Geotechnical Aspects of Container Port Development

C. F. Leung
National University of Singapore

Y. D. Zhang, B. K. Liao & P. W. Lam,
Surbana Jurong Consultants Pte Ltd

ABSTRACT: This article presents the key geotechnical challenges related to gravity caisson quay walls and soil improvement works on reclaimed land for container port development in Singapore. The caisson foundation issues encountered in Jurong Formation are limestone cavities, presence of sandwich soft soils and potential soil softening beneath sandkey trench. Ground improvement using surcharge preloading and PVD is often implemented to accelerate the primary settlement, reduce the residual settlement and enhance the ground stability for the reclaimed land using various sources of filling materials.

1 HISTORICAL DEVELOPMENT OF CONTAINER PORT WHARF FRONT STRUCTURES

When Sir Stamford Raffles first established a trading post in Singapore to further British influence in the region in 1819, the port activities then started along the Singapore River. After its independence, Singapore opened its first container port in Tanjong Pagar in 1972. The terminal has three container berths and received the first container vessel from Rotterdam in June 1972 with a cargo of 300 containers (Wong & Chng, 2016). To meet the increasing trading volume over the years, the city terminals including the Tanjong Pagar Terminal, Keppel Terminal and Brani Terminal were progressively developed from 1960s to 1990s.

With rapid growth in the number and size of container vessels, a big leap in the provision of container port facilities was necessary to complement the existing terminals. Pasir Panjang Terminal was then strategically developed as the new container port since 1990s. The development of Pasir Panjang Terminal took place in four stages. Phase 1 of the terminal kick-started in 1993, with both Phases 1 and 2 completed by 2010 while Phases 3 and 4 were completed by 2015.

Singapore's long term plan is to move all its container ports to Tuas South, see Figure 1. The Tuas Terminal is being developed in four phases over 30 years. Phase 1 of the terminal is scheduled to be completed by the early 2020s, able to handle about 20 million twenty-foot equivalent units (TEUs) of cargoes annually. The entire Tuas Terminal could eventually handle 65 million TEUs of cargoes annually, nearly double what Singapore handled in 2016 (Kaur, 2017).

The design and capacity of the ports change significantly over the years due to rapid increase in the cargo carrying capacity and dimensions of container vessels (Wong & Chng, 2016). The development of the ports would have to take into account deeper draft of container vessels and adequate wharf front capacity for loading, unloading and storage of increasing cargo volumes brought by larger vessels. In addition, to accommodate these larger container ships, quay cranes of a greater height and outreach span are needed to move containers from the ship to the shore, see Figure 2.

Pile decks were commonly deployed as the wharf front structures in Singapore till early 1990s. Since Pasir Panjang Terminal Phase 1, gravity caissons have been adopted as the wharf front structures instead (Leung & Shen, 2008; Leung, 2014). Gravity caisson wharf front structures provides more flexibility to the layout of the port, requires much lesser maintenance and is comparable in cost when the water depth of port is deep and there is economies of scale for caisson production. Geotechnically, it presents an advantage over pile deck in area such as Pasir Panjang, where infilled limestone cavities of a few meters are present, as the large caisson base spreads the heavy load allowing it to span over the cavities.

2 GRAVITY CAISSON WHARF FRONT STRUCTURES

Gravity caisson is a gravity-type retaining structure prefabricated on land or on a large floating barge. The height and width of the caissons would be designed accordingly to suit the actual site conditions and requirements (Loh et al., 2010). The caisson wharf front structure system (Figure 3) comprises foundation, caisson structure, caisson infill, reclamation fill and subsequent wharf decking and accessories.

The caisson structure and foundation shall be designed to withstand various loading cases comprising loads during construction when the caisson is transported from land to the sea and when it is being infilled and pre-loaded, as well as the loads under operation stage due to port equipment and vessel berthing and mooring. The loadings arising from construction to port operations will be transmitted to the foundation through the caisson via the underlying compacted rock mound. The various geotechnical ultimate limit states related to the caisson shown in Figure 4 shall be checked and designed for (Esteban & Rey, 2009). Besides the ultimate limit states, the serviceability limit state of caisson system during port operation has to be ensured to meet the settlement requirement, in particular, the differential settlement between the two quay crane legs as the front leg of the quay crane rests on the caisson while the rear leg is supported on piles.

As such, it is of great importance to ensure the caisson rests on competent ground. Any soft material immediately below the caisson founding level shall be either treated by soil improvement works such as installation of sand compaction piles (Leung & Shen, 2008) or dredged away and replaced by sand to form a sandkey foundation. The sandkey will be subsequently compacted to increase the stiffness to reduce the caisson movements.

Furthermore, all caissons after installation are preloaded to an appropriate surcharge load not less than the dead loads above the top of the caissons and operational live loads acting directly on and behind the caissons to minimize non-recoverable settlement from the caisson foundation and the founding strata. The backfill behind the caissons is also subject to surcharge preloading to accelerate the soil consolidation. These measures enhance the overall stability of the caisson structures and reduce the residual settlement of the ground during port operation.

When the gravity caisson wharf structures were used for the first time in Singapore, centrifuge model study was also carried out to examine caisson movements and effectiveness of caisson preloading for Pasir Panjang Terminal Phase 1 (Leung et al., 1997). In addition, the monitoring results of caisson preloading/unloading/reloading in Pasir Panjang were reported (Tan et al., 1999; Khoo et al., 2013).

Some geotechnical issues associated with caisson foundations during design and construction of gravity caissons are discussed below. They are cavities in Jurong Formation, presence of soft soils sandwiched between hard soils and potential softening of residual soil beneath sandkey trench.

2.1 Limestone with cavities

The Jurong Formation covers the south, west and southwest parts of Singapore (DSTA, 2009). It is composed of a variety of sharply folded sedimentary rocks including conglomerate, sandstone, shale, mudstone, limestone and dolomite. It was deposited during late Triassic to early or mid-Jurassic periods. The Formation has been folded and faulted due to past tectonic movements.

Depending on the degree of weathering, the Jurong formation comprises residual soil, completely weathered, highly weathered and fresh rock. It is generally encountered at the surface or beneath the Kallang formation in some areas. Typically its upper portion has been weathered down heavily to residual soils or completely weathered rocks.

Being part of the Jurong Formation, the location and top level of limestone are highly variable with pinnacles, cavities, and deep solution channels. Cavities are generally identified during borehole drilling with sudden water loss or sinking of drill rods, or during bored pile installation with loss of concrete and escape of grouting fluids when back filling boreholes (Pakianathan & Jeyatharan, 2005).

Jeyatharan & Pakianathan (2003) reported the difficulties experienced in Pasir Panjang during bored piling works in 1996. Many cavities were encountered at the site till 60 m deep and extensive jet grouting being carried out prior to piling. At each pier location, minimum one borehole was made to a depth of at least five times the pile diameter beneath the pile toe, or the last cavity, whichever was deeper. Despite this, concrete losses during pile concreting still took place.

