CAISSON DESIGN AND VERIFICATION OF FRICTION COEFFICIENT OF CAISSON FOUNDATION FOR TUAS PORT PHASE 1

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Reinforced precast concrete caisson has been selected as wharf front structure for the project of Tuas Port Phase 1 in Singapore. The caisson structure and foundation were designed to withstand various loading cases comprising loads during both construction and operation stages. It was identified that sliding failure mode of the caisson structures could be one of the critical design checks. The friction tests were conducted under various conditions to assess the Coefficient of Friction (μ) between rock mound surface and concrete slab of caissons. The results show that the design value μ of 0.6 for rock mound and concrete interface could be achieved for normal flat concrete surface.

Keywords: caisson; wharf front structure; sliding; coefficient of friction; field tests

INTRODUCTION

Singapore's container port development started since the development of City and Pasir Panjang terminals. The design and capacity of the ports in Singapore change significantly over the years due to rapid increase in the cargo carrying capacity and dimensions of container vessels as shown Table 1. (Wong & Chng, 2016). The development of the ports take into account various factors such as 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.

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 city 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. Tuas Port development offers the Singapore a unique opportunity to reset the container port industry landscape. Three primary objectives for port development were hence identified: to consolidate and increase Port container handling capacity to 65 million TEUs, offer maximum flexibility to berth ships of different sizes including catering to the longest beam and draught by designing linear and deep berths, meet safe navigational requirements by planning for adequate fairways and anchorage capacity.

When completed in four phases over 30 years, Tuas Port could eventually handle up to 65 million TEUs of cargoes annually, nearly double what Singapore handled in 2016 (Kaur, 2017). The 414 ha Tuas Port Phase 1 ("TP Phase 1") development that commenced in 2015 and completed in end of 2021, and would be able to handle up to 20 million twenty-foot equivalent units (TEUs) of Containers annually.

Table 1. Development of port facilities in Singapore (Adapted from Wong & Chng, 2016)

Locations	City Terminals	PPT Phase 1 & 2	PPT Phase 3 & 4	Tuas Port Phase 1
Berth Length	270m	340m	400m	400m
Berth Area	9ha	14ha	16ha	16ha
Max Depth	13.5m	16m	18m	23m
Quay Crane	34m height 13 rows outreach	46m height 23 rows outreach	52m height 24 rows outreach	55m height 25 rows outreach
Wharf Structure	Sheet pile wall Piled deck	Piled deck Caisson	Caisson	Caisson

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Figure 1. Singapore's Container Port Development

USE OF GRAVITY CAISSONS FOR SINGAPORE's CONTAINER PORT DEVELOPMENT

Wharf front structures for container ports are designed to resist the vertical forces arising from container handling operations and horizontal loadings due to vessel berthing and mooring operations for both long and short terms.

Pile decks were commonly deployed as the wharf front structures for container ports in Singapore till early 1990s. However since Pasir Panjang Terminal Phase 1, gravity caissons have been adopted as the wharf front structures for port construction in Singapore instead (Leung & Shen, 2008; Leung, 2014).

The reasons for the use of gravity caissons ("caissons") were largely due to Singapore's unique constraint of limited land and lack of natural linear coastlines for container berths. Installed caissons forms a perimeter retaining structure to withstand the forces due to the reclamation fill materials and operational loadings behind, creating additional land and solid berths for container vessels berthing. The 8.6km of quay wall of TP Phase 1 was constructed using gravity caissons as indicated in Figure 2 and additional 57 ha of land were reclaimed as a result..

