Case History

Ground Improvement in Port of Brisbane (PoB) Clay

Jay Ameratunga, Cynthia De Bok
Coffey Geotechnics, Brisbane, Australia
Peter Boyle
Port of Brisbane Corporation, Fisherman Islands, Brisbane, Australia
Daniel Berthier
Menard Bachy, Sydney, Australia

The Port of Brisbane (PoB), located at the mouth of the Brisbane River at Fisherman Islands, is undertaking a reclamation expansion of a 235ha sub-tidal area using maintenance dredged materials from the adjacent river. The reclaimed site is underlain by soft dredged materials up to 9m thickness as well as soft to firm Holocene clays in the natural sea bed as deep as 30m. Extensive ground improvement is required for such deposits prior to releasing the land for development. A well planned set of ground improvement trials involving international operators were conducted to optimise and select suitable techniques prior to rolling out. The project won the Innovation Award in 2009 from the Institution of Engineers Queensland Division in Australia. This paper describes the project, characteristics of the PoB clay, the reasons for deciding on trials and a discussion of some results from the trials.

Port of Brisbane

Port of Brisbane (PoB) is the main container port of the State of Queensland on the east coast of Australia (Figure 1). It is located in the lower reaches of the Brisbane River on reclaimed land at the mouth of the river. The land reclamation had been in progress since the early 1980's and the current Port footprint, now called Fisherman Islands, is almost entirely constructed in the adjacent Moreton Bay. Figure 2 shows the history of reclamation in Fisherman Islands and the gradual reclamation and development towards the northeast since 1958.

Figure 1: Site location (Courtesy of Port of Brisbane Corporation)

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Figure 2: Fisherman Islands and Port of Brisbane (Courtesy of Port of Brisbane Corporation)

Being the third largest container port in the country, Port of Brisbane has been steadily developing to cater for increasing trade growth. In recent years, there has been a rapid growth in the demand for port land as a result of increased trade growth in the South East Queensland region, which is expected to continue beyond the next 25 years. In 1999, the Port of Brisbane Corporation embarked on the expansion of a 235ha sub-tidal area immediately adjacent and northeast of the existing land. As a first step, a 4.6km long Future Port Expansion (FPE) seawall (Figure 3) was constructed around the perimeter of the site in Moreton Bay so that progressive reclamation works can be carried out within the boundaries of the seawall (Ameratunga et al, 2005).

The area contained by the FPE seawall required that the Port should achieve two objectives:

- To provide land for development to cater for future needs
- To act as a receptacle to dispose of the materials generated from the river maintenance dredging activities in the Brisbane River channel

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Figure 3: Future Port Expansion (FPE) Seawall - 2004

Although the above benefits are significant to the Port, the site posed significant challenges to make the reclaimed land ready for development. The most challenging is the weak and compressible soil formation across the site. The latter can be summarised as Holocene deposits overlying Pleistocene deposits. The Holocene deposits include weak and compressible clays, and occur as paleo-channels across the site. The depth of the Holocene clays (or PoB clays) varies from a few metres to as deep as 30m. The conditions are exacerbated after reclamation, because the reclamation is undertaken using the maintenance dredged materials from the adjacent Brisbane River. These materials are classified as clay or silt and commonly referred to as “dredged mud”, significantly weaker than the underlying PoB clays. The thickness of these layers also varies, up to about 9m across the reclamation areas, leading to overall clay thicknesses of more than 30m. As both in-situ clays and dredged materials are highly compressible, settlement due to filling alone could be as high as 2m even before any service loads are imposed. Improving such deep deposits provided a challenging exercise in ground improvement considering the performance criteria required by the Port of Brisbane Corporation in its short and long term utility.

Therefore it is imperative that the land is improved prior to any development unless expensive solutions such as pile foundations are adopted for all infrastructure, and even then, serviceability could still be an issue. It was estimated that it would take in excess of 50 years to treat this land using conventional reclamation and surcharging methods, due to the existing soft clay depths compounded by the thickness of the overlain dredged mud.

Initial desk studies by Coffey made an assessment of relative merits of the many ground improvement methods that could be used under the site conditions. They included surcharging with wick drains or sand drains, vacuum consolidation, stone columns, deep soil mixing, and controlled modular columns. Preliminary cost estimates were carried out based on rates provided by the industry and/or similar construction works in the State of Queensland or any other State. The results indicated that wick drains would be the least expensive solution for the wider area with more specific solutions to be adopted for the boundaries, i.e., edges abutting the Moreton Bay Marine Park.

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Wick drains, to our knowledge, had never been used to improve such deep deposits of clays nor dredged muds in Australia although case histories from overseas are available in the public domain. Initial enquiries from the local market indicated that none of the machines had mast lengths sufficiently long to penetrate the deep clays found at the Port. Furthermore, in South East Queensland, there have been several documented cases of underperformance of wick drains on projects since the late 1980's. There can be several reasons for such underperformance of which the subsoil characteristics are most likely to be one of them. In addition, there were many unknowns, especially relating to the installation technique and the potential smearing of the wick drains. The Port of Brisbane Corporation (PBC) was concerned because a small trial within Fisherman Islands a few years back also provided similar poor results. PBC accordingly required some certainty of performance, including timelines, with respect to the assessment of the effectiveness of the various techniques and to optimise designs before embarking on a full scale treatment of the site, to ascertain the most applicable methods of consolidating the soft clay soils. However, neither PBC nor Coffey wanted to suppress innovation and therefore formulated the trials to allow the ground improvement specialists/contractors to propose their own solutions.

Tenders were called internationally for ground improvement techniques that could be effectively applied over a selected area of the Port as a trial. Once the tenders were closed it was clear that wick drains were nominated as the preferred solution for the main trial areas with vacuum consolidation being nominated by one of the contractors for the edge areas. The trials were expanded so that both types could be trialled at the same time. The trials were completed in December 2008 and provided valuable learnings on wick drain and vacuum treatment in reclamation works.

Geological Setting

The geological setting is described in many of the geotechnical reports related to the development of the Port. A brief description is given below (Ameratunga et al, 2010).

The geomorphology of northern Moreton Bay and the southern Sunshine Coast area has undergone major changes in the last twelve thousand years. This period marks the end of the previous Ice Age at a time when the sea level was around 150m lower than it is today and the coast line was approximately 25km to the east of its current location.

Since then, the bay has been emptied approximately four times and partially in-filled approximately five times in response to world-wide changes in sea level caused by minor Ice Ages at various times. These fluctuations in sea level resulted in a complex series of sediment layers and erosion surfaces; each incursion laid down sediments, which were then partially or totally eroded as the sea level subsequently fell. The remaining sediments were then covered over by subsequent incursions and the cycle continued. At the present day, the sea level is unusually high when compared to the typical sea levels over the previous one hundred and twenty thousand years. In late Pleistocene times, the sea level rose progressively from around 150m below its current level (with a shoreline around 25km east of Moreton Island) approximately 19,000 years ago, to slightly above its current level around 6000 years ago, before settling at its current level. The buried land surface between the older Pleistocene sediments and the more recent Holocene sediments is of particular interest in formulating a geological model for the study area.

The massive barrier islands of North and South Stradbroke, Moreton and Bribie Islands dominate the Moreton Bay area. In the protected landward area of Moreton Bay, there is a general zoning of recent sediments roughly corresponding to the local sedimentary conditions.

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Near the mouth of the Brisbane River, the dominant sediments are marine sand, silt and clay deposited from the waters of the Brisbane River as they enter the still water of the bay. Around the coastal areas of the northern suburbs, Redcliffe and Deception Bay, marine muds derived from local terrestrial erosion dominate the sediments. In the northern and southern parts of the bay, sandy sediments are present, which have been washed into the bay by tidal currents.

In the study area, four distinct geological units have been recognised and they are listed from top down in Table 1.

Table 1 Geological Units

Unit	Description
Recent	Marine and dune sands with layers of silt and clay. This material may include fill including dredged fill. It is generally silty clay, although a variation in material characteristics across the paddocks is expected because of the single point discharge system with coarser materials depositing closer to the discharge point.
Holocene	Normally consolidated marine clay, silt and sand. A more detailed description is given later in this paper.
Pleistocene	These are older sediments that lie below the PoB clays or the pre-Holocene land surface and consist of overconsolidated, stiff to very stiff silty and sandy clay with layers of clayey sand. The compressibility of these materials is relatively low compared to the soft/firm clays of the Holocene deposits.
Tertiary	The weathered basalt bedrock of the Petrie Formation underlies the site and is described as grey-green clay (extremely weathered basalt) grading downwards into dark grey to black, moderately to slightly weathered basalt.

Port of Brisbane (PoB) Clay

The depth of Holocene sediments or PoB clays has a significant impact on the development because the clayey materials within this profile are compressible leading to high settlements. The basal contours of the PoB area (Figure 4) show how the paleochannels cut across the site. The base of the layer appears to vary from about RL-5m to deeper than RL -35m.

