Reclamation and Safety of High Cantilever Strong Box Pipe Pile Seawall in Marine Clay

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ABSTRACT: *A high cantilever seawall was constructed in soft clay of 15 m to 32 m thickness and the enclosed water between the shoreline and the seawall was reclaimed. The area was 7 hectare and the reclamation height ranged from 10 m to 20 m. The reclamation methods were controlled and not intended to cause additional incremental deflection to the already high design deflection of -385 mm or 0.89% of H (H = 44 m is defined as rotating and not cantilever height). However the different methods of reclamation had a significant influence on the slurry clay flows and thus caused additional incremental deflection. The slurry clay flows pushed the deflection of seawall to an additional maximum magnitude of -353 mm or 0.80% of H. The maximum seawall deflection of -526 mm or 1.20% of H was based on inclinometer IW1903 readings. At such high seawall movement, the inclinometer deflection profiles indicate safety in the design and the long term deflection is stable. This reclamation experience indicates that it is difficult to control slurry clay flows in future high reclamation filling of 10 m to 20 m height above seabed with underlying thick soft clay of 15 m to 32 m. The deflection is far exceeding BS8002 guideline of 0.5% of H, which stipulates the geo-structural stability of the seawall. This long term stability at such a high deflection is attributed to the robust turning back capacity of the strong box. The turning back capacity of the strong box against the active force above the rotating point is from the integration of vertical bending capacity of the pipe piles, the rigid connection of the capping beam, and utilisation of the P-P interlock to produce the box action. Below the rotating point the adequate passive resistance into Old Alluvium maintains the external equilibrium.*

KEYWORDS: Cantilever seawall, earth pressure, pipe piles, reclamation, retaining structure, clay flows, soft marine clay

SITE LOCATION: IJGCH-database.kmz (requires Google Earth)

INTRODUCTION

The strong box pipe pile seawall was constructed in the marine environment as shown in Figure 1 and subsequently the enclosed water was reclaimed for the land construction of a piled roadway. Reclamation fillings by the different methods indicated that the reclamation procedure had a significant influence of the seawall deflection. For purpose of explaining the deflection and safety, the seawall is divided into 4 sections as shown in Figure 2 and Table 1. Section 1, 2 and 4 registered deflection within the calculated deflection and insignificant incremental deflection. Only section 3 showed deflection significantly higher than the theoretical active wedge deflection and significant sudden incremental deflection. Section 3 has the highest rotating height of 40 m to 44 m and only a portion of section 2 shows a rotating height of 40 m. Method B created the very thick and huge slurry clay ponds B and A beyond the 44m active wedge. The combination of the high rotating height and the closeness of the inappropriate reclamation method B caused the sudden plunge in deflection at section 3. At the high deflection of -526 mm or 1.20% of H, the question arising is whether the seawall is stable in the long term. Inclinometer deflection of section 2 and 3 are compared.

Another observation of high rotating height of 30 m to 44 m is that construction works had a higher noticeable effect within the active wedge. Tidal effect was also observed to oscillate the 1500 ton/m box up to a magnitude of 8 mm.

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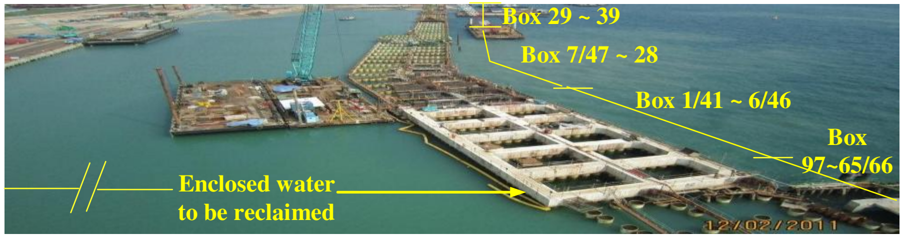


Figure 1. Photograph showing the enclosed water to be reclaimed.