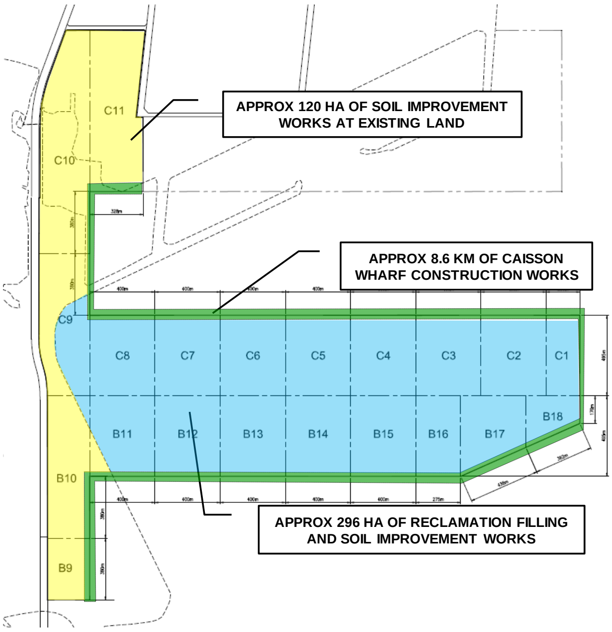


Figure 2. Layout Plan and General Scope of Works for Tuas Port Phase 1

CAISSON FABRICATION FOR TUAS PORT PHASE 1

The caissons for TP Phase 1 were prefabricated on a site nearby under sheltered factory-controlled environment, thus ensuring consistency of concrete quality and strength as well as providing a conducive working environment for the workers.

A set of proprietary lifting and transportation system consisting of hydraulic jacks were used to transfer the caissons along the rails that were laid on ground. A total of 84 hydraulic lifting/pushing jacks must be well synchronized for even lifting of each caisson approximately 7 cm above the ground. The completed caisson is then transferred to a floating dock, which is one of the largest of its kind, for positioning in the sea.

The dual-lane production of caissons using innovative slip-form method had led to increased productivity and early completion of caisson construction. (See Figure 3)

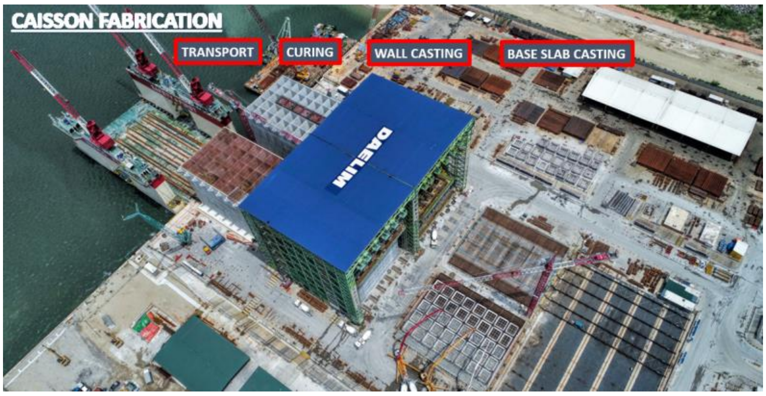


Figure 3 Caisson fabrication process

CAISSON DESIGN FOR TUAS PORT PHASE 1

The caissons for TP Phase 1 were designed with open cells for ease of transportation as they may be floated into position when the foundation bed is ready for the installation of the caissons. The open cells of the caissons are then filled with material after the caissons are placed in position to increase the overall weight of the caissons for stability.

The caisson system (Figure 4) for wharf front structures of TP Phase 1 comprised the caisson foundation, caisson structure, caisson infill, reclamation fill behind, scour protection. Subsequent wharf decking and accessories form part of the future wharf for port operations.