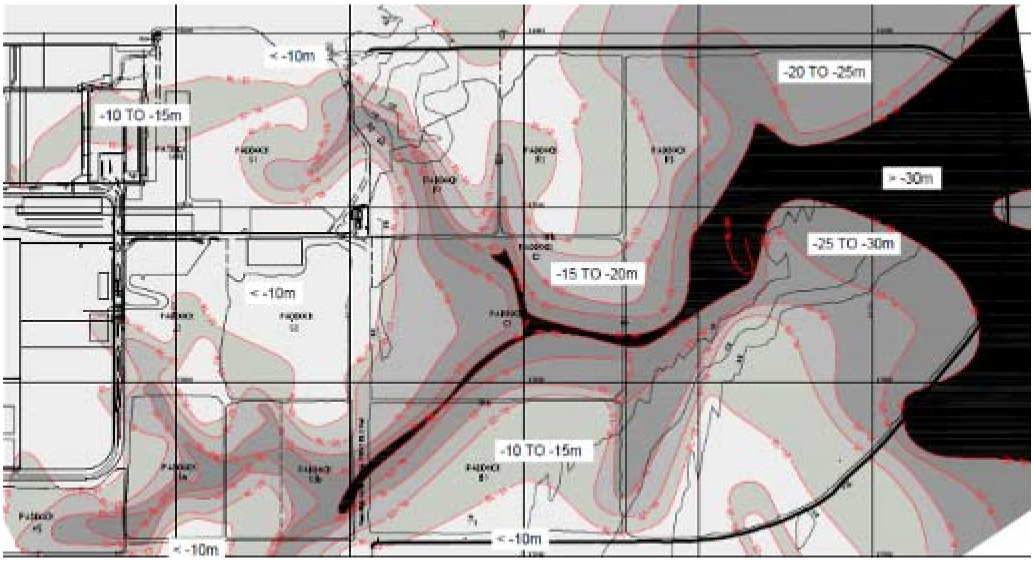

Figure 4: Base contours of PoB clay

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Generally, the Holocene sediments are divided into an upper layer and a lower layer of normally consolidated, low strength silty clay with shell bands ("marine clay"), separated by a discontinuous layer of sand. The upper layer generally consists of sand layers interspersed with layers of soft clays and silts. Sand layers or lenses are relatively few or absent within the lower layer. A typical profile is shown in Figure 5 as a piezocone plot.

The upper layer does not pose significant constraints although it is compressible and leads to settlement. The rate of settlement of this layer is generally rapid because of interspersed sand layers accelerating the dissipation of pore pressures. In the natural soil profile at the site, the lower layer generally controls the rate of settlement because of its greater thickness and the absence of sand layers to accelerate pore water pressure dissipation. Therefore, apart from the reclamation fill, this layer poses the most significant constraint on the development of the land for future use. More attention was therefore directed during the investigation and design phases to understand the behaviour of this lower layer. The bulk of the results presented in this paper are related to this layer where most testing was carried out.

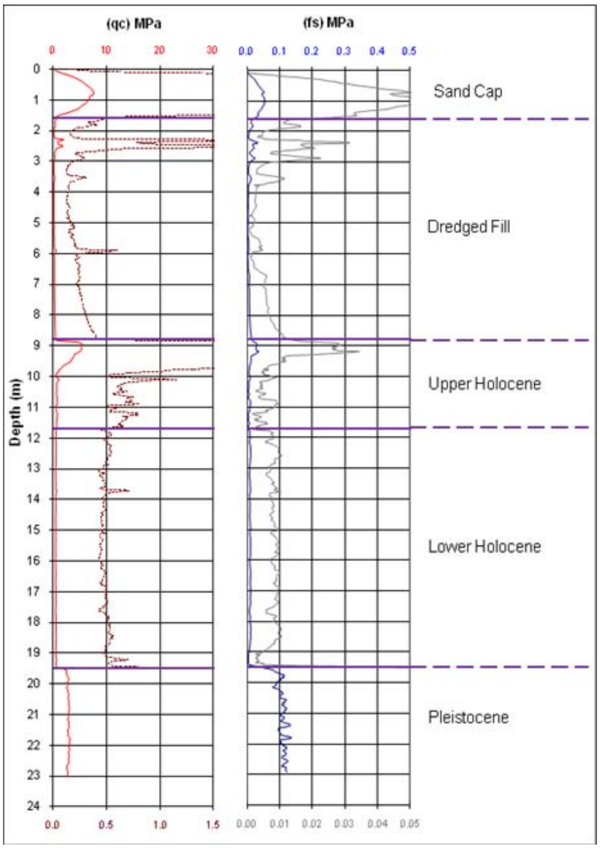

Figure 5: Typical piezocone profile

Index Properties of PoB Clays

PoB clays are found to be highly plastic as can be seen from the Atterberg limit test results plotted on the Casagrande plasticity chart (Figure 6). This figure shows a wide variation of the liquid limit, generally ranging from about 40% to 100%, and the plasticity index generally ranging from 20 to 70. Most importantly, the results clearly show the materials to be clays rather than silts. These laboratory results confirm visual observations on site when clay samples are taken from the PoB clay deposit.

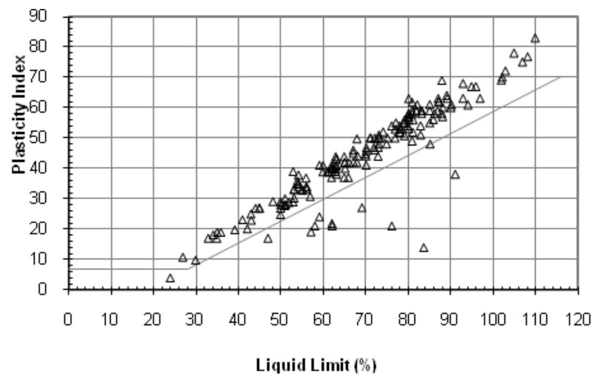

Figure 6: Atterberg limits in Casagrande plasticity chart

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Index tests such as Atterberg limits and the moisture content provide valuable information on the characteristics of a clay. As Balasubramaniam (2010) emphasised, these tests are most valuable when quality undisturbed soft clay samples cannot be retrieved. The results of moisture content, liquid limit, plasticity index and liquidity index are plotted against elevation in Figure 7. The average moisture content appears to be about 60% with the liquid limit slightly above that, indicating possible slight over-consolidation of the soils. The liquidity index is another important parameter which has been plotted on Figure 7 as it provides guidance on how sensitive particular clay is, with higher values greater than 1, indicating higher sensitivity. The liquidity index for the PoB clays was found to be on average less than 1 which indicates the clays may be only moderately sensitive to disturbance. At shallow depth, a more sensitive layer having a liquidity index slightly over 1 was observed but this layer is generally found to be thin.

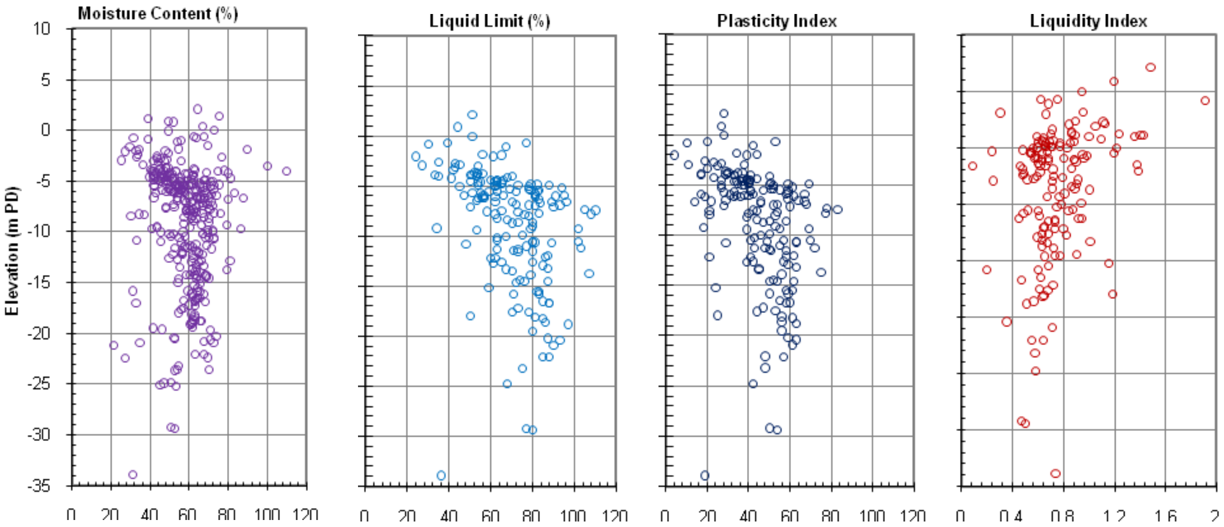

Figure 7 Results of laboratory index tests on PoB clay

<u>**Undrained Shear Strength of PoB Clay**</u>

It is quite common in Australia to rely on insitu tests to assess the undrained shear strength of a clay deposit rather than laboratory tests such as unconsolidated/consolidated undrained triaxial tests or unconfined compression tests. For softer soils there is a strong perception that samples do get disturbed during the field investigation, transportation and/ retrieval for laboratory testing. There is merit in this argument because, in general, most site investigations are conducted not for research but mainly for actual construction projects. The time or the money allocation for more sophisticated testing is therefore limited by the programme or the budget.

Insitu tests within the Holocene deposits have been carried out using the piezocone (cone penetrometer test with pore pressure measurements, CPTu) and/or Electric Friction Cone (EFC) and insitu vane shear equipment. The industry generally accepts that results from vane shear tests are likely to provide a good indication of the undrained shear strength of the clays and therefore has become a reference test for most projects. The piezocone/EFC test is a probing test and therefore a direct shear strength parameter cannot be measured. However, if calibrated with corrected vane shear data, it provides a continuous strength profile at any test location.

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The common method of converting piezocone/EFC data to obtain a shear strength profile is by using the following empirical equation:

$S_u = (q_t - \sigma_v)/N_{kt}$

where q_t = corrected piezocone resistance,

σ_v = overburden pressure and

N_{kt}= cone factor

Cone factor is found to vary between about 12 and 20 for clays in South East Queensland. Assessment of the field test results at the Port site suggests that an average value of 15 is appropriate as the cone factor for the PoB clays.