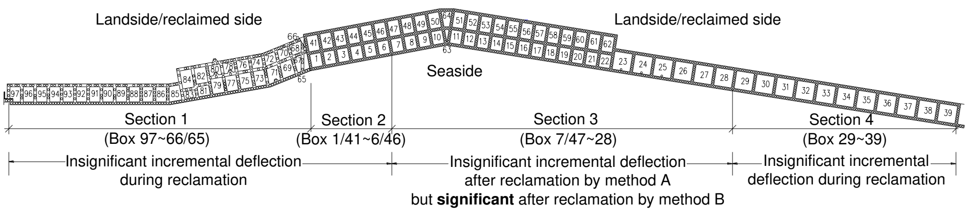
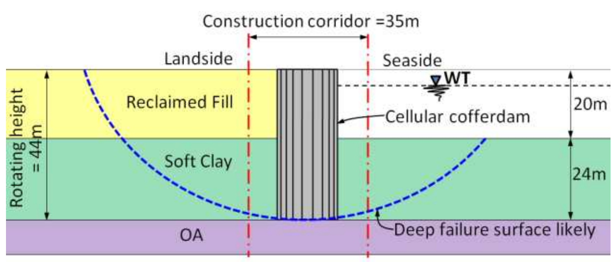


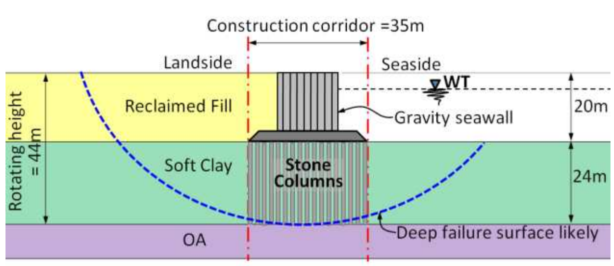
Figure 2. Various sections of strong box pipe pile seawall.

Table 1. Sections of significant and insignificant incremental deflection.

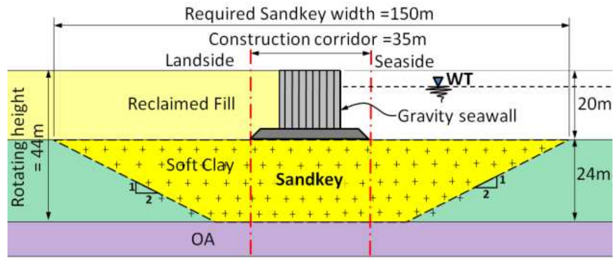
Sections	Cantilever height	Soft clay thickness	Rotating height (H)	Sudden incremental deflection	Maximum deflection
Section 1	7.5~18.5m	7.5~14m	14~19.5m	Insignificant	Within design deflection
Section 2	15~18.5m	15~21m	25~40m	Insignificant	Within design deflection
Section 3	12~20m	23.5~32m	40~44m	Significant	Exceed design deflection
Section 4	5~10.5m	14~24.5m	25~34.5m	Insignificant	Within design deflection



(a) A Cellular cofferdam

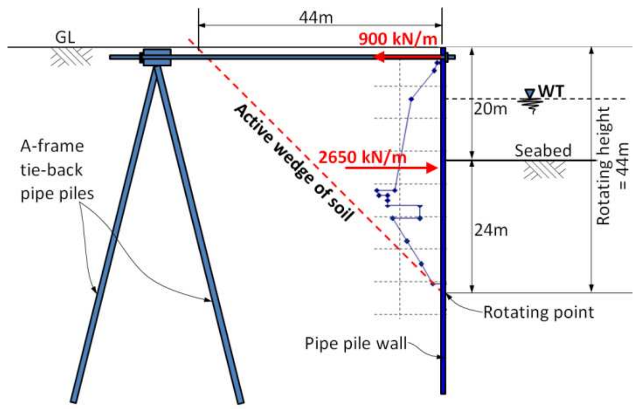


(b) Ground improvement using grouted stone column or deep soil mixing.



* 150 m wide and 44 m deep sandkey is necessary to cut off the deep slip failure plane.

(c) A safe sandkey solution but not buildable.



* Not applicable to provide a horizontal pull back reaction in the A-frame and an adequate single row steel structural wall.

(d) A-frame tie-back system is not applicable.

Figure 3. Typical seawall solutions that are not applicable.

The applicable strong box seawall that is buildable in these physical circumstances are detailed by Soh et al. (2014). The high rotating height and the construction corridor of 35 m limit the seawall solutions. A cellular cofferdam is not able to resist the deep seated failure as shown in Figure 3(a). Ground improvement such as stone column or deep soil mixing are impossible to construct in the marine condition and also likely to fail by tensile breakage as shown in Figure 3(b).