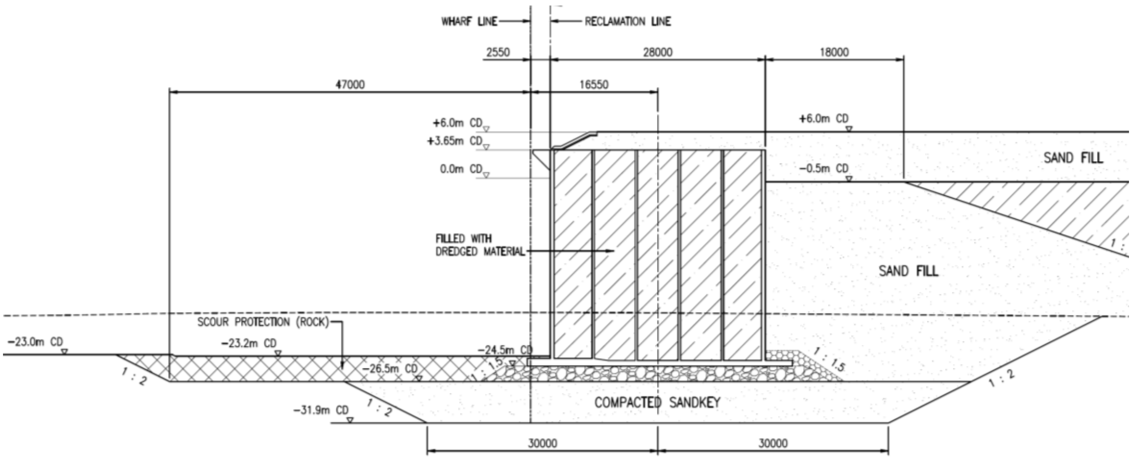


Figure 4. Typical cross section for caisson system with compacted sandkey

To construct the 8.6km wharf for TP Phase 1, a total of 221 caissons with 10 unique design types were fabricated. A typical caisson for TP Phase 1 is 28m wide, 28.15m high and 39.9m long as shown in Figure 5. Other than trapezoidal caissons, the main difference for other types of caisson is their length, which ranges from 29.9m to 39.9m. There is also a 3m toe in front at seaside and 3.5m heel at back. Caisson height is determined by the requirement of final platform level and the depth of seabed level for container vessel operation. Caisson width is determined based on stability and other design checks while the caisson length is determined by the capacity of the floating dock which is used for transportation of the caisson from casting yard.

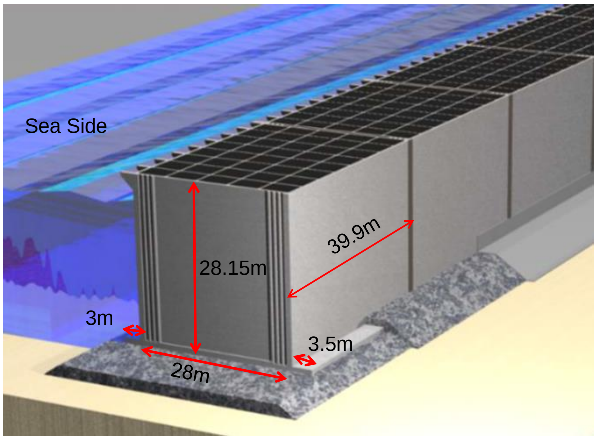


Figure 5. Typical caisson dimensions

The caisson structure system is 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, backfilled and pre-loaded, as well as the loads under operation stage due to port equipment and vessel berthing and mooring.

The various geotechnical Ultimate Limit States (ULS) related to the caisson such as sliding, overturning, bearing capacity and overall stability are checked and designed for (Esteban & Rey, 2009). Besides the ultimate limit states, the Serviceability Limit State (SLS) 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 behind the caisson.

STRUCTURAL DESIGN

The caisson is designed as a normal reinforced concrete structures with the adequate structural strength to withstand the stresses under all possible loading conditions meeting the requirements for both ultimate limit state and serviceability limit state specified in the relevant codes.

The caisson structure consists of base slab, front wall, rear wall, side wall, internal transverse walls and internal longitudinal walls. Caisson structure is assessed under loading conditions at various stages such as floating, sinking, construction and operation stages and is analysed for the loads and effects specified to obtain the most severe combinations and envelopes of stress resultants on each structural member.

The reinforced concrete member sizes are evaluated for the proposed design and sufficient steel reinforcements were provided for both bending and shear. Caisson base slabs are designed with sufficient thickness and reinforcement for the loads to be transmitted through the caisson structures and finally to the respective base slabs. The key structural elements i.e. walls and slabs are effectively tied together in the longitudinal, transverse and vertical directions to provide adequate restraint and stability to the structures. Besides the caisson structure, other structural members such as corbel are also designed accordingly.

The durability of the concrete is maintained with adequate concrete cover and limited crack width. The proposed values of minimum concrete cover which are determined by the durability requirements are presented in Table 2 following Code of Practice.

Table 2. Minimum Concrete Cover for Durability Requirement

No	Area	CP 65 / BS 5400		Nominal cover adopted in the design
		Exposure class	Concrete cover	
1	Splash and tidal zone	Extreme	55 mm	75 mm
2	Other area	Moderate	30 mm	50 mm

Table 3 shows the suggested crack width limitation by Code of Practice, which are adopted for the caisson design with crack width of 0.1mm on the splash and tidal zone and 0.25 mm on other area. The crack width is assessed at the nominal cover for durability based on serviceability limit state calculation. In addition, silane coating is also applied at both internal and external caisson surfaces from -1mCD to +3.65mCD in the splash and tidal zones to enhance the durability of the caisson structures.