Limestone with cavities was also detected through boreholes at caisson location in Pasir Panjang. Additional boreholes with cross borehole seismic tomography were carried out to determine the size and extent of the cavities. Figure 5 shows a schematic diagram for the marine tomography. The extent of the cavities can be identified from the boreholes and the various velocity distributions of the seismic tomography in between the boreholes. Using three dimensional modelling images, the geologists assessed that the biggest cavity is a funnel-shaped cavity infilled with soil and rock fragments. Engineering assessment was then carried out to evaluate the impact of the infilled cavity on the stability and movement of the caisson foundation. Various cases of cavity properties had been modelled in the assessment and the results indicated that the presence of this cavity beneath the caisson had negligible effect on the stability and movements of the caisson.

The necessity to treat a cavity largely depends on its depth, thickness of limestone above the cavity and extent of the cavity and the properties of infilled materials inside the cavity. For the case described above, the cavity is located at a relatively large depth and its size is limited in plan. The additional stresses from the quay wall structures are able to be transferred through the arching effect of the surrounding limestone without affecting the stability of the cavity and the caisson foundation. On the other hand, when the cavity exists at a relatively shallow depth beneath the caisson, with thin arching limestone above the cavity and the size of the cavity is extensive, the cavity needs to be treated. Hence, the impact of the cavity to the integrity of caisson structure should be carefully investigated with adequate soil investigation and robust engineering assessment. The tomography test is a useful tool which can be used to determine the extent of cavities between bored holes. Proper ground treatment method should be adopted if the impact of the cavities is deemed significantly affecting the safety and serviceability of the quay walls and container stacking yard.

2.2 Presence of weaker soils between hard soils

Owing to high variability of the Jurong Formation as a result of the inter-bedding and folding of parent rocks, sandwich soft soil layers (Figure 6) could exist in between two hard layers, which raises potential concerns on the stability and settlement of caissons sitting above it.

In one such case, soil investigation revealed the presence of weaker soil layer beneath the targeted sandkey founding layer at one location of Pasir Panjang. Extensive engineering assessment predicted possible excessive settlement if the weak soil layer was left untreated. It was decided that jet grouting should be carried out to increase the strength and stiffness of the sandwiched weaker soils.

Trial grouting was performed on site to establish the grouting parameters used to treat the weaker soils. Unconfined compression tests were conducted from core samples collected at the top, middle and bottom along the circumference of the grouted piles to verify the quality of the jet grouts. The minimum total core recovery of each recovered core shall be 85% and cored samples shall meet the target unconfined compressive strength of 600 kPa. Besides laboratory tests, in-situ pressuremeter tests at 2 m interval were performed to establish the stiffness of the treated column after 28 days of jet grouting. Figure 7 shows the schematic of the test. The target desired stiffness for the treated column was 60 MPa.

The actual effective treatment zone of each grouted column will vary depending on the component of the soils (e.g. granular soil or clayey/silty soil). Analysis based on as-built grouted columns was also carried out to evaluate the movements of caisson under service loading.

2.3 Softening of soils beneath sandkey

The foundation for caisson structures typically comprises compacted sandkey underlying the compacted rock mound. As mentioned earlier, dredging of incompetent seabed materials is necessary. During sandkey dredging, as a result of soil swelling and temporary release of overburden pressure, the soils beneath the sandkey could soften before sand is filled in the trench. It was noted that the reduction in strength and the depth of the softening varied.. Such difference in the degree and depth of the softening may be caused by two different soil softening mechanisms: swelling and soil bond destruction (i.e. slaking).

In the case of consolidation related softening, the reduction in strength might simply be due to dissipation of negative excess pore pressure and change in vertical stress and water content due to sandkey trenching (Picarelli et al., 2006). In this respect, the frictional component of the soil strength is proportional to the effective stress acting on the soil. For over-consolidated soils, this relationship remains approximately correct when considering the undrained strength. It is expected that the soil would regain its strength under the vertical loading pressure of the caisson structure of over 300 kPa. It may not be possible to prevent such soil strength reduction as a consequence of sandkey trenching. Since this is a time and stress dependent phenomenon, one should place the sand fill in the sandkey trench as soon as possible and practicable.

On the other hand, slaking may be due to the combined effect of pressure relief after sandkey trenching and water intrusion causing soil bond destructuring. Slaking occurs when bonding between soil particles are not strong enough to withstand internal stresses caused by volumetric and shearing strains due to unloading together with rapid water intake (Leroueil & Vaughan, 1990).

The slaking tests were carried out for samples taken from Jurong Formation. Figure 8 presents the results of typical slaking tests. It was established that the slaking class of Jurong Formation covered the full range between least slaking Class 0 to highest slaking Class 4. The observations are consistent with that reported earlier (Leung & Radhakrishnan, 1990).

The reduction in strength and stiffness of the caisson foundation due to slaking and softening can be significant, which may affect serviceability and stability of the caisson structure if left untreated. The design of the sand key foundation may need to take into account possible location and extent of the slaking soil. The softening soil layer may be tackled by simply dredging it away if the thickness is not excessive or soil improvement.

One of the ways to soil improve this weaken layer is by dynamic replacement with rock tamping (Hamidi et al, 2010). The process was carried out by installing compacted rock columns into the softened clay to form a composite soil-column mass as shown in Figure 9. A 1.8m thick granite rock fill was placed over the soft clay layer to be tamped into the softened soil to form the desired columns. The rock columns were designed to be 2 m in diameter, in a 4.5 m grid with a replacement ratio of 15%. Post tamping pressuremeter tests were then carried out to verify the improved strength and stiffness as specified in the design.

3 LAND RECLAMATION AND SOIL IMPROVEMENT

3.1 Land Reclamation

The rapid economic development over the last few decades in Singapore has led to a continuous demand for land to be used for housing, transportation, commercial and industrial needs. As land is scarce, a solution is to create land by land reclamation. Since the first reclamation works began in 1822, Singapore's land area has expanded by almost 25 percent from 578 to 719 square km (Lim, 2017). Most of the airport and container terminals in Singapore are situated on reclaimed land.

As reclamation sites entered considerably deeper waters, large amount of fill materials were required. On the other hand, Singapore's economy is rapidly growing with many deep excavations and tunneling activities, generating large quantity of excavated soils. In addition, Singapore needs to dredge its seabed regularly to maintain the depth of its navigation channels, which generates dredged soils. It is certainly attractive then, to use the abundance of construction waste soils and dredged materials for land reclamation as dumping grounds in Singapore is very limited. The Building Construction Authority (BCA, 2008) of Singapore published a guideline to promote sustainable construction and encouraged the use of excavated soils for land reclamation. In fact, all the present ongoing land reclamation projects specify that they must receive these otherwise "unwanted" soils which are classified as 'soft clay' and 'good earth' (BCA, 2008). The use of these soils as reclamation fill provides a sustainable and cost effective solution not only to protect the environment but also to meet the large demand of the fill material for land reclamation projects.

3.2 Ground improvement

However, the use of the excavated and dredged soils for land reclamation poses both short-term construction and long-term operation challenges such as stability issues during construction and residual settlement during port operation. Ground improvement is often necessary to accelerate the settlement of the fill and the in-situ soils ensuring ground stability.