The undrained shear strength of the PoB clays is found to increase gradually with depth. Figure 8, reproduced from Ameratunga et al (2005), shows the derived shear strength Vs depth along the eastern part of the FPE Seawall, i.e., the eastern boundary of the FPE reclamation area. The shear strength values have been derived using an N_{kt} factor of 15. To obtain a calibration corrected field vane shear test results were used. The linear variation against depth indicates that the undrained shear strength increases at a rate of about 1.5kPa per metre depth.

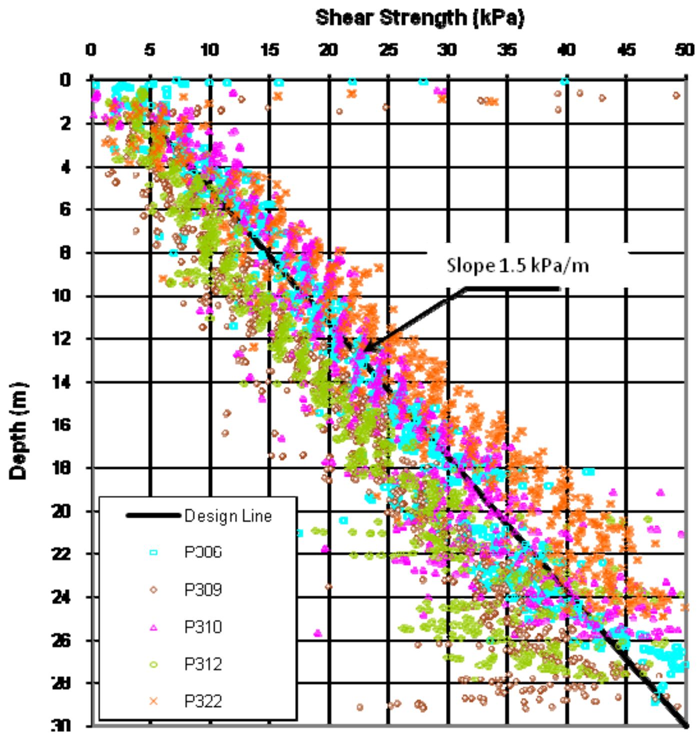

Figure 8 Typical undrained shear strength profile interpreted from a CPTu using a cone factor of 15

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Another important parameter obtained for clays from the vane shear tests is the residual shear strength. The residual shear strength provides guidance on the sensitivity of the clay and the ratio between the peak shear strength, $S_{u(peak)}$, and the residual shear strength, $S_{u(res)}$, is generally known as the sensitivity ratio. The sensitivity ratio values obtained from the tests conducted at the Port are shown in Figure 9 and they indicate that the ratio generally falls between 2 and 4. This confirms the assessments based on liquidity index that the sensitivity of PoB clays can be described as moderate.

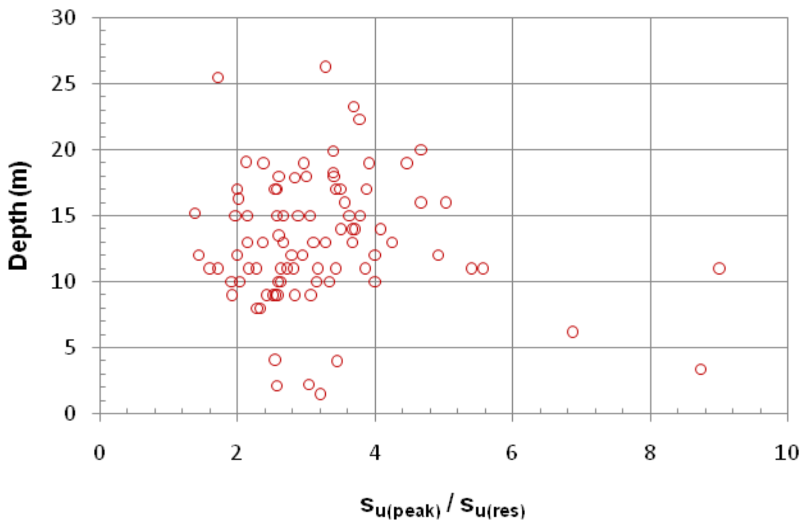

Figure 9 Sensitivity ratio from vane shear test results

<u>Drained Shear Strength of PoB Clay</u>

Although the drained shear strength of a material is of importance in the long term it does not play a major role in the initial stages of construction, specifically where surcharging is the main ground improvement technique adopted. However, the results from the tests conducted for the construction of the FPE Seawall were analysed to assess the long term strength parameters and to understand the behavior of PoB clay characteristics. The results are summarised as q Vs p´ as shown in Figure 10 where

$q = (\sigma_1 - \sigma_2)/2$ $p' = (\sigma_1' + \sigma_2')/2$ and

σ_1 and σ_2 = total vertical and horizontal stress respectively
σ_1' and σ_2' = effective vertical and horizontal stress respectively

After removing the outliers, a best fit line through the origin is shown to give a value of φ'=27.7 degrees. Although a cohesive intercept is evident in some data, considering that the soils are likely to be normally consolidated to slightly overconsolidated, it is prudent to adopt a zero effective cohesion. These values are generally of the same order as for many South East Queensland clayey soils.

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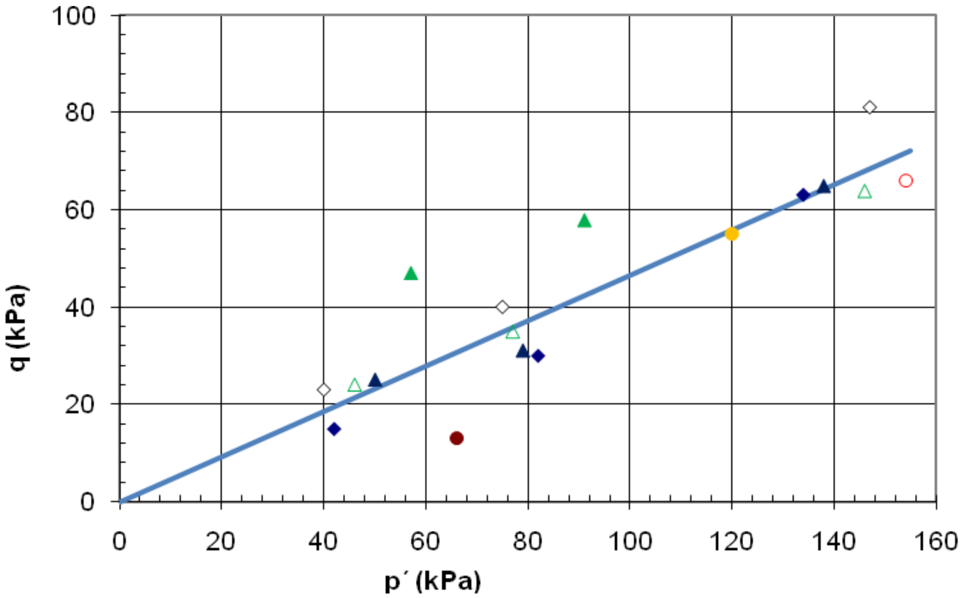

Figure 10 q vs p´ plot for PoB Clay

Consolidation Parameter CR of PoB Clay

On a site where the compressible clay thickness is variable and as high as 30m, the magnitude of settlement expected under development loads is high, in the order of 2m to 3m. Ground improvement by surcharging has to be designed to remove the majority of the expected settlement and thus ensure that the long term residual settlement is within limits of performance expected by the PBC, ranging from 150mm to 250mm over a period of 20 years for loads up to 60kPa. Accurate prediction of the consolidation settlement, both primary and secondary, is therefore of key importance.

The Australian geotechnical fraternity still relies heavily on the Terzaghi 1-D consolidation theory and there was no reason not to adopt the same theory for all the project work at the Port. To assess the magnitude of settlements using the Terzaghi theory, two main design parameters are required, viz, CR and $C_{\alpha\varepsilon}$, which are commonly known as Recompression Ratio and Secondary Compression index respectively.

The importance of these parameters necessitated conducting several consolidation tests on samples collected during the field investigations. The majority of the tests used for this assessment were conducted during the ground improvement trials. However, results of a few tests conducted at the site prior to the current trials, and which were available in summary tables and or summary results sheets, were also used in the database.

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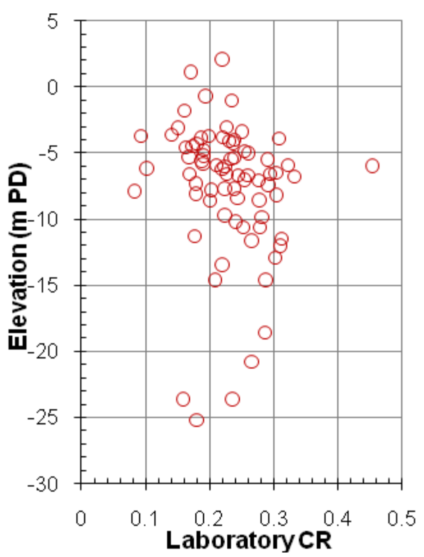
The CR values as obtained from the laboratory e-log p curves for the normally consolidated stress range are plotted against RL in Figure 11(a). The scatter of the data does not appear to be high and CR values generally range between 0.15 and 0.3 with an average value of the order of 0.25. CR can also be obtained from various relationships proposed in published literature linking to physical characteristics such as moisture content and Atterberg limits. Considering that a significant amount of moisture content and liquid limit data is available for the site soils, empirical relationships proposed by the following equations (all relationships taken from Djoenaidi, 1985 who cites the original authors) were used to derive CR values as shown in Figure 11(b) drawn against elevation in PD (Port Datum).