Table Error! No text of specified style in document.**. Limiting Values for Crack Width**

No	Area	CP 65 / BS 5400		Value adopted in the design
		Exposure class	Crack width	
1	Splash and tidal zone	Extreme	0.1 mm	0.1 mm
2	Other area	Moderate	0.25 mm	0.25 mm

GEOTECHNICAL DESIGN

The overall stability of caisson structure is of paramount importance in caisson design as the consequence of caisson failure will be catastrophic. Overall stability design checks such as caisson sliding, overturning, deep seated failure as well as bearing capacity failure are carried out to ensure that the geotechnical ultimate limit states of caissons meet the minimum requirement specified for the project. These stability checks are carried out to examine the various loading combinations at difference stages throughout the caisson life cycle. Minimum design requirements for various stability checks at any loading scenarios used in the design are as follows.

The overall stability of the caissons in terms of sliding, overturning and overall slip failure is complied with the factors of safety from without mobilisation factors and with mobilisation factors in accordance with BS 8002 as given in Table 4, whichever is more onerous.

Table 4. Factors of Safety for Stability

Factor of Safety against:	**Without Mobilisation Factors**	**With Mobilisation Factors in accordance with BS 8002**
Sliding	1.75	1.0
Overturning	2.5	1.0
Overall failure	1.5	1.0

For the allowable bearing capacity, a factor of safety of 3.0 against ultimate shear failure is applied to dead loads alone. A lower FOS of 2.5 is adopted when applied to dead plus live loads. The more onerous condition is adopted. Table 5 summaries the factor of safety for each loading case.

Table 5. Factors of Safety for Bearing Capacity

Cases	Factor of Safety
Dead Load only	3.0
Dead Load + Live Load	2.5

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. Stringent limits are imposed on the quay crane rail tracks to ensure smooth movements of the quay crane without compromising the container handling operations at the wharf. The limits are given for both settlement and differential settlement, as shown in Table 6. With the determination of caisson dimensions based on the stability analysis, Finite Element (FE) analysis is used to evaluate the movement of the caisson system to meet the design requirements.

Table 6. Maximum Allowable Movements for Caisson During Operation

Caisson Movement	Total	Differential between adjacent caissons
Vertical Settlement	30mm	1:750 over 20m spacing
Horizontal Deflection	30mm	1:750 over 20m spacing
Rotation/Tilt	1:750 (both seaward and landward tilt)	

To meet the stringent total and differential settlement for the caissons, it is of great importance to ensure that the caissons rest on competent foundation and caissons are preloaded (Tan et al., 1999, Khoo et al., 2013, Leung & Zhang, 2017).

Caisson Foundation

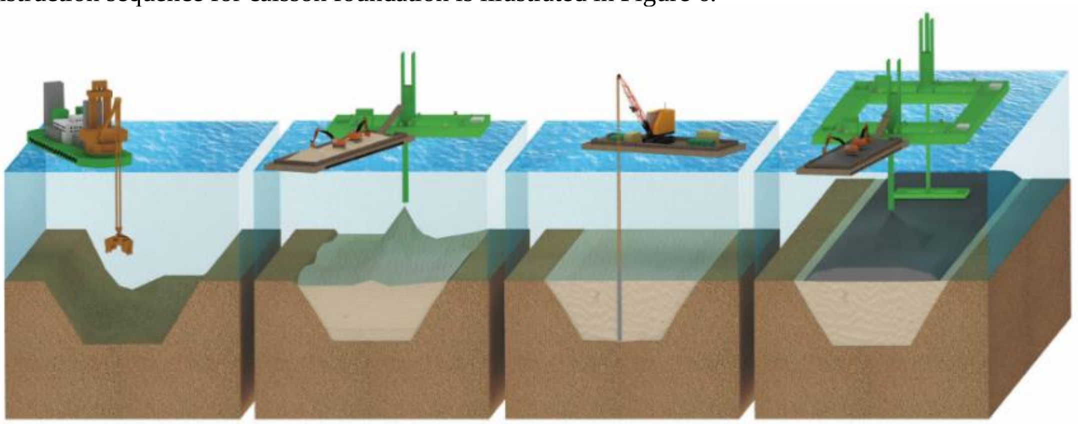
The caisson foundation is provided to satisfy not only the overall stability of caisson structure, but also the stringent settlement requirement of caissons. From the soil investigation carried out at Tuas