For a container port, a stringent long-term residual settlement requirement during port operation is typically kept to 40mm to ensure minimal disruption of port operation and the various container handling equipment on the road and thoroughfares. Such stringent settlement requirement is particularly crucial for an automated port where Automatic Guided Vehicles (AGV) and other unmanned port equipment will be used.

The massive reuse of excavated and dredged soils, which comprises mainly of clay and silt with high water content, as reclamation fill will be expected to have very significant settlement. Ground improvement is necessary not only to eliminate all primary consolidation settlement due to reclamation filling but also to minimize the long-term total residual settlement.

In order to minimize the residual settlement during the service life of the ports, the soft and compressible soils will be treated with a surcharge higher than the service loads to ensure that the compressible soils are over-consolidated after preloading and would not go beyond the re-compression line during port operation stage. The consolidation of these soft materials under surcharge is normally accelerated by the insertion of Prefabricated Vertical Drains (PVD) to reduce drainage paths. Depending on the functional requirement, the specified degree of consolidation, based on the surcharge loading, needs to be achieved within the thickness of the compressible soil to ultimately meet the residual settlement required.

The consolidation period under surcharge typically varies from 3 to 12 months depending on the construction schedule. The PVD design is to adopt an appropriate PVD spacing and termination depth taking into consideration of soil conditions and time constraint. The PVD shall be anchored into the stiff layer of Jurong Formation, of a minimum desired pre-consolidation pressure, so as to minimize the consolidation settlement of the compressible soil layers above it.

3.3 Characteristics of dredged materials

As mentioned in Section 3.1, the reclamation fill material may come from various sources including land excavation and marine dredging. For the construction of container port, dredging works using grab dredgers with various grab sizes are usually deployed to deepen the basins and fairway and to form the sandkey trenches. The materials dredged from seabed are termed as dredged material in general and comprise soil lumps of various sizes and soil types from Kallang Formation and Jurong Formation. The dredged materials will be loaded onto split hopper barges, box barges or flat top barges for subsequent filling. Reclaimer barges and hydraulic filing method may also be used to reclaim the land to a higher level above water where normal barges are not accessible.

The dredged materials are expected to be highly variable and non-homogeneous as a result of variability in the material sources, disturbance during dredging, filling and preloading process. Karthikeyan et al (2004) reported the results of site investigation after about 12 years of deposit of large dredged clay lumps in Punggol Timor Island. It was found that the strength and deformation characteristics of the lumpy fill layer to be highly scattered and variable with normally consolidated and over consolidated zones. The over consolidated zone is a part of the original clay lump which has been over consolidated in the past. On the other hand, normally consolidated zones were found from areas around the initial inter-lump voids filled by disintegrated small clay lumps and the softened surface areas of large clay lumps. It was concluded that the ultimate state of the land reclaimed using clay lumps is highly variable and distinctively different from that using clay slurry, where homogenous properties are expected.

3.4 Pilot Tests for soil improvement works

For projects with large scale soil improvement works for reclaimed land such as container port, it usually requires pilot tests to determine the type of PVDs, the suitable spacing, the appropriate installation depth and the surcharge duration. The pilot tests can also serve as a guide to ascertain the suitability of the proposed installation rig and mandrel types. In a pilot test carried out in Pasir Panjang for soil improvement works by PVD, the test site was sub-divided into two clusters of CT-1 and CT-2, each with a plan area of 50m by 30m. A PVD spacing of 1.5m square grid was adopted for cluster CT-1 with 1.0m square grid spacing adopted for CT-2.

The soil profile within the pilot test areas can be broadly classified into three distinct layers consisting 4m to 8m thick of sand fill, 10m to 15m thick of dredged material and the in-situ soils beneath the original seabed. Surface settlement markers and deep settlement plates were installed to monitor the surface settlement and subsurface settlement of individual soil layers. After placement of full surcharge for about 4 months, the surface settlement for CT-1 ranged from 1.22m to 1.37m. As for CT-2, the surface settlement ranged from 1.13m to 1.43m.

The final ground settlements were estimated using the Hyperbolic method (Tan et al 1991, Tan, 1994) or Asaoka method (Asaoka, 1978) based on instrumentation monitoring results. From measured field settlement and predicted ultimate settlement, the average degree of consolidation can be estimated. It was established that the hyperbolic and Asaoka methods could estimate the final ground settlement with high accuracy if the recorded settlement data has already reached at least 60%

degrees of soil consolidation. The settlement within this range will be used to predict the settlement by initialization of both time and settlement. The predicted final settlement can be obtained by

$$S_f = S_0 + S_{fp} \tag{1}$$

where S_f is the total predicted settlement under current surcharge; S_0 is the measured settlement at the initialisation time; S_{fp} is the predicted final settlement relative to the time of initialization using the data with degree of consolidation more than 60% data. The degree of soil consolidation, U_t, can be determined as

$$U_t = \frac{S_t}{S_0 + S_{fp}} \tag{2}$$

where S_t is the measured settlement at the time t.

Figure 10 shows a predicted final settlement of 1.539m for the surface settlement plate SP-1 for CT-1 using the hyperbolic method. The same procedure was also performed for the surface settlement plates to estimate the achieved degree of consolidation. Considering that a higher surcharge has been applied as compared to the design load of 180kPa, the required minimum degree of consolidation has been achieved within the pilot test area. Removal of the surcharge can then be carried out.

The effectiveness of the soil improvement works after removal of surcharge may be examined by comparing undrained soil strength, moisture content and soil pre-consolidation pressure before and after ground improvement. Figure 11 shows the comparison between the pre-SIW and post-SIW undrained shear strength profile from a typical bored hole within the reclaimed land. It is evident the post-SIW undrained shear strength is higher than the pre-SIW undrained shear strength. This finding was further verified by comparing the Pre-SIW and Post-SIW CPT results. Figure 12 shows a comparison of pre-SIW and post-SIW pre-consolidation pressure from a typical bore hole. No noticeable change between the pre-SIW and post-SIW data was observed. The reason could be that fill materials used for reclamation came from various sources and highly variable as mentioned in Section 3.3. Furthermore, due to large settlement of the soft ground during preloading stage, it would be difficult to obtain representative samples from of the same soil layer from pre-SIW and post-SIW bored holes for a proper comparison.

The above findings are similar to the results observed from the Changi reclamation projects. In the earlier land reclamation project involving Changi Airport in 1970s and Changi East in 1990s, it was reported that (Vijiaratnam et al., 1982, Chu et al., 2009) only the undrained shear strength of soft clay showed a significant increase while the comparisons of pre and post soil pre-consolidation pressure and moisture content were inconclusive. Hence the appreciation in undrained shear strength provides the most direct indication on the effectiveness of a soil improvement scheme.

3.5 Residual settlement

The settlement analysis usually involves two stages. The first stage is to carry out consolidation analysis to determine the degree of soil consolidation and settlement at the end of the desired preloading duration taking into consideration the spacing and depth of PVD. The consolidation settlement and degree of consolidation can be measured and verified by extensive site instrumentation monitoring. The second stage is to predict the long term residual settlement over the service life of the port.