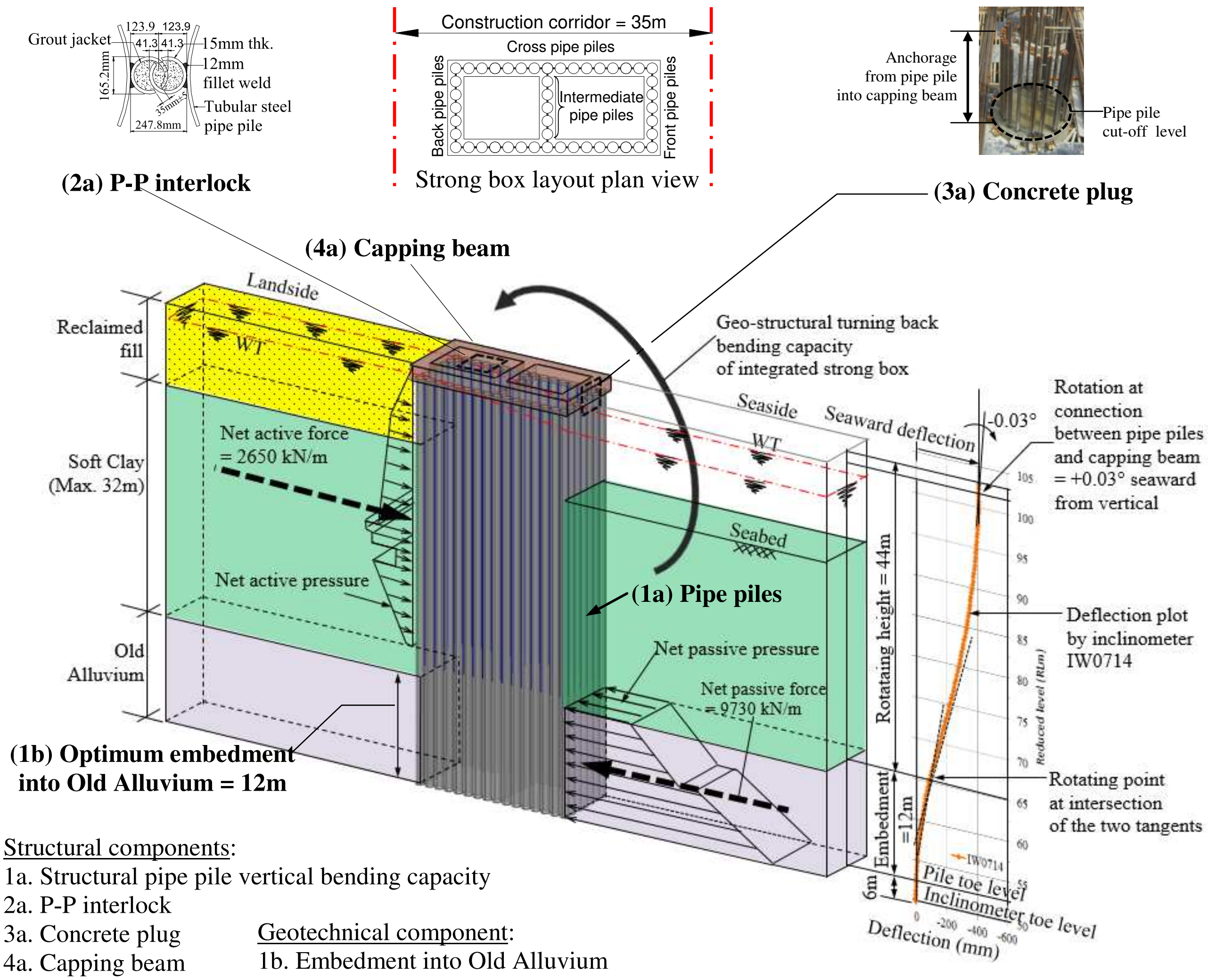
A safe sandkey of 150 m width is impossible to build within the 35 m construction corridor as shown in Figure 3(c). A-frame tie-back as shown in Figure 3(d) is not able to derive the adequate pull-back force and there is no single row steel structural wall strong enough to resist the high active force. It is necessary to design an integrated strong box structure with the adequate geo-structural turning back capacity above the rotating point, adequate geotechnical passive resistance below the rotating point, and buildable within the 35 m construction corridor as shown in Figure 4.