Port Phase 1, it revealed that the ground conditions at project site generally comprise Kallang Formation and Jurong Formation. Kallang formation is generally soft which is not suitable for the caisson foundation.

Soft soil below caisson are hence first dredged from the seabed till a competent layer with SPT N=50 to form the sandkey trench. The depth of the sandkey varies across the alignment of the wharf in accordance to the ground conditions, which could be up to 30m deep.

Sand of good quality is subsequently filled into the dredged trench. The filled sandkey is then well comapcted through vibro compaction to achieve a minimum relative density of 75% to increase the stiffness and reduce the caisson movements. The quality of the compaction is confirmed by the in situ cone penetration test (CPT) to meet the equivalent minimum cone tip resistance as shown in the Table 7.

Durable and strong rocks are then laid on top of the sand layer, compacted and levelled to form the rock mound on which the caisson is placed. This rock mound layer will serve as a flat surface for easy installation and as a load distribution from the caisson base to the foundation layer. The typical construction sequence for caisson foundation is illustrated in Figure 6.



(a) Sandkey dredging (b) Sandkey filling (c) Vibro compaction (d) Rock mound construction

Figure 6. Typical construction sequence of caisson foundation

Table 7. Minimum Cone Tip Resistance of sandkey after compaction

Depth of sand fill (m)	Minimum Cone Tip Resistance, qc (MPa)
Top of 1m	8
Top of 2m	10
2 to 3	12
3 to 4	13
4 to 8	13
8 to 10	15
10 to 15	18
15 to 20	20
> 20m	22

Caissons Preloading

In addition to the compacted sandkey and rock mound, caisson preloading is required to meet the stringent movement requirement. All caissons are preloaded to an appropriate load intensity higher than the dead loads above 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 allowable residual settlement requirement behind the caissons within TP Phase 1 for operating driverless automated guided vehicles is 40mm. Soil improvement works within the container stacking yard adopting Prefabricated Vertical Drains (PVD) with surcharge are carried out. In practice, the caisson preloading and soil improvement works behind the caissons are carried out concurrently to minimize the impact to the caissons.

The required caisson surcharge magnitude of 120kPa are determined based on operational live loads and the allowable residual settlement. The foundation after removal of the preloading will be in an over-consolidated state. These measures enhance the overall stability of the caisson structures and reduce the residual settlement of the ground during port operation.

Analysis Results

During the design stage of the Tuas Port Phase 1, various loading conditions during construction and operational stages were considered in the design to ensure the integrity and stability of the entire caisson system and the most critical loading condition for each stage was identified. During the construction stage, the critical loading condition is when there is maximum surcharge loading behind the caisson while there is minimum loading on top of the caisson. During the operational stage, the most critical loading condition is when there is maximum vertical operational loading on top of caisson from quay crane together with the bollard pull and surge loading.

Table 8 summarizes the stability check results for the adopted design. It shows that all the design items can meet the stipulated design requirements. It can also be seen that the calculated factor of safety against sliding under critical load condition just meets the minimum design requirement. Hence, it becomes a critical design check which is proposed to be verified through in situ confirmation tests during project implementation stage before the mass construction of caissons.

Table 8. Summary of checks for geotechnical ultimate limit state

Item	Min Achieved FOS	Requirements
Bearing Capacity	2.55	≥ 2.5
Sliding	1.75	≥ 1.75
Overturning	3.96	≥ 2.5
Overall Stability	1.40 (Construction)	≥ 1.4
	1.606 (Operation)	≥ 1.5

VERIFICATION OF COEFFECIENT OF FRICTION

Various factors which may affect the factor of safety for sliding had been studied during the design process to optimize the design. The factor of safety for sliding is the ratio between the resistance and driving force. The design process involved to maximize the resistance while minimizing the driving forces by examining various components of the caisson system.