The residual settlement, $S_{residual}$, after surcharge comprises the consolidation and secondary compression including the settlement of the deep stiff soil layers where PVDs cannot be installed. It can be calculated as follows:

$$S_{residual} = S_{rec} + S_{sc} = \frac{C_r}{(1+e_p)}\log\left(\frac{\sigma'_{vp} + \Delta\sigma'_v}{\sigma'_{vp}}\right)H + \frac{C_\alpha}{1+e_p}\log\left(\frac{t}{t_p}\right)H \tag{3}$$

where S_{rec} consolidation settlement under recompression region
S_{sc} secondary compression settlement
H thickness of compression layer
C_a coefficient of secondary compression
e_p void ratio at end of primary consolidation
t design period for calculation of secondary settlement
t_p time at the end of primary consolidation
c_r recompression index
σ'_{vp} effective stress after removal of surcharge
$\Delta\sigma'_v$ service load

Mesri & Castro (1987) proposed that the coefficient of secondary compression, Cα, is proportional to the compression index Cc. They concluded that the volume changes during primary consolidation and secondary compression essentially follow similar mechanism. It was also found that Cα /Cc lies within a narrow range of 0.02 to 0.1 for the soils shown in Table 1. For most inorganic clays, Cα/Cc = 0.04 ± 0.01 while for granular soils, Cα/Cc = 0.02 ± 0.01. As such, the secondary compression can be predicted.

Table 1. Typical values of Ca/Cc for soils (after Terzaghi et al., 1996)

Material	Ca/Cc
Granular soil including rock fill	0.002±0.01
Shale and mudstone	0.003±0.01
Inorganic clays and silts	0.004±0.01
Organic clays and silts	0.005±0.01
Peat and muskeg	0.006±0.01

3.6 Secondary compression of clay

After the soft ground has been treated with PVD, the secondary compression may still exceed the desired magnitude in certain locations. Surcharge preloading can be used to reduce the post-construction secondary settlement of the ground to a lower and acceptable magnitude (Mesri & Feng, 1991). The effective surcharge ratio, R'_s, is defined as

$$R_S' = \frac{\sigma_{vs}'}{\sigma_{vf}'} - 1 = OCR - 1 \tag{4}$$

where σ'_{vs} is the maximum effective vertical stress achieved before the removal of surcharge, σ'_{vf} is the final permanent effective stress after the removal of surcharge, and OCR is the over consolidation ratio.

Mesri & Feng (1991) and Tergazhi et al. (1996) observed that post-surcharge secondary compression appears after primary and secondary rebound t_l, see Figure 13. An approach based on laboratory test results was proposed to estimate the reduced secondary compression after surcharge. The equation for secondary compression can hence be updated as

$$S_{sc} = \frac{C_\alpha''}{1+e_p} \log\left(\frac{t}{t_l}\right) H \tag{5}$$

The reduced post surcharge secant coefficient of secondary compression Ca" (refer to Figure 13 for definition) depends on both effective surcharge ratio, Rs' (=OCR-1) and the design time relative to the time for reappearance of secondary compression t/t_l as shown in Figure 14. This clearly shows the effectiveness of surcharge in reducing secondary compression. It is noted that Ca'' is reduced significantly when Rs' is larger than 0.4 and Ca'' also becomes negligible when Rs' is bigger than 1. The magnitude of surcharge can be determined based on the amount of secondary compression to be reduced to meet the serviceability requirements.

The value of t_l shown in Figure 13 is determined from the empirical correlation between t_l/t_{pr}, and R' for soft clay and silt shown in Figure 15. This figure shows that the time to the appearance of post surcharge secondary compression increases with surcharging effort. The duration of primary rebound t_{pr} depends on the rebound characteristics of the soil as well as on the permeability and drainage boundary conditions.

3.7 Secondary compression of sand

In certain areas of the port, the top several meters of the reclaimed sand serves as the capping layer providing a stable platform to install PVD and competent subgrade for the port pavement. These granular coarse grain soils are also placed immediately behind the caissons as the backfill to reduce the lateral pressure acting the caissons hence increasing the overall stability of the caissons. The approach using reduced secant Ca'' after preloading as mentioned earlier was developed for soft clays and silts. Its applicability to granular soils, such as the capping sand is subjected to verification and requires more test results to support the usage. Therefore, the constant Ca/Cc concept could be adopted to estimate the secondary compression of sand even though the reclamation sand may also be subject to surcharge.

Figure 16 shows the oedometer test results of Ca and Cc at loose and dense states for 11 sand samples from a reclamation site. The Ca/Cc ratio is generally in line with the reported values of 0.01 to 0.003 for granular materials shown in Table 1. Interestingly, the Ca/Cc ratio of dense sand is slightly larger than that of loose sand. Mesri & Vardhanabhuti (2009) reported that the test results of Cc for three groups of sands and established that the compression index Cc varies and typically increases with vertical effective stress less than about 10 MPa, see Figure 17. This trend was also observed from the results of the 11 sand samples.

It is noted that the reported Ca from laboratory tests was obtained at the final stage of loading of 1600 kPa while Cc was obtained from the stress range of 800 kPa to 1600 kPa with an average pressure of 1200 kPa. Adopting the concept of constant Ca/Cc, the Ca over the practical stress range at various depths can be derived. For example, for the purpose of analysis, Cc under stress range 100-200 kPa (average 150 kPa) can be conservatively used for the capping sand at shallow depths while the Cc under stress range 200-400 kPa (average 300 kPa) can be used for the sand below.

4 CONCLUDING REMARKS

This paper presents the key geotechnical challenges related to gravity caisson quay walls and soil improvement works on reclaimed land for container port development in Singapore. Gravity caissons should rest on competent in-situ soil foundation or compacted sandkey foundation to meet the desired design requirements under ultimate and serviceability limit states. The caisson foundation issues encountered in Jurong Formation are limestone cavities, presence of sandwich soft soils due to folding, and potential soil softening beneath sandkey trench. The necessity for treatment depends on their impacts on the caisson stability and settlement. Additional soil investigations are necessary to ascertain the location and extent of the problematic foundation to facilitate detailed assessments. Extensive instrumentation monitoring and interpretations are required to ensure the caissons and their foundations behave as predicted.

The container terminals in Singapore were mostly built on reclaimed lands using various sources of fill materials. The presence of soft soils within the reclaimed land posed both short and long term geotechnical challenges. Ground improvement using surcharge preloading and PVD is often implemented to accelerate the primary settlement, reduce the residual settlement and enhance the ground stability. The secondary compression of the soft soils may contribute the most to the residual settlement during port operation stage. Effective surcharge above the final permanent and service loading is required to reduce the secondary compression. The magnitude of surcharge can be determined based on the amount of secondary compression is to be reduced. Using the concept of constant Ca/Cc and the fact that Cc increasing with effective stress, the Ca over the practical stress range at various depths can be obtained.

ACKNOWLEGEMENTS

The authors would like to thank the Maritime and Port Authority of Singapore (MPA), who is the Government agency developing the container ports in Singapore, for their adoption of innovation, unwavering support for engineering excellence and sustainable reclamation techniques. Appreciation also goes out to all Contractors and Advisors who had been involved in the port development in Singapore through the years. The views expressed in this article are entirely those of the authors and do not represent the views of the Authority and other organisations that were involved in the port development.