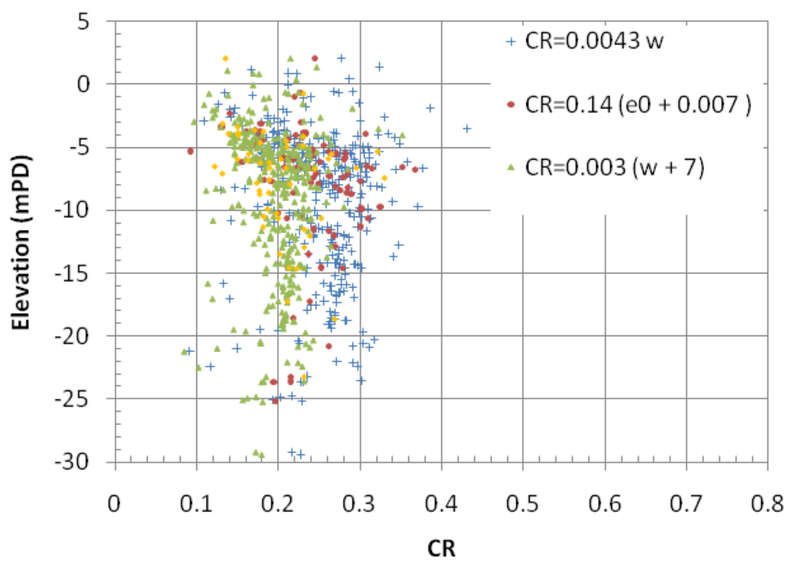
CR = 0.0043 w
CR = 0.14 (e_0 + 0.007)
CR = 0.003 (w + 7)
CR = {0.009 (LL – 10)}/(1 + e_0) (Equation modified from Terzaghi & Peck, 1967 to obtain CR from Compression index)

Where w = natural moisture content
e_0= initial void ratio and LL = Liquid Limit

The range of results appears to confirm the trend shown in Figure 11(a).



(a) Laboratory CR



(b) CR from Correlations

Figure 11 Compression Ratio of PoB Clays

<u>Consolidation Parameter Cαε for PoB Clay</u>

Theoretical prediction of settlement is usually confirmed during construction by instrumentation, at least by using simple settlement plates. However, secondary consolidation is almost never monitored in projects because of the long time period required to collect sufficient data. Therefore, designers have almost no information on past projects to refine any measured parameters from laboratory tests. More attention is therefore paid to laboratory tests and correlations based on laboratory tests.

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At the site, as the compressible clay thickness is large, the secondary consolidation component is a significant amount to be dealt with. Therefore it is important that the surcharge design takes into account the likely secondary consolidation and removes some of the settlement by having a higher surcharge. When the subsoils are subjected to a higher load than the expected development design loads, the clays will become overconsolidated (OC). The amount of over consolidation to be targetted in the design is a difficult question to the designer because of the uncertainties in secondary consolidation behavior. In the past, many researchers have shown that the secondary consolidation expected from an overconsolidated clay is significantly lower than that expected when the same layer is in a normally consolidated (NC) state. Generally, the industry believes that a drop to 1/5 or 1/10 the value of NC state can be expected if a clay layer is surcharged above its normally consolidated state. Several researchers have demonstrated that $C_{\alpha\varepsilon}$ is dependent on the amount of over consolidation achieved, which is generally expressed as OCR (over consolidation ratio). OCR is calculated by dividing the final effective stress under the surcharge load by the effective stress expected under the stresses imposed by the expected design loads. Figure 12 shows available data from several researchers and cited by Wong (2007) plotted as a Creep Ratio of $C_{\alpha\varepsilon\ OC}$/ $C_{\alpha\varepsilon\ NC}$ against OCR. Results available from the PoB site have also been plotted on the same figure, which appear to show that the Creep Ratio drops very quickly with a small increase in OCR. Further testing would be required, especially at low OCR values in the range 1.0 to 1.2, to confirm these findings. These results suggest that, if clays in the field behave in a similar manner to that in the laboratory and surcharging can achieve at least a nominal OCR, the risk of high long term settlement due to secondary consolidation can be reduced significantly.

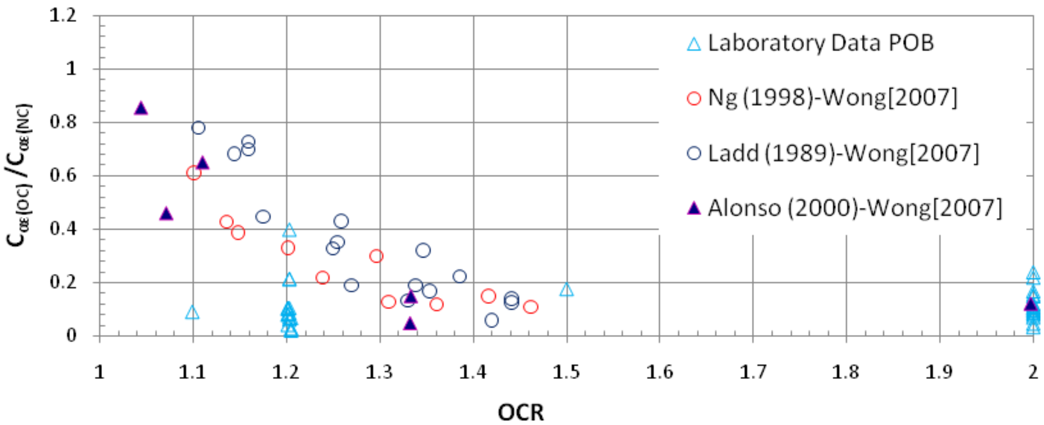

Figure 12 Creep ratio Vs OCR

<u>**Coefficient of Consolidation of PoB Clay**</u>

The settlement rate is governed by the coefficient of consolidation (c_v) and it is one of the most difficult parameters to assess from routine testing. In South East Queensland, generally the laboratory consolidation tests provide values that are too low when compared to back-calculated values from field monitoring. Field values of 5 to 10 times the laboratory values are not uncommon (Lambe and Whitman, 1969). In South East Queensland, more attention is given to values derived from piezocone dissipation tests than from laboratory test results. Generally a dissipation test is conducted in a layer identified from information available at the site and is important to the designer. The best method of identifying a layer is by carrying out a piezocone test at the location of interest which provides a full profile of the subsoil strata. Once the test depths are identified the piezocone is pushed again slightly offset from the original location and dissipation tests are carried out. Carrying out dissipation tests by interrupting a normal probing test is not recommended because layer identification is not possible before the test.

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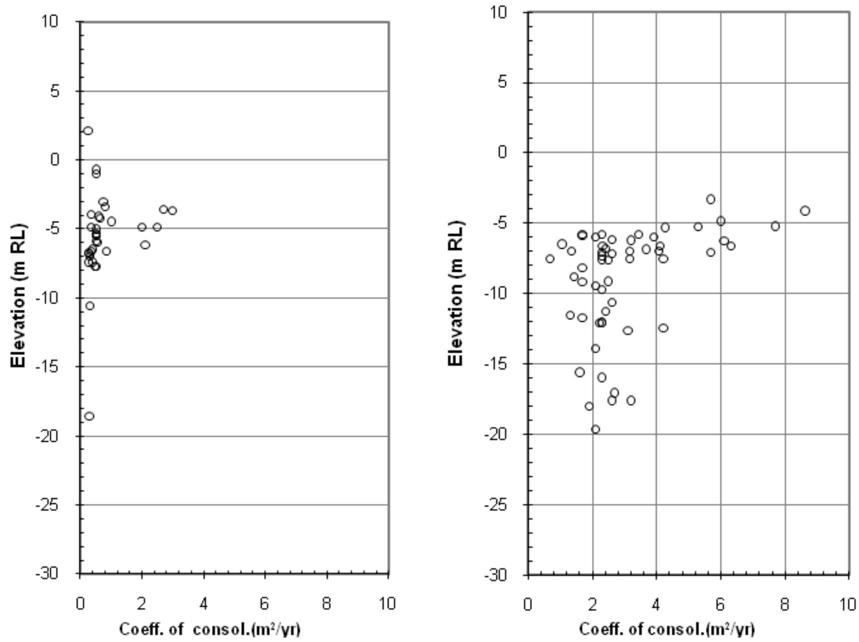
Figure 13(a) shows the results of c_v obtained from laboratory tests on samples of the PoB clays. The results indicate an average value less than $0.5m^2/yr$. The results of the piezocone tests across the same site are shown in Figure 13(b). We have assumed that a piezocone dissipation test provides a horizontal coefficient of consolidation (c_h) rather than a c_v value but this is debatable because of the complex conditions associated with pore pressure dissipation. The piezocone test results in Figure 13(b) show that the insitu value is 4 to 8 times higher than the laboratory values from consolidation tests. Back-calculation of trial results appear to suggest that c_h obtained from piezocone tests is of the right order, assuming a horizontal to vertical ratio (c_h/c_v) of 2. Therefore piezocone tests are considered to offer better assessments of the coefficient of consolidation than the laboratory tests for the PoB clays and this seems to confirm the general trend in South East Queensland.