Sand backfill behind the caisson instead of the clayey dredged material is adopted to provide higher friction angle to reduce the earth pressure for the driving force. The dredged material will be only filled behind the sand backfill as shown in Figure 4 which is beyond the influence zone. In addition, a permeable caisson joint is designed to minimize the water level difference between in front and behind the caisson which reduces net water pressure acting on the caisson.

On the other hand, the dredged material is used for the caisson infill instead of traditional sand infill to maximize the use of non-sand material, which will result in less resistance due to smaller unit weight of the dredged material. In the design, this is overcome by increasing the caisson width with toe and heel for the caisson base slab.

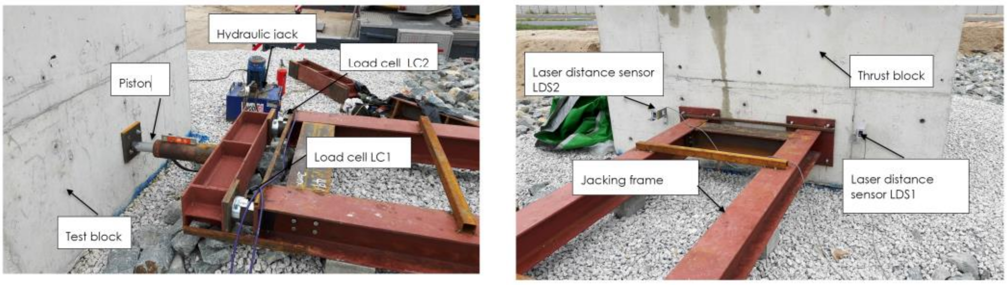
Moreover, a minimum 2 m thick of compacted rock mound with friction angle $\theta_r = 45$ degrees is proposed to support the caisson and provide the friction resistance against sliding. Hence, interface between caisson concrete base slab and rock interface has higher friction resistance to improve the sliding stability than the concrete and sand interface without rock mound.

For normal flat concrete base with rock bed, BS6349 recommends using two third of the rock mound friction angle as the interface friction angle, which result in friction coefficient of 0.58. The Japanese Technical Standards for Port and Harbour Facilities recommends adopting 0.6 as the friction coefficient. A coefficient of friction μ of 0.6 is adopted for design, which is proposed to be verified by the in situ friction test under similar conditions before commencement of caisson construction.

Test Setup

During construction stage of Tuas Port Phase 1 project, a large-scale field test with real time instrumentation monitoring was hence carried out to verify the design coefficient of friction of between concrete base slab and rock mound surfaces.

The test setup as shown in Figure 7 comprises two main components, the testing and measurement components. The testing ground was prepared by using the same sizes of rock required for actual rock mound design and compacted to the requirements. After that, reaction frame (concrete block) was placed on the testing ground and followed by installation of necessary jacking frame platform, hydraulic jack, instrument such as load cell and laser sensor, and test slab. The test block was displaced under the horizontal load applied by hydraulic jack through the jacking frame and thrust blocks. Heavier thrust blocks provide the reaction force and remain stable during the tests.



(a) Test blocks and load cells (b) Thrust blocks and laser distance sensors

Figure 7. Test Setup

Once the testing system was set up, the friction test was started by jacking the test slab with uniform loading and the real-time displacement and loading were recorded simultaneously. Once sliding observed during the test which signaled constant friction at the testing interface, the jacking operation was terminated. The load applied throughout the test is measured by two Nos of load cells LC1, LC2. The displacement of the test block is measured by laser distance sensors LDS1, LDS2 mounted on the thrust block.

Test Conditions

The friction tests were carried out under different conditions, as listed in Table 9. Contribution of varying variables to the coefficient of friction such as wet and dry condition, roughness of concrete surfaces and applied forces, were considered.

The friction tests were conducted under both the wet and dry surface conditions with real time monitoring of the movement of concrete blocks.

- The dry condition: The interface between concrete surface and rock base does not contain any water.
- Wet condition: The interface between concrete surface and rock base is placed below water. This is same condition as the actual caissons.

A minimum depth of 0.25m above the top of the rock mound was maintained through provision of membrane at the testing area.