REFERENCES

Asaoka, A. 1978. Observational Procedure of settlement prediction, Soils and Foundations, Vol. 18, No. 4: 87-101.

Building Construction Authority (BCA) 2008. Sustainable Construction: A Guide on the Use of Recycled Materials.

Chu, J., Bo, M.W., and Arulrajah, A. 2009. Soil improvement works for an offshore land reclamation. Geotech. Engr. 162(1): 21-32.

DSTA, 2009. Geology of Singapore, 2nd Edition.

Esteban, L.F. & Rey, R.V.D. 2009. Reinforced concrete caissons for port structures in Spain. Marine Engineering 162: 73 – 81. DoI: 10.1680/maen.2009.162.2.73.

Hamidi, B., Yee, K., Varaksin, S., Nikraz, H. & Wong, L.T. 2010. Ground Improvement in Deep Waters Using Dynamic Replacement. Proc. 20th International Offshore and Polar Engineering Conference. Beijing. 20-26 June 2010, 848-853.

Jeyatharan, K., Lee K.W., Pakianathan L.J. & Seh C.P. 2003. Limestones of the Jurong Formation. Proc. Underground Singapore 2003 and Workshop Updating the Engineering Geology of Singapore, 27-29 November 2003, 372-387.

Karthikeyan, M., Dasari, G.R. & Tan T.S. 2004. In situ characterization of land reclaimed using big clay lumps. Canadian Geotechnical Journal, 2004, 41(2): 242-256

Kaur, K. 2017. Full steam ahead for new Tuas mega port. The Straits Times, 24 April 2017, Retrieved on 10 September 2017 from: http://www.straitstimes.com/singapore/full-steam-ahead-for-new-tuas-mega-port/.

Khoo, E., Leong, T.P., Low, C.K. Lam, P.W., Liao, B.K. & Leung, C.F. 2013. Settlement of gravity caisson wharf structure. Proc. 13th Southeast Asian Geotechnical Conf., Singapore.

Leroueil, S. & Vaughan, P. R. 1990. The general and congruent effects of structure in natural soils and weak rocks. Géotechnique 40(3): 467-488.

Leung, C.F. & Radhakrishnan, R. 1990. Geotechnical properties of weathered sedimentary rocks. Geotech. Engr. 21(1): 29-48.

Leung, C.F., Lee, F.H. & Khoo, E. 1997. Behavior of gravity caisson on sand. J. Geotech Engr., J. Geo. & Geoenv. Egr. 123(3): 187-196.

Leung, C.F. & Shen R.F. 2008. Performance of gravity caisson on sand compaction piles. Canadian Geotechnical Journal 45: 393 – 407.

Leung, C.F. 2014. Historical development of container port wharf front structures in Singapore. Proc. Int. Conf. Ports for Container Ships of Future Generations, Hamburg:

199-210.
Lim, T.S. 2017. Land from sand: Singapore's reclamation story. Retrieved on 16 September 2017 from: http://www.nlb.gov.sg/biblioasia/2017/04/04/land-from-sand-singapores-reclamation-story/
Loh, Y.H., Seah K.H., Loh C.K., Liao B.K., Lim S.K. & Loh C.K. 2010. Massive caisson quay wall of Pasir Panjang Terminal Phases 3 & 4, Conference on Structural Marvels, 13-14 December 2010, Singapore
Mesri, G. & Castro, A. 1987. $C\alpha/Cc$ Concept and K_0 during Secondary Compression, J. Geotech. Eng. 113(3), 230-247.
Mesri, G. & Feng, T.W. 1991. Surcharging to reduce secondary settlements. Proc. International Conference on Geotechnical Engineering for Coastal Development, 1, Yokohama: 359-364.
Mesri, G. & Vardhanabhuti, B. 2009. "Compression of Granular Materials", Can. Geotech. J., 46, pp. 369-392.
Pakianathan, L.J. & Jeyatharan, K. 2005. Limestone Rocks of the Jurong Formation Engineering Experiences, Proc. Underground Singapore.
Picarelli, L., Urciuoli, G., Mandolini, A. & Ramondini M. 2006. Softening and instability of natural slopes in highly fissured plastic clay shales. Natural Hazards and Earth System Science 6(4): 529-539.
Tan, A., Wong, K.Y., Lay, K.H. & Leung, C.F. 1999. Performance of Gravity Caissons. Proc. 5th Int. Symp. On Field Measurements in Geomechanics, Singapore, 297-302.
Tan, S. A., 1994, Hyperbolic method for settlements in clays with vertical drains, Canadian Geotechnical Journal, 1994, 31(1): 125-131
Tan, T.S. Inoue, T. & Lee, S.L. 1991, Hyperbolic method for consolidation analysis, Journal of Geotechnical Engineering, Vol. 117, Issue 11
Vijiaratnam, A., Tan, S.B., Ng, P.F.H. and Choa, V. 1982. Reclamation and soil improvement works for Changi Airport. Proc. Intl. Symp. on Airport Planning and Development, Singapore: 88-105.
Wong, K.Y. & Chng, W.P.M. 2016. Singapore: A Port City – the Case of Port Development and its Impact on the City. Proc. 15th World Conference Cities and Ports "Crossovers", Rotterdam 5-7 October 2016
Terzaghi, K., Peck, R.B. & Mesri, G. 1996. Soil Mechanics in Engineering Practice, Third Edition

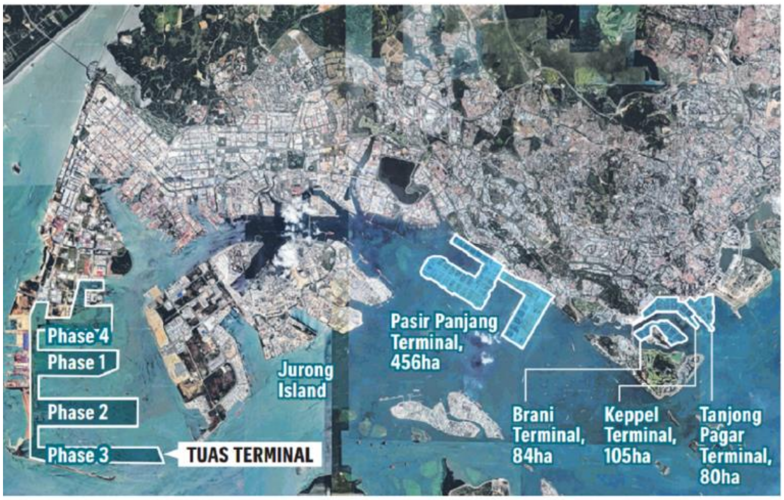


Figure 1. Container port terminals of Singapore (Source: http://www.straitstimes.com/singapore/full-steam-ahead-for-new-tuas-mega-port)

	City Terminals*	PPT Phase 1/2	PPT Phase 3/4
Berth Length	270 m	340 m	400 m
Area per berth	9 ha	14 ha	16 ha
Max depth	13.5 m	16.0 m	18.0 m
Quay Cranes	34 m height 13 rows outreach	46 m height 23 rows outreach	52 m height 24 rows outreach