SHEAR STRENGTH OF SOFT CLAY

The strength parameters of soil layers are indicated in Table 2. The undrained shear strength based on field vane test is shown in Figure 5. The inappropriate reclamation method resulted in significant clay flows, and the tremendous drop from undrained strength to slurry consistency.

STAGES OF RECLAMATIONS

The different stages of reclamations are identified to be in accordance with the different reclamation methods. Reclaimed fill consists of either sand of less than 15% fine (Grade A material) or sand with more than 15% but less than 35% fine (Grade B material). Figure 1 shows the enclosed water to be reclaimed after the construction of the interlocking pipe piles box structure together with partial completion of the capping beam. Figure 6 shows stage 1 reclamation,



Structural components:
1a. Structural pipe pile vertical bending capacity
2a. P-P interlock
3a. Concrete plug
4a. Capping beam

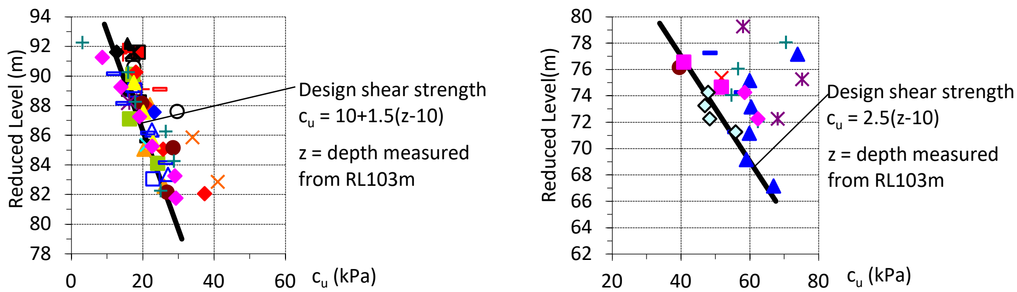
Geotechnical component:
1b. Embedment into Old Alluvium

Figure 4. Four structural and one geotechnical components that define the geo-stability of the seawall.

Table 2. Design soil parameters.

Soil type	Max. thk.	Ave. SPT-N	Drainage condition	γ (kN/m^3)	c′ (kPa)	ϕ' (Deg.)	c_u (kPa)	E_{50} (kPa)
Reclamation Fill	20m	7	Drained	19	0	30	-	10000
Upper Marine Clay	32m	0	Undrained	16	-	-	10+1.5(z-10)	250*c_u
Upper Peaty Clay		4	Undrained	17	-	-	30	200*c_u
Upper Intermediate Clay		8	Undrained	20	-	-	50	250*c_u
Lower Marine Clay		4	Undrained	16	-	-	2.5(z-10)	250*c_u
Lower Peaty Clay		5	Undrained	17	-	-	45	200*c_u
Lower Intermediate Clay		12	Undrained	20	-	-	65	250*c_u
Old Alluvium (OA)	-	10~30	Drained	21	5	28	-	1500*N
Old Alluvium (OA)	-	30~50	Drained	21	10	28	-	2000*N
Old Alluvium (OA)	-	>50	Drained	21	10	30	-	2000*N

Note: N = SPT; γ = unit weight; c' = drained cohesion of soil; ϕ'= drained angle of friction of soil; c_u= undrained shear strength of soil; E_{50}= elastic modulus of soil; z= depth measured from RL103m; RL103m is the elevation of the top of capping beam.



(a) Upper Marine Clay. *(b) Lower Marine Clay.*

Figure 5. Undrained vane shear strength of soft clay. Note: Reduced level is the ground elevation level of the reclaimed fill.

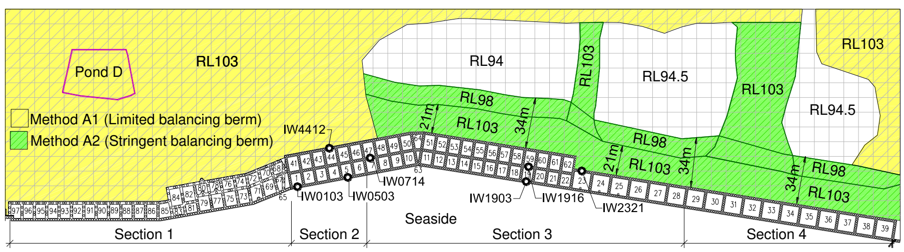


Figure 6. Stage 1 reclamation - Extent of reclamation where the seawall deflection was in accordance with the anticipated deflection (Method A1 & Method A2).