Table 9. Summary of test configurations

Test No	Water Condition	Roughness	Test blocks
No. 1	Dry Condition	Normal Roughness	Concrete Block 2mx2mx1.5m
No. 2			Concrete Block 2mx2mx1.5m + Kentledge 4x1mx1mx1m
No. 3		Normal Roughness + Groove	Concrete Block 2mx2mx1.5m
No. 4			Concrete Block 2mx2mx1.5m + Kentledge 4x1mx1mx1m
No. 5	Wet Condition	Normal Roughness	Concrete Block 2mx2mx1.5m
No. 6			Concrete Block 2mx2mx1.5m + Kentledge 4x1mx1mx1m
No. 7		Normal Roughness + Groove	Concrete Block 2mx2mx1.5m
No. 8			Concrete Block 2mx2mx1.5m + Kentledge 4x1mx1mx1m

(a) Dry Condition (b) Wet Conditions

Figure 8 Test water conditions

In addition, the tests were carried out under two types of surface roughness condition for the caisson base slab as illustrated in Figure 9.

- Normal Condition: The concrete surface is flat, which is similar to the condition at caisson fabrication yard for caisson base slab fabrication.
- Groove Condition: The concrete surface has grooves with width 20mm, height 5mm and spacing 500mm in order to increase of roughness of the concrete surface.


(a) Normal surface condition


(b) Groove surface condition

Figure 9. Concrete surface roughness conditions

Lastly, to obtain a more representative of the test results, two vertical load conditions for the test blocks were proposed as illustrated in Figure 10.

- Without additional kentledge blocks: The size of test block is 2m x 2m x 1.5m.
- With additional kentledge blocks: 2m x 2m x 1.5m base test block with additional 4 numbers of 1m x 1m x 1m kentledge blocks on top.

The thrust blocks comprise of a 3mx3mx1.5m block with additional 9 numbers of 1m x 1m x 1m kentledge blocks on top, which provide the sufficient reaction force during the tests to displace the test blocks.


(a) 2mx2mx1.5m test block

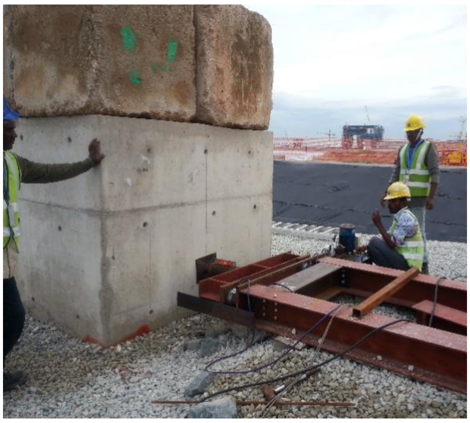

(b) 2mx2mx1.5m test block

Figure 10. Vertical load conditions for test blocks

Test Results

After the tests, the test results are interpreted and the coefficient of friction for each test can be derived based on the formula as conceptualized in Figure 11. The Total horizontal load is sum of the measurements from the two load cells. The displacement of the test block is the average of the readings from the two laser distance sensors.

$$\mu_{test} = \frac{P_{max}}{W} \quad (1)$$

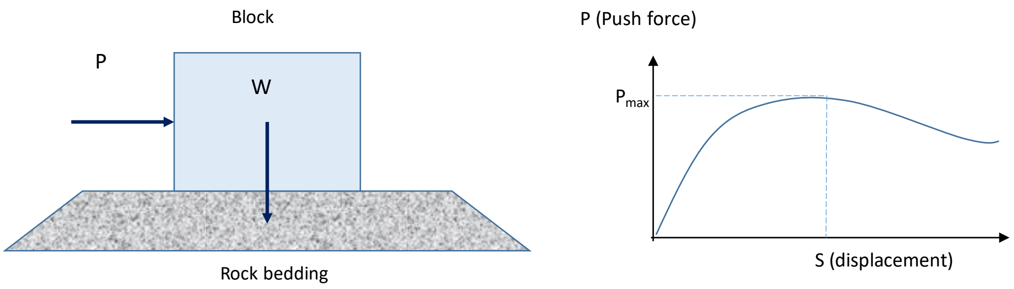


Figure 11. Concept and interpretation for friction tests

A typical test result with the real time measurements is presented in Figure 12 for Test No 2. The test results all tests are tabulated in Table 11 and shown in Figure 13. It is shown that the coefficient of friction from all tests are more than the requirement of 0.6. are similar among different testing conditions. Dry condition is slightly higher than wet condition. Grooved base is slightly higher than normal base condition.