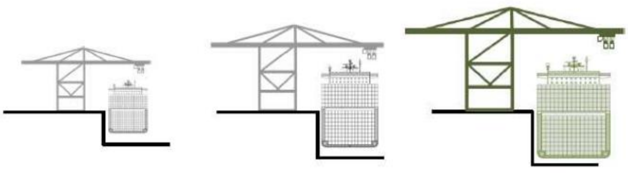


Figure 2. Development of port facilities in Singapore (after Wong & Chng, 2016)

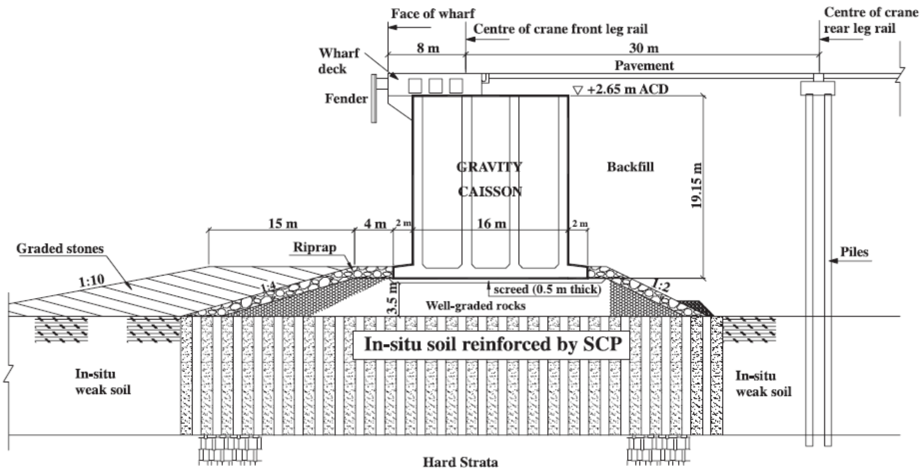


Figure 3. Typical section of gravity caisson wharf founding on sand compaction piles (after Leung & Shen, 2008)

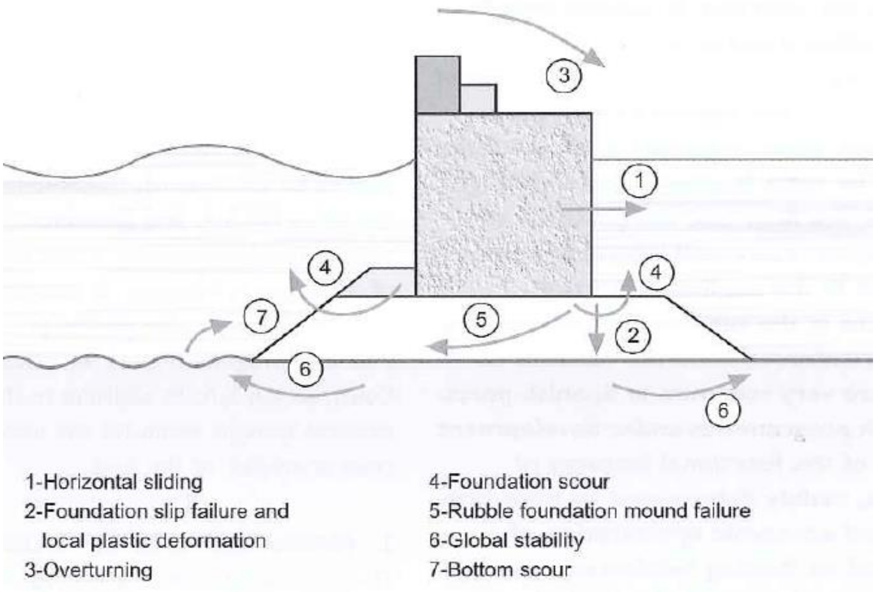


Figure 4. Geotechnical ultimate limit states for gravity caissons (after Esteban & Rey, 2009)