(a) Laboratory tests (b) Piezocone tests

Figure 13 Coefficient of consolidation

Wick Drain Performance in South East Queensland, Australia

Case histories of wick drains are widely available in geotechnical literature and the reader is referred especially to the works of Brand and Brenner, 1981, Balasubramaniam et al, 1984, Bergado et al, 1996, Indraratna and Chu, 2005. Wick drains and sand drains have been used to improve soft clays in conjunction with preloading/surcharging over many years in Australia and overseas. While there are many case histories presented in conferences and seminars, only success stories are generally available in the public domain because many do not wish to discuss failures and even if they want to, because of commercial and/or legal consequences, they practically cannot.

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Compared to three decades back, more research has been conducted on wick drains and sand drains in the laboratory as well as calibrating field observations in trial embankments and actual projects. The most famous ones are the works associated with Bangkok airports (see Balasubramaniam, 2010) and the Malaysian trial embankments (Poulos et al, 1989; Nakase and Takemura, 1989; Indraratna et al, 1992; Brand and Premchitt, 1989). With more knowledge gained from these projects and research work, improvements on machinery with vibration not currently used for pushing a mandrel, and increased expertise of installers, it is reasonable to accept that the performance of wick drain installations would be better than in the past. Just because the technology is robust, it does not mean the use of wick drains is without risk because the performance of a wick drain still depends on various factors, especially smear, which are directly related to the subsoil characteristics. Hence the issue of wick drain performance still challenges geotechnical professionals, even the leading academics, researchers and practitioners.

In South East Queensland, especially along the coastline, the occurrence of soft soils is widespread and the use of wick drains in conjunction with surcharging has been prevalent. Acceptance of wick drains as a genuine ground settlement acceleration technique by the industry has been slow because of anecdotal evidence, as well as reported cases where wick drains have underperformed or not performed at all.

Robertson (1984) reported a case of preloading in the Oyster Creek vicinity where an estuarine depositional environment existed and the Pacific Highway approach embankments were up to 9m high. The subsurface profile consisted of 3m to 5m of silty sand overlying up to 16m of soft organic clay. The solution adopted was surcharging with wick drains spaced at 1.7m and 1.9m triangular. However, a closer spacing of 0.8m was adopted under the culvert area (three cells of 1800mm pipes) to achieve early completion. Robertson concluded that, overall, wick drains performed well except for the closely spaced wick drains in the culvert area. The more demanding time of 12 months was not satisfied and the target settlement was not achieved until 20 to 22 months after embankment construction. The author ascribed this slower rate of consolidation to most probably the disturbance caused by a closed mandrel 150 x 75 mm driven at 0.8m centres.

Wijeyakulasuriya et al (1999) described the results of trial embankments on soft sensitive clays constructed along the eastern coast, Sunshine Motorway and the Gold Coast. The undrained shear strength of the clays was around 10 to 15kPa, with natural moisture contents generally between 60% and 120%, and a liquidity index ranging from 1.5 to 2.5. These characteristics suggest the clays to be highly sensitive to disturbance. The authors also stated that piezocone dissipation tests were masked by the remoulding of the clay caused by insertion of the cone. At the Sunshine Motorway site the Motorway traversed a swamp underlain by very soft to soft organic marine silty clay, ranging in thickness from 4m to 10m. The trial embankment comprised three 20m sections; Section A had wick drains of 1m spacing, Section B had wick drains with 2m spacing, and the middle Section acted as a Control Section. Comparison of results of settlement plates indicated that wick drains had not accelerated settlements significantly, with the 2m section settling the most. Wijeyakulasuriya et al (1999) concluded that the results suggested that the advantage of closer spacing of the wick drains had probably been almost wiped out by installation disturbance of the wicks in these sensitive deposits. They recommended that if ground improvement techniques such as wick drains were to be considered in sensitive soils, careful consideration should be given to the geotechnical conditions because of the potential for underperformance of the wick drains.

Case History

Ground Improvement in Port of Brisbane (PoB) Clay (cont.)

PBC also carried out a small trial towards the west of the current reclamation site about 10 years back. The trial was conducted in an area located at the Berth 8 Extension, i.e., about 1 km west of the current site. As seen from Figure 14, the site is underlain by about 10m of compressible clay. The trial consisted of wick drain areas of different spacings, one 0.8m diameter sand drain area and a control area. The analysed results (Coffey 2004) suggested that wick drain areas with 1.2m and 1.6m square spacings did not perform at all compared to the control area whereas the use of 0.8m diameter sand drains installed using vibro-replacement techniques appeared to produce an accelerated rate of consolidation at the early stages of loading. A closer examination and analysis of the results (Figure 14) indicates that the settlement rate was significantly quicker for the sand drain area than the control area, but subsequently, the settlement rate per log (time) cycle was practically identical. The settlement separation between the two plots is only about 50mm which could be related to the upper layers interbedded with sand. The fact that wick drains did not appear to have performed (in fact slightly lower rate of consolidation compared to control area) suggested, among other things, that the effect of smearing may have been significant and suggested that steps must be taken in the installation method to reduce disturbance if wick drains are to be considered. It is now generally accepted that vibratory methods should be avoided in the installation of wick drains, especially where the soils are sensitive.

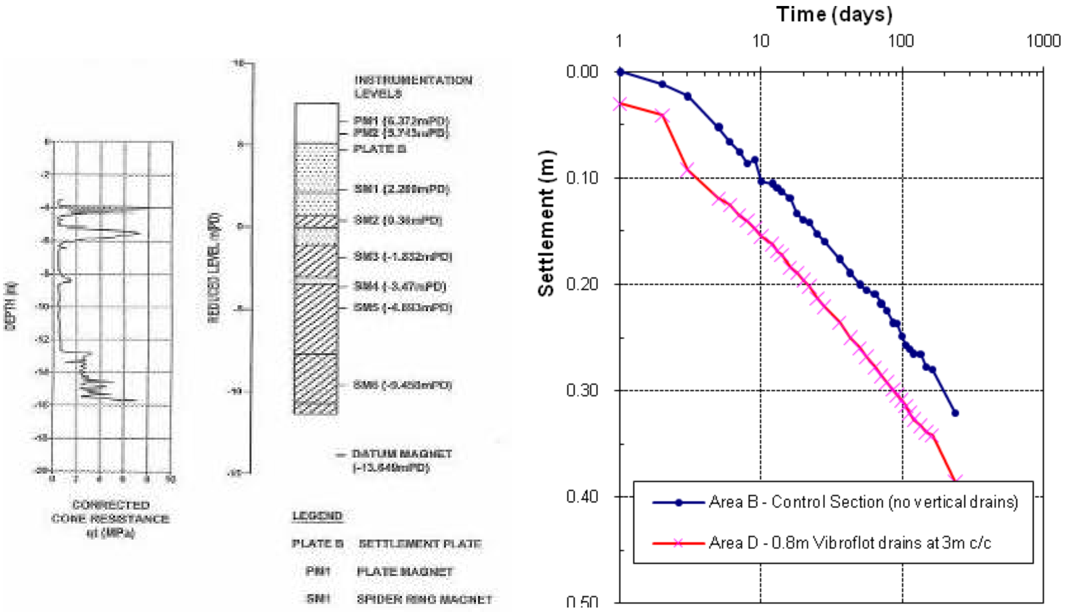


Figure 14 Trial embankment at the Port of Brisbane in 2000

<u>**Port of Brisbane – Ground Improvement Trials**</u>

PBC and Coffey carried out in-house studies to assess the best course of action to move forward and develop the land available for development. As previously discussed, the clay thickness significantly increases towards the northeast in addition to the increased thickness of dredged materials that could be held in the paddocks created and contained by the seawall and internal bunds. Therefore, conventional techniques such as surcharging without wick drains would not be feasible if the land is to be developed over the next 25 years or so along commercial timelines.

Case History

Ground Improvement in Port of Brisbane (PoB) Clay (cont.)

Internal studies carried out by Coffey and the PBC into local and overseas practices of treatment of soft soil indicated that, apart from conventional surcharging, the two main groupings of techniques available to treat and improve the reclamation spoil and in-situ soils were:

1. Improve, reinforce or stabilise the soils to reduce settlements and to improve shear strength and stiffness
2. Surcharging in conjunction with accelerating settlements.

The suite of techniques falling under group 1 includes stone columns, piling of the ground, deep soil mixing and mass stabilisation. Most of these techniques needed large scale equipment to deal with deep soils on site and therefore the mobilisation costs for a small trial were excessive, as the equipment had to be brought either from Asia or Europe. The methods falling into group 2 comprise vertical drains and sand drains. Vacuum consolidation, which is a process whereby a vacuum pressure is applied to an area already installed with wick drains to potentially increase their effectiveness, also falls into this category.

The most economical consolidation technique identified was to use prefabricated vertical drains, commonly known as 'wick drains', in combination with surcharging, to accelerate the consolidation process. However, PBC was concerned about wick drain underperformance. As it ultimately needs to treat a reclaimed area in excess of 235ha, PBC needed some certainty in relation to the performance and timelines for consolidation and therefore decided to conduct large scale wick drain and vacuum trials to assess the effectiveness and performance of the various techniques. This would enable optimised designs to be used in future treatment of the large areas of the reclamation site.

PBC invited Expressions of Interest (EOI) from Australian and overseas specialist ground improvement contractors for the design, supply, installation and monitoring of applicable specialist ground improvement systems. Although available information on subsoil profiles and preliminary design parameters were provided in the EOI documentation, no recommendations were given on the type of technique to be used and it was left to the potential tenderers to propose what was best for the site. The EOI did permit any solution and specifically requested suggestions for improvement techniques for the edges bounding the Moreton Bay Marine Park, because it was critical to avoid disturbance to the Marine Park and nearby internationally recognised Ramsar site for migratory wader birds.