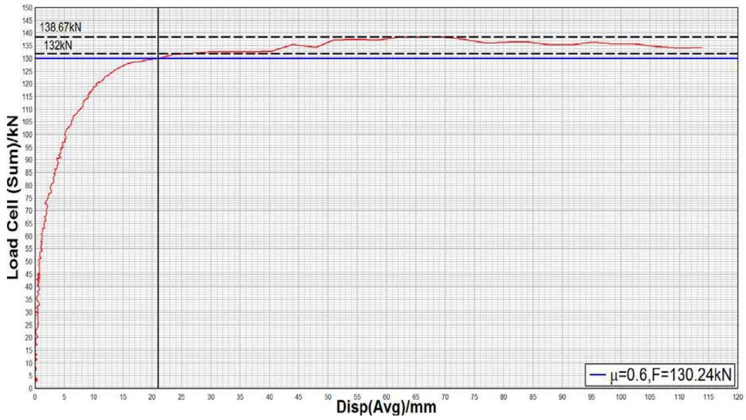


Figure 12. Typical test curve

Table 11. Summary of the coefficient of friction from tests

Test No	Water Condition	Roughness	Test Slab	μ
No. 1	Dry Condition	Normal Roughness	Concrete Block	0.67
No. 2			Concrete Block + Kentledge	0.64
No. 3		Groove	Concrete Block	0.67
No. 4			Concrete Block + Kentledge	0.71
No. 5	Wet Condition	Normal Roughness	Concrete Block	0.62
No. 6			Concrete Block + Kentledge	0.64
No. 7		Groove	Concrete Block	0.70
No. 8			Concrete Block + Kentledge	0.65

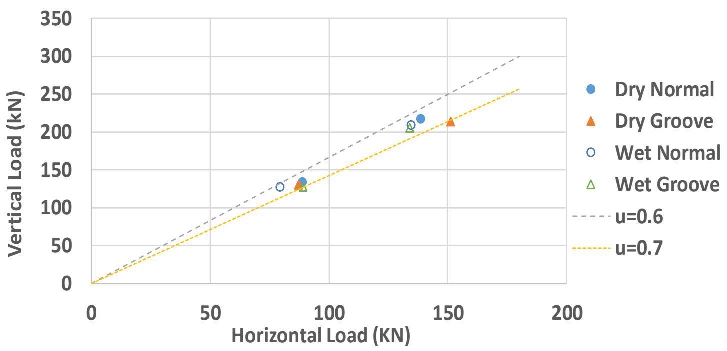


Figure 13. Test results

The above test results demonstrated that the design value 0.6 of coefficient of friction for rock and concrete interface could be achieved under the normal concrete surface and wet condition with adequate compaction to the rock mound. Hence, the caisson base slabs were constructed with normal surface conditions without additional treatment of the surface roughness, which saved project time and cost.

SUMMARY

The development of Container Ports in Singapore over the years have evolved. Tuas Port Phase 1 was completed in Nov 2021 with various scope of works on reclamation, dredging, wharf construction and soil improvement works.

Gravity Caissons are adopted as the best option for port wharf structures due to its ability to create more land, flexibility to receive dredged materials. Caissons need to sit on competent foundation and preloaded to meet the stringent design requirements under both ultimate and serviceability limit states.

Contribution of varying variables to the coefficient of friction such as wet and dry condition, roughness of concrete surfaces and applied vertical forces, were carried out. The results show that the design friction coefficient of 0.6 for the normal surface condition is verified through the in-situ trial tests. Hence, the caisson base slabs were constructed with normal surface conditions which provides adequate sliding resistance.

REFERENCES

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

Kaur, K. 2017. Full steam ahead for new Tuas mega port. The Straits Times, 24 April 2017, *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.

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.

Leung, C.F. & Zhang Y.D. 2017. Geotechnical Aspects of Container Port Development. *GeoSS a Decade of Geotechnical Advances*, 261-286

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

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

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.