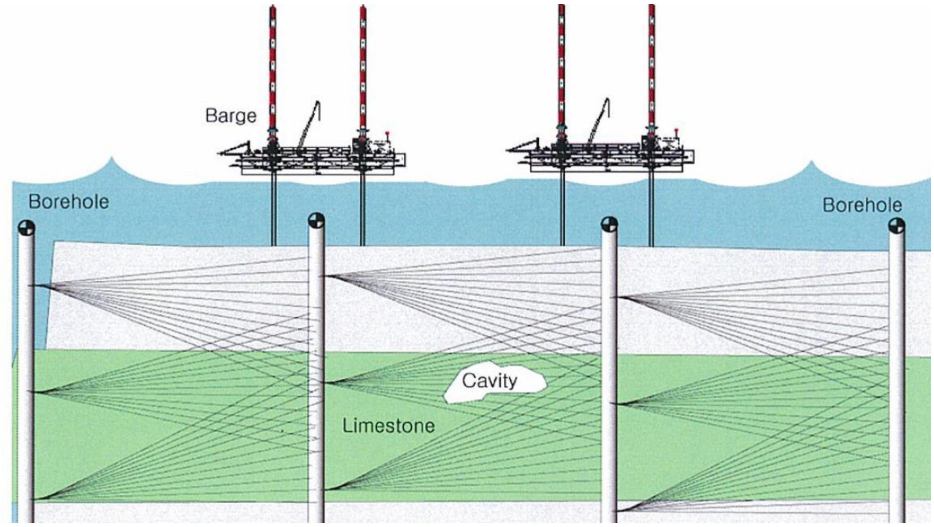


Figure 5. Schematic for marine tomography

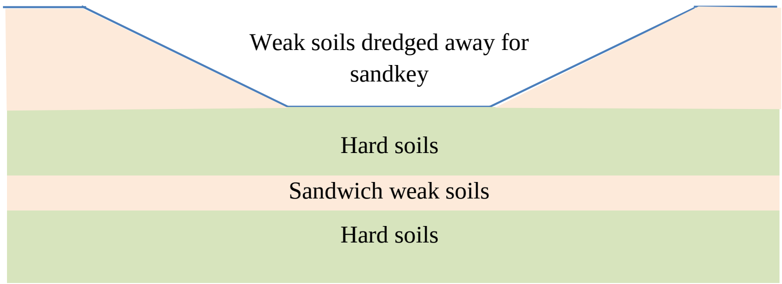


Figure 6. Schematic of soil profile with sandwich soft soil beneath hard soils

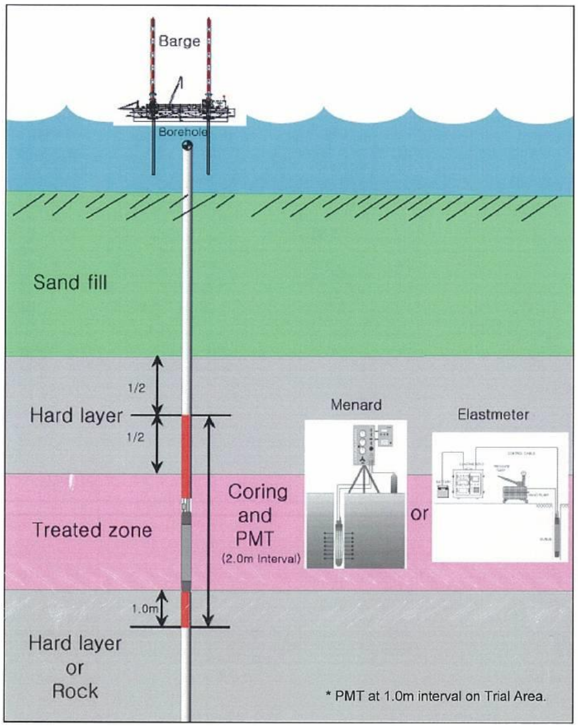


Figure 7. Schematic of In-situ Tests for treated soil layer

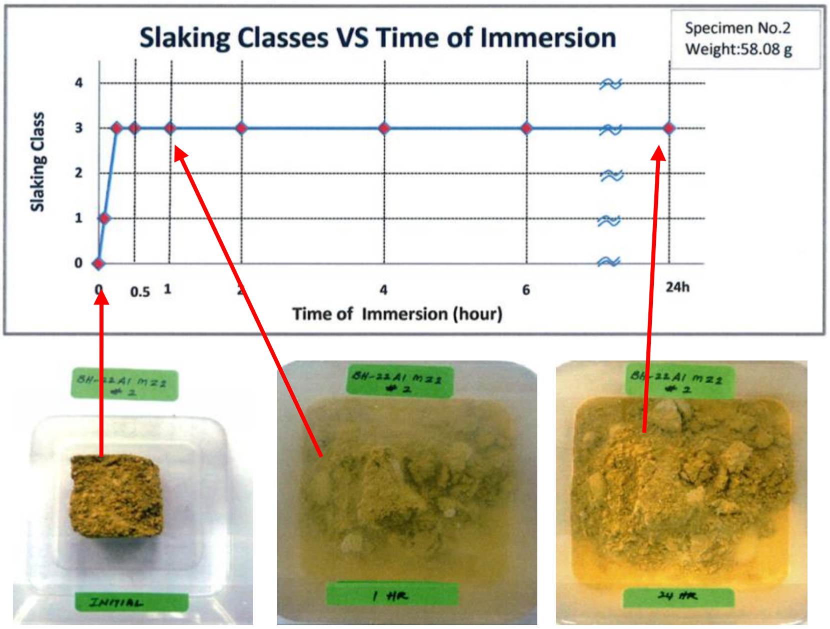
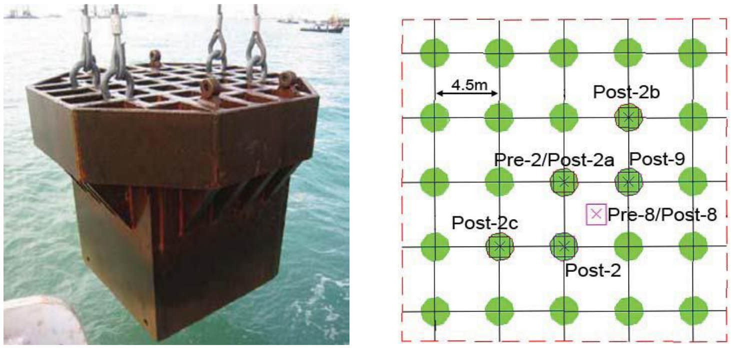


Figure 8. Slaking tests for Jurong Formation



(a) (b)

Figure 9. Dynamic Replacement (a) Specially designed and fabricated offshore DR pounder and (b) Dynamic replacement column and pressuremeter test locations (after Hamidi et al., 2010)

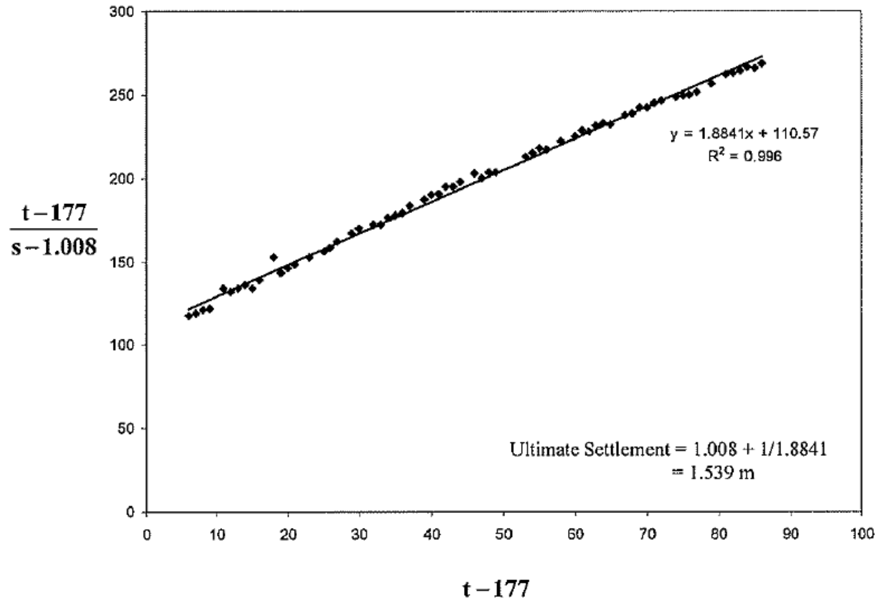


Figure 10. Estimation of ultimate ground settlement by hyperbolic method for SP-1

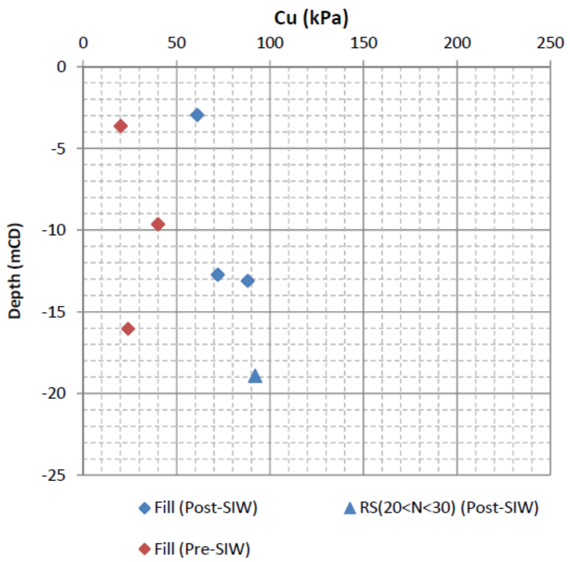


Figure 11. Comparison of undrained shear strength from Pre-SIW and Post-SIW lab tests

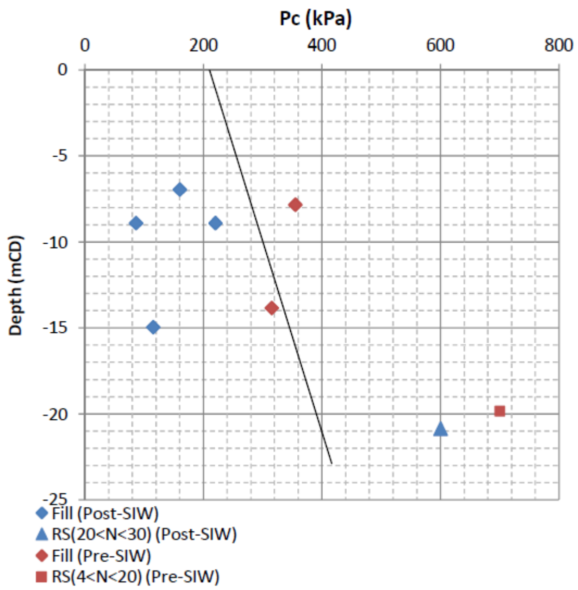


Figure 12. Comparison of pre-consolidation pressure from Pre-SIW and Post-SIW lab tests