PBC and Coffey set up selection criteria to assess the eight tenders received based on solution, design, price, quality systems and environmental considerations. All proposals were assessed by scoring them against the selection criteria. This resulted in the short listing of three preferred proposals. These three submissions could not be substantially separated in terms of the selection criteria, with all three offering wick drain solutions. Vacuum consolidation was also proposed, mainly for the edge boundary conditions.
PBC decided that there was considerable merit in trialling all three contractors rather than further reducing the number of trials. The three successful tenderers were Austress Menard (Menard), Boskalis and Van Oord. To optimise the designs, a range of drains were proposed for trialling, with different spacings, patterns and filter fabrics. Menard's vacuum consolidation system was more applicable where slope stability was an issue, such as the treatment of the edges of sites, as the vacuum pressure applied reduces the amount of surcharge loading required, thus improving edge stability. The applied vacuum pressure was about 80kPa, which is equivalent to about 4m of sand surcharge fill. Therefore it was selected for a trial site which posed significant stability risks on two sides as it is bounded by the Moreton Bay Marine Park and the Port's purpose built Migratory Wader Bird Roost, which has a perimeter moat around it to protect the birds from predators. A unique feature of the vacuum system adopted for this latter site was the 15m deep soil-bentonite cut-off wall which was required to isolate the vacuum area because of the deep, permeable, sand lenses and layers within the subsurface profile. Appropriate testing was carried out to assess the depth of sand lenses and properties of the overburden prior to excavation of the trench. This is the first occasion that vacuum consolidation with a cut-off was ever used in Australia.

Case History

Ground Improvement in Port of Brisbane (PoB) Clay (cont.)

The areas set aside for the trials were expanded and rationalised to allow different types of trials to be conducted. The trial types varied from the wick drain type, filter type, spacing and configuration. Boskalis also trialled its BeauDrain vacuum system in a small area. Coffey, in consultation with the trialists, carried out detailed in-situ and laboratory investigation for all areas and provided the results to the trialists.

Figure 15 shows the trial areas and Table 2 provides the main information related to those areas. Three specialist reviewers, Profs Harry Poulos from Coffey Sydney office, Prof A S Balasubramaniam from the Griffith University and Prof Buddhima Indraratna from the University of Wollongong were appointed by the PBC to act independently as Specialist Reviewers of trial performance.

Extensive instrumentation was installed by Coffey and the PBC in consultation with the trialists. Instruments were regularly monitored during and after construction and the results were made available via an on-line data management system 'Insite', specifically modified by the supplier to cater to the needs of the trials. The Insite system enabled monitoring results to be viewed by the relevant trialists and others such as Coffey and Specialist Reviewers. Interim reviews of the results and back-analyses were performed by the trialists, checked by Coffey and the performance independently assessed by the Specialist Reviewers.

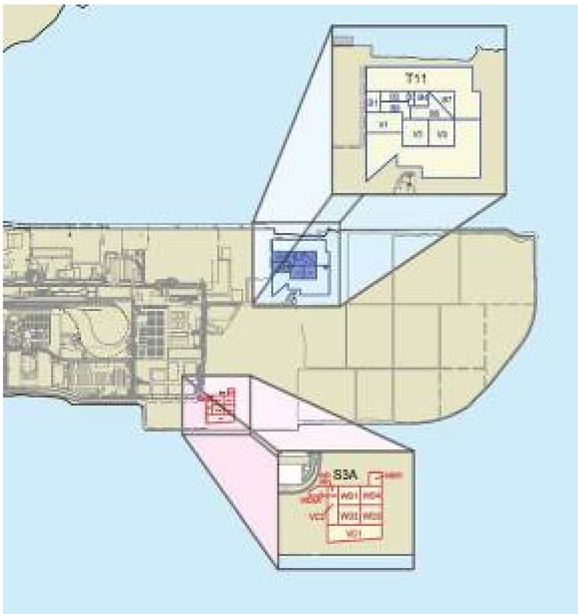


Figure 15 - Trial areas

Table 2 Details of the Trials

	Boskalis	Van Oord	Austress Menard (wick)	Austress Menard (vacuum+wick)
Design load	60kPa	60kPa	15kPa	15kPa
Trial size	30,000m^2	30,000m^2	27,210m^2	16,380m^2
Wick types	MD7007, MD88H, MD88HD	MD7007, MD88H	MD88, FD767, MCD34	MCD34
Wick grid	Triangular	Triangular	Square	Square
Wick spacing	1.25m	1m - 1.4m	1.1m - 1.3m	1.2m
Design Preload period	12 months	6-12 months	7 - 12 months	8 months
Surcharge height	5 - 9m	7.5 - 10m	3 - 8m	2.5m

Case History

Ground Improvement in Port of Brisbane (PoB) Clay (cont.)

Technical Objectives of the Trials

The purpose of the project was to acquire knowledge on effective techniques to accelerate the process of reclaimed land development at the Port of Brisbane and to optimise the treatment solutions for the varying ground conditions encountered across the Future Port Expansion Precinct:

- achieve design loads ranging from 15kPa up to 60kPa;
- reach a post-development residual settlement of 150 mm to 250mm maximum over 20 years under service loadings;
- complete consolidation within 12 to 18 months maximum;
- identify contributions to overall settlement made by the various compressible layers of dredged mud, upper Holocene and lower Holocene;
- develop a range of optimal PVD solutions applicable to the variable conditions that occur at the site;
- identify the most efficient and cost effective PVDs best suited to the varying ground conditions;
- optimise the installation procedures and relative spacing of the PVDs to ensure that excessive surcharging is not required;
- identify designs that promote low risk edge stability of sites to be surcharged;
- establish future strategies, options and trials for the deeper and more complex areas of the reclamation area; and
- develop long term strategies involving the consolidation of dredged mud, to limit the use of sand for fill and surcharging.

Results and Back Analyses

The monitoring results were available to the contractors, Specialist Reviewers and Coffey. Each party used different techniques to back-calculate design parameters as well as predict future residual settlements. Coffey adopted the following methodology:

1. Set up a geotechnical model and filling history for each assessment location.
2. Derive time settlement curves using its in-house software package CAOS for all monitoring locations, based on design parameters adopted after the sensitivity analysis for the trial area in Paddock S3A. This prediction takes into account the actual fill thickness, thickness of compressible units, fill construction history etc. In areas where extensometers have been installed, assess the compression of individual layers to avoid the uncertainties associated with the multi-layered system and thick soft soil issue.
3. Assess the curve fitting and adjust the fill history and loading where appropriate. The analyses have shown that the settlement predictions are mostly affected by fill history and load applied.
4. Review results using the Asaoka method (Asaoka, 1978) as an additional tool. The corresponding pore pressure data from piezometer monitoring was reviewed for consistency.

The inhouse software package CAOS (Consolidation analysis of Soft soils) is a FORTRAN program that carries out a finite difference numerical solution of the one-dimensional equation of consolidation (Poulos, 2002). It can consider a multi-layered soil profile subjected to a series of loading sequences which may include both constant and time-depending loading. In the analyses, the following have been taken into consideration:

- Buoyancy effect of the fill below the groundwater level due to settlement;
- Design limit on post-construction settlement;

Case History

Ground Improvement in Port of Brisbane (PoB) Clay (cont.)

- Long-term creep effects.
- Effect of smear due to mandrel insertion

The output from the program includes the surface settlement, the distribution of settlement and excess pore pressure with depth at specified times, and the soil state (i.e. normally- or over-consolidated state) at various depths in the profile.

The theory developed by Hansbo (1981) has been utilised in CAOS. The effects of smear during the insertion of mandrel have been also incorporated in the programme. The effect of smear can be controlled by inputting the radius of smear zone and ratio of permeability of undisturbed soil to that of remoulded soil. The time of installation of the vertical drains can also be specified.

The program allows the user to select one of two options for computing the settlements due to creep:

1. A method based on Bjerrum's (1967) concept of instantaneous and delayed consolidation;
2. A method in which creep is initiated when the percentage dissipation of excess pore pressure reaches a value specified by the user. This approach was adopted.

Furthermore the post-construction settlements have been based on the concept that the rate of creep reducing with increasing over-consolidation ratio (OCR) (Mesri et al., 1994) as described in Wong (2006). An exponential law for the reduction of $C_{\alpha\varepsilon}$ with OCR has been adopted and given below.

Creep Ratio = $\dfrac{C_{\alpha\varepsilon(o/c)}}{C_{\alpha\varepsilon(n/c)}} = \dfrac{1-m}{e^{(OCR-1)n}} + m$ where "m" and "n" are constants.

Constant "m" represents the minimum values of Creep Ratio when the OCR is large. From Mesri (1991) "m" will be equivalent to the ratio between recompression and compression index (C_r/C_c). The magnitude of "n" controls the rate of reduction of the Creep ratio with OCR. The adopted values for the project were *m = 0.1 and n = 6.* If the Creep Ratio is plotted against OCR based on these selected values, the resulting curve is shown in Figure 16 with laboratory Creep Ratio values reproduced from Figure 12 in the background. The adopted equation is considered reasonable and conservative.

