Undisturbed samples of Bangkok clay show higher C_c values than of PoB dredged mud. CR values, and c_v and c_h properties of Changai marine clay are similar to the design values of PoB dredged mud obtained from laboratory tests and back calculations, although the former soil layer is in over consolidated.

5 CONCLUSIONS

Consolidation characteristics of remoulded and reconstituted PoB dredged mud together with its basic properties were assessed with laboratory tests and compared to the design parameters used in the PoB reclamation site. At the Port, the design parameters are estimated using laboratory tests, back calculations, and correlations. Compression ratio CR and recompression ratio RR obtained in the present study from the oedometer tests conducted on reconstituted samples are in the same range of design values chosen. Design c_v adopted in the field is generally 2-3 times higher than the laboratory c_v values. As the dredged mud is pumped in a slurry form into the containment paddocks, the coarser particles settle in the vicinity of the pipe and the finer particles carried away. This results random inclusions of sand or silty material over the reclamation fill. In present experimental study, dredged mud samples was remoulded at high water content and allowed to settle, and thus segregation of soil particles along the depth of settlement column can be expected; the deviation between the consolidation characteristics of the four specimens tested in the present study can be accounted to this segregation occurred. The oedometer tests conducted on reconstituted samples can be a good guide in estimating CR and RR values. For the design c_v and c_h values back calculations are more reliable as the effect of potential inclusions of sandy, silty materials throughout the fill is neglected in the oedomter tests. More oedometer tests are required to arrive at a better conclusion on the secondary compression behaviour of dredged mud samples. The consolidation properties of PoB dredged mud obtained in the present experimental study are similar to those of Ohio clay.

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NOTIFICATIONS

CR	Compression ratio
c_c	Compression index
c_h	Coefficient of horizontal consolidation
c_r	Recompression index
c_u	Undrained shear strength
c_v	Coefficient of consolidation
c_α	Coefficient of secondary compression/creep
$c_{\alpha e}$	Creep strain
e	Void ratio
e_o	Initial void ratio
e_p	Void ratio at the end of primary consolidation
k	Permeability
LL	Liquid limit
LS	Linear Shrinkage
m_v	Coefficient of volume compressibility

NC	Normally consolidated
OC	Over consolidated
OCR	Over-consolidation ratio
PoB	Port of Brisbane
PI	Plasticity Index
PL	Plastic limit
RR	Recompression ratio
σ'_{v}	Effective vertical stress
ϕ'	Friction angle