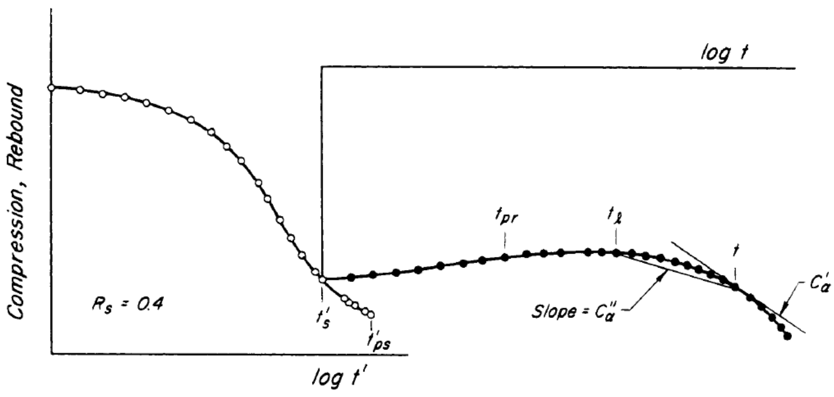


Figure 13. Definition of secant Ca'' and t_l (after Terzaghi et al., 1996)

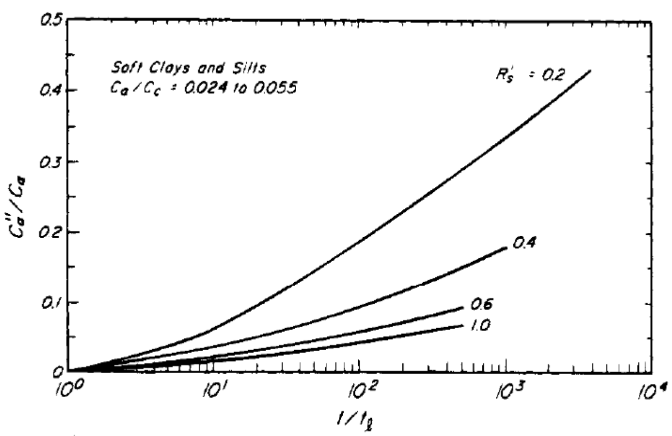


Figure 14. Post surcharge secondary compression index ca''/ca in relation to t/t_l and Rs' (after Terzaghi et al., 1996)

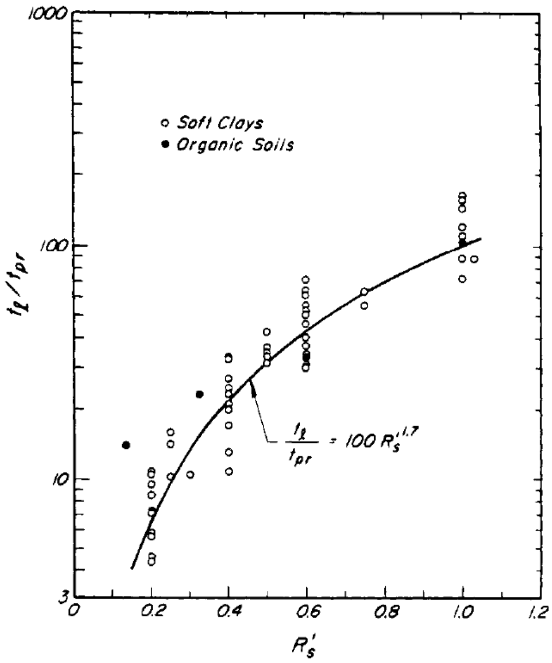


Figure 15. Empirical relationship between tl/tpr and effective surcharge ratio, Rs' (after Terzaghi et al., 1996)

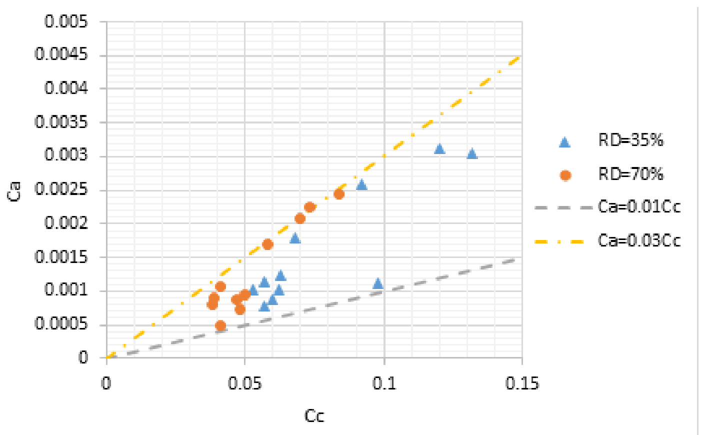


Figure 16. Ca/Cc for tested sand samples

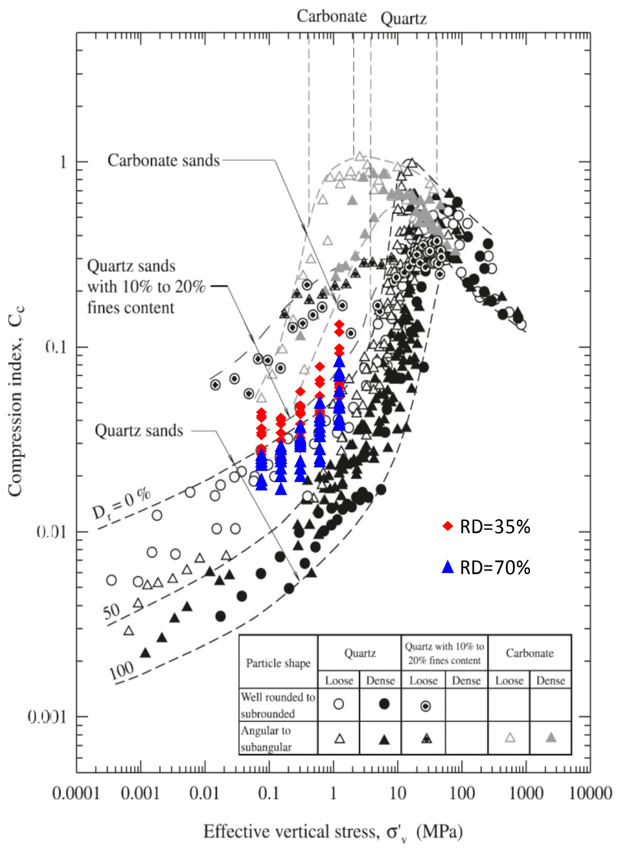


Figure 17. Compression index in relation to effective vertical stress for sand (adapted from Mesri and Vardhanabhuti, 2009)