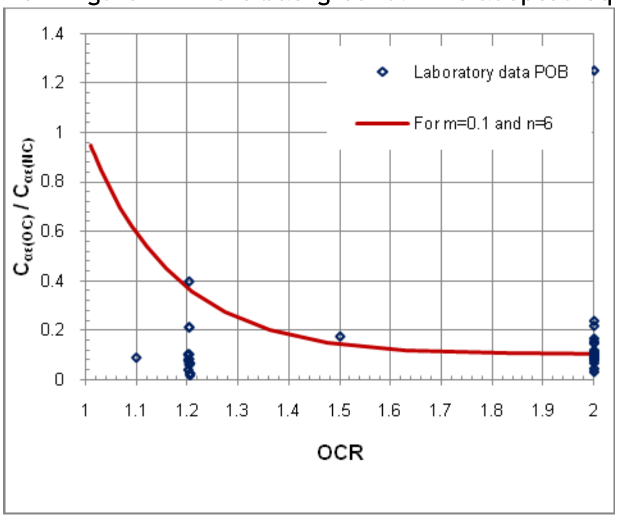


Figure 16 Creep ratio Vs OCR

Case History

Ground Improvement in Port of Brisbane (PoB) Clay (cont.)

Due to logistical difficulties not associated with the contractors, it was not possible to begin all trials at the same time. Menard vacuum and wick drain trial area was the first to commence and generally was not affected by a temporary shortage of fill for surcharging operations as for the other trials. This assisted the Menard trial to be completed according to the programme envisaged. This also provided all parties, especially the three Specialist Reviewers, with the opportunity to analyse the data in a more detailed manner. Therefore the Menard vacuum trial was selected in this paper to highlight some of the more interesting lessons learnt through the trial process before summarising other findings.

<u>**Menard Vacuum and Wick Drain Trial**</u>

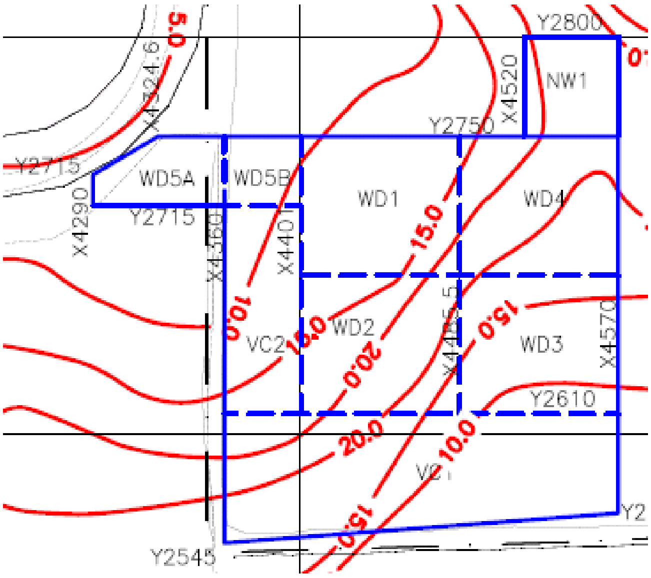
The Menard vacuum trial was located on the southern side of the reclamation in Paddock S3A as shown on the plan in Figure 15. An aerial view of the trial area taken when the membrane has been installed is shown in Figure 17.

Figure 17 Menard Trial-Vacuum area with membrane placed and adjacent wick drain area

The subsurface profile across the Menard trial area can also be described as recent fill overlying Holocene deposits which in turn overlie the stiffer Pleistocene deposits and the basalt bedrock. The recent fill consists of the dredged mud across the paddock which is variably thick, overlain by a sand capping mainly constructed to facilitate the movement of construction vehicles. As the dredged mud is very soft, not more than 5kPa in undrained shear strength, it was necessary to have a minimum thickness of sand of 2m with localised thickening where necessary. During placement of the sand capping in March 2006, the dredged mud in the reclamation paddocks displaced significantly (mud waving), giving rise to a variable distribution of the mud and sand cap across the trial site. The sand layer also acted as the drainage layer for the wick drain operations. The thickness of the Holocene layer is also variable as demonstrated by the clay thickness contours shown in Figure 18. As previously discussed, the upper Holocene layer consists of mainly sand with inter-layered soft clays and is therefore highly permeable with the lower layer, comprising very soft to firm clay, being generally normally to slightly consolidated and therefore highly compressible. A summary of soil strata is given in Table 3.

Case History

Ground Improvement in Port of Brisbane (PoB) Clay (cont.)



VC1 and VC2 - Vacuum + wick drain areas

WD1 to WD5 - Wick drain areas

NW1 - Control area

Figure 18 - Lower Holocene clay thickness (in metres) contours across the Menard trial area

Table 3 Geological units

Unit	Description	Thickness Range
Recent - Sand capping	White sand	0.9m~5.0m
Recent - Dredged materials	Dredged mud, marine and dune sands with layers of silt and clay.	2.5m~7.6m
Holocene	Marine clay, silt and sand.	8.8m ~24.4m
Pleistocene	Generally stiff clay, sand and gravel	
Tertiary	Bedrock	

The Menard Vacuum Consolidation method consists of installing vertical and horizontal vacuum transmission pipes under an airtight membrane and sucking the air below the membrane thus imposing a partial atmospheric pressure on the soil, creating an accelerated isotropic consolidation; which can be combined with a conventional surcharge placed on top of the membrane, in order to achieve the required degree of consolidation (See Figure 19) (Berthier et al, 2009). A critical element of the method as applied to the Port site was the construction of a deep soil bentonite cut-off wall, to isolate the site from the surroundings because of the frequent sand lenses and layers observed in the upper Holocene layers. This seal is crucial to the efficient functioning of the vacuum method. It is in fact the first occasion the vacuum consolidation with a cut-off wall was ever adopted in Australia. The cut-off wall depth was 15m, the deepest ever employed with the Menard Vacuum method.

As detailed in Berthier et al (2009), a vacuum pressure of 80kPa was maintained throughout the trial period as measured in piezometers embedded in selected wick drains at deep levels, and in vacuum gauges placed under the membrane. This vacuum is equivalent to about 4m of fill surcharge in a routine surcharging scenario. Therefore only an additional surcharge of up to 2.5 m was required for the vacuum area of the trials.

Case History

Ground Improvement in Port of Brisbane (PoB) Clay (cont.)

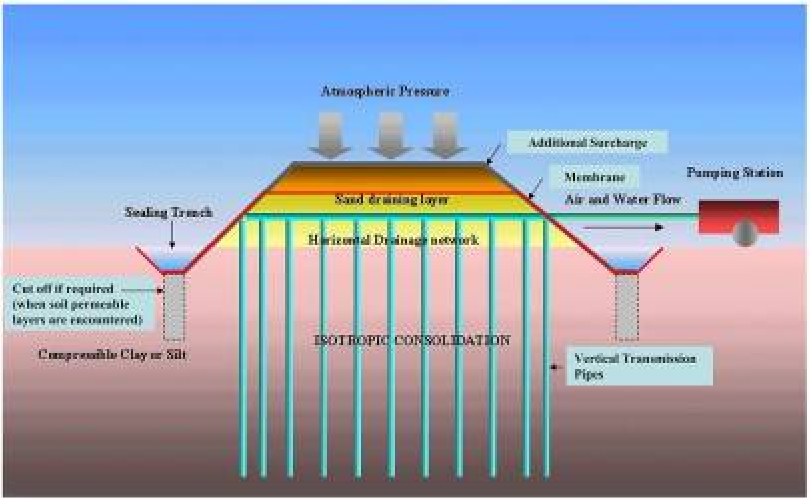


Figure 19 Menard vacuum consolidation method (after Berthier et al, 2009)

As shown in Figure 17, the non-vacuum wick drain trial sites of Menard were adjacent to the vacuum areas and consisted of one Mebradrain type, one Flexidrain type, and a circular drain 34mm diameter. Spacings ranging from 1.1m to 1.3m square were adopted for the Menard wick drain trials.

As Balasubramaniam et al (2010) discuss, Asaoka method (1978) and hyperbolic models are the most widely used methods by engineers, with Asaoka method most popular within the Australian industry. These methods are most useful in estimating the percentage of consolidation under the surcharge. It has its limitations mainly because it is applicable only to a single layer of homogeneous clay. This in itself is not a major issue if one of the layers is significantly thick relative to the other and only the latter stages of consolidation are of concern. Thus, it is generally accepted that the Asaoka method would be applicable only in the latter part of the consolidation curve, viz., after at least 60% of consolidation is completed. However, if it is required to interpret the behavior from an early stage, especially when time is limited and any remedial measures need to be implemented earlier, other methods need to be adopted such as the CAOS software previously discussed. Even such software would not eliminate the possibility of deviations in the latter part of the curve, but they allow sensitivity studies to be conducted and perhaps provide the engineer useful guidance necessary to take intervention measures early. In the current trials, both Asaoka and curve fitting techniques were adopted. While Coffey used the CAOS software, other trialists and the Specialist reviewers used their own software (e.g. Indraratna, 2010) and/or commercially available software such as MSettle.

Figure 20 shows a typical settlement curve measured in the field and a calibrated curve in the vacuum area (Berthier et al, 2009). Calibrations were carried out by trial and error but using knowledge gained on parameters from laboratory tests and/or empirical knowledge to select which parameters were likely to be variable. In complex sites such as that at the Port of Brisbane, where the variables are numerous because of the layering of the soils, there is the potential that several solutions could provide similar answers.

The comparison of vacuum and non-vacuum provided interesting observations as illustrated by the average Degree of Consolidation (DOC) plots in Figure 21 (Indraratna, 2010a). It is quite clear that the rate of settlement under vacuum (VC1 and VC2 areas) during the initial stages is greater than in the non-vacuum area. Although the two curves should merge at the end of primary consolidation, accelerated rates of settlement in the early part provide a significant advantage especially where stability is a key issue of concern.

Case History

Ground Improvement in Port of Brisbane (PoB) Clay (cont.)

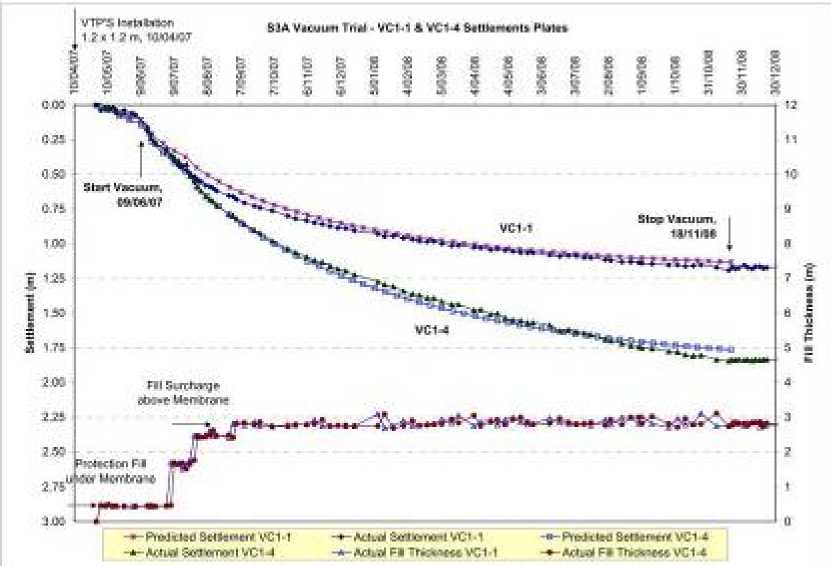


Figure 20 – Typical settlement curve in the Menard vacuum area

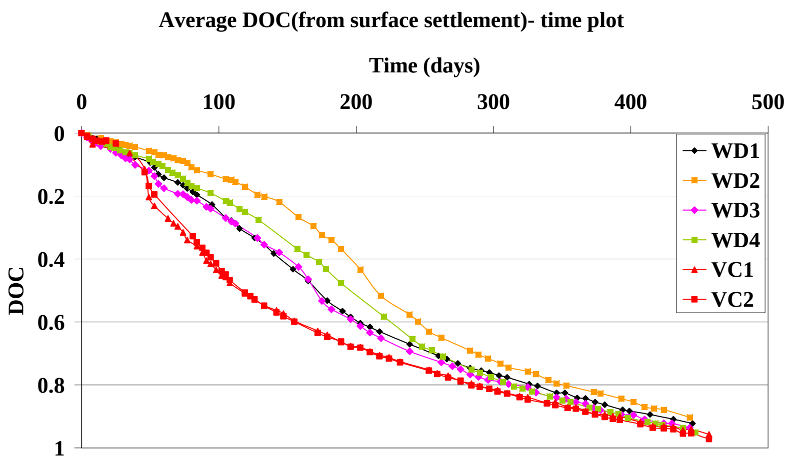


Figure 21 – Time settlement curves for vacuum and non-vacuum areas

The stability during construction was assessed by monitoring the lateral movement by inclinometers as well as measuring the excess pore pressure behavior using piezometers. The primary reason a vacuum solution was adopted at the edges was the high risk of instability under the surcharging operations. The lateral movements from the inclinometers were studied in detail by the Specialist Reviewers to assess whether the vacuum system provided a significant advantage. Figure 22 (Indraratna, 2010a) shows the lateral displacements at two inclinometers, one at the vacuum area and the other at the non-vacuum area. Indraratna has removed the difference in the applied stress at the two locations by "normalising" using the effective stress. The reduction in lateral movement is clearly shown in the figure confirming the advantage the vacuum system offers. The benefits of the vacuum in controlling the lateral movement is further discussed by Indraratna (2010b) in his 2009 E H Davis Memorial lecture.

Case History

Ground Improvement in Port of Brisbane (PoB) Clay (cont.)

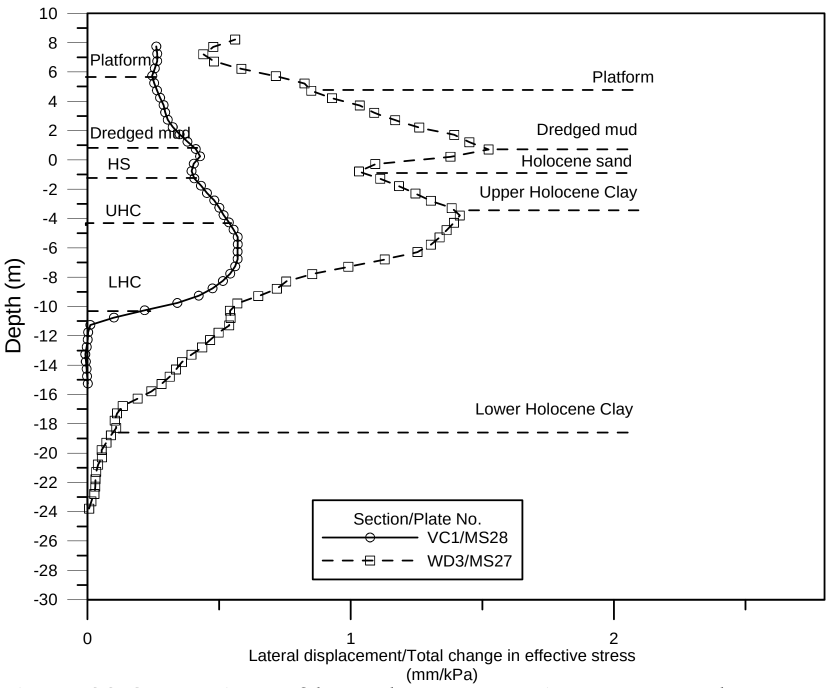


Figure 22 Comparison of lateral movement in vacuum and non-vacuum areas

Learnings from the Trials

Some of the learnings from all the trials are summarised below.

- All flat and round drain trials worked satisfactory. The following drains and filters were trialled successfully: MD88*, MD88H, MD88HD, MCD34, FD767, MD7007 and MD88 drains. None of the drain cores indicated unusual behavior such as breaking, kinking etc although significant strains were experienced by the drains. MD88 showed a slightly better performance in the degree of consolidation achieved but the difference of 5 to 10% is insufficient to make a concrete conclusion.
- All the drains trialled had filters; there was no marked difference when a larger filter pore size was used, indicating that the pore size is not critical, at least for the conditions at the Port (the filter pore sizes of 75µm, 80µm and 150 µm were trialled); It was suggested that a pore size close to 75µm be used in the future as it indicated a slightly better performance;
- Spacings from 1m to 1.4m were trialled and a definite reduction in performance was observed for the closer 1m triangular spacing. This was assessed to be due to the greater effect of smear when the drains were closely spaced. The advantage of closer spacing seems to be negated by the increased effect of smear.
- In one area of the Port, wick drains were installed but the surcharging was delayed for 18 months. The drains continuously worked under the load from the sand capping but only low volumes of water were discharged under this surcharge. The performance of these drains under the full surcharge suggests that leaving the wick drains in the ground over such a long period after installation, does not diminish the performance of the drain providing the ground is settling i.e. as water is discharging through the drains due to consolidation caused by loading such as sand capping. If there is no water discharge, there may be possibilities of blocking of pores caused by clay particles.

Case History

Ground Improvement in Port of Brisbane (PoB) Clay (cont.)

- Vacuum consolidation, although relatively expensive, is a better performing system compared to wicks and surcharge alone. The cut-off wall provided a very effective seal. There is a much faster strength gain in the early stages because of quicker dissipation of pore water pressure, compared to wicks only, providing a significant advantage in terms of lateral stability.
- Vacuum consolidation clearly shows accelerated settlement rates in the initial critical stages of surcharge performance compared to non-vacuum trials. This is most helpful in relation to stability.
- Vacuum consolidation also shows that the lateral movement is clearly reduced by the application of the vacuum and the less surcharge heights involved. This emphasises its advantage in controlling stability.
- The round drain worked well in the vacuum consolidation areas, but there was no advantage observed for other areas.

Concluding Comments

A series of ground improvement trials using wick drains and vacuum consolidation was carried out at the Port of Brisbane because of concerns of underperformance of wick drains in previous projects in South East Queensland. The purpose of the project was to ascertain whether wick drains would perform efficiently under the conditions at the Port and to acquire knowledge on effective techniques to accelerate the process of reclaimed land development and to optimise the treatment solutions for the varying ground conditions encountered across the Future Port Expansion Precinct.

Although field and laboratory tests of samples undertaken previously provided valuable assessment of the design parameters, they cannot replicate the full-scale field testing of the in-situ soil conditions undertaken in these trials.

The trials have provided PBC with confidence that wick drains can be adopted to treat the deep soft clays at the Port of Brisbane. Excessive smearing can reduce the performance of wick drains significantly, although this may not be a major issue at the Port site, probably because the sensitivity of PoB clays is not as high as in some of the soft clays found elsewhere in South East Queensland. Nevertheless, the trials showed that too close a spacing could lead to some reduction in performance, due the effects of smearing.

The Menard vacuum consolidation trials clearly demonstrated that vacuum consolidation application for PoB clays does work effectively and that excellent results can be obtained once cut-off walls are adopted to isolate the sand lenses in the upper horizon (upper Holocene deposits) from the wider area.

The trials emphasise the need to conduct full scale trials in major projects whenever possible. They are better than any other tests, insitu or laboratory, and provide valuable information to the designer which ultimately will assist the client in reducing his risk and the cost of completion of a project.

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Case History

Ground Improvement in Port of Brisbane (PoB) Clay (cont.)

Port of Brisbane in 2009 (Courtesy of Port of Brisbane Corporation)

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