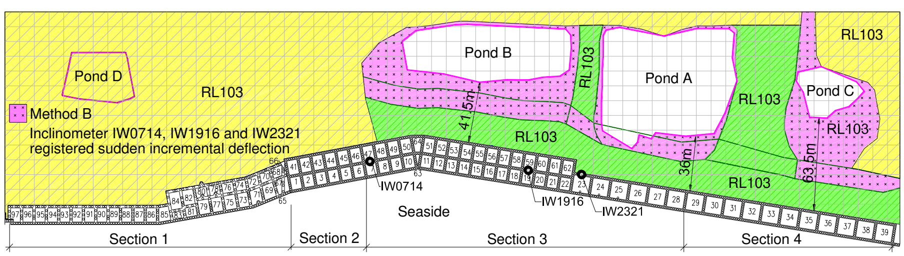


Figure 7. Stage 2 reclamation - Extent of reclamation where three inclinometers IW0714, IW1916 and IW2321 showed significant incremental deflection or sudden plunge (Method B).

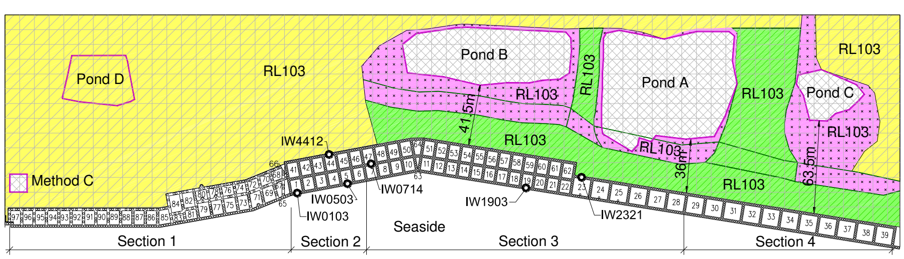


Figure 8. Stage 3 reclamation - Filling of ponds B, A, and C showed insignificant incremental deflection of the seawall (Method C).

where the seawall was behaving in accordance with the anticipated deflection as shown by seven inclinometers IW0103, IW4412, IW0503, IW0714, IW1903, IW1916 and IW2321. The slow limited balancing berm method A1 or stringent balancing berm method A2 were enforced up to this stage 1 reclamation.

Figure 7 shows stage 2 reclamation, where three inclinometers IW0714, IW1916 and IW2321 of the seawall were observed to deflect significant incremental amount or sudden plunge. The surface area of reclamation was noted to be relatively small in comparison to the stage 1 reclamation of Figure 6. It was interpreted that the full earth discharge method B was applied around the edges of the then created clay slurry ponds beyond the 44 m active wedge and were not observed at that period of time. Figure 8 shows stage 3 reclamation whereby very stringent controlled filling was carried out to prevent the created clay slurry ponds from further slurry flows.

RECLAMATION METHODS

There are three reclamation methods used for this 7 hectare reclamation.

Reclamation by Limited Balancing Berm Method A1

Reclamation steps of limited balancing berm method A1 is shown in Figure 9. The reclamation filling of Grade A material by the limited balancing berm was likely to cause minimal localized squeezing of the soft clay below the seabed. The squeezing effect was probably limited at 1 m to 2 m below the seabed as the factor of safety was slightly less than 1. The lowest factor of safety was at step 2 and step 3 and was localized after completion of 20 m berm to RL103 m.

Reclamation by Stringent Balancing Berm Method A2

Reclamation by the stringent balancing berm method A2 as shown in Figure 10 was a slower filling of Grade A material. The lowest factor of safety for the stringent balancing berm method A2 at step 2, step 3 and step 4 was 1.18. This method was unlikely to cause any clay squeezing below the seabed.

Reclamation by Full Earth Discharge Method B

Reclamation steps of full earth discharge method B are shown in Figure 11, after stringent balancing berm method A2. Direct full discharge of Grade B material straight from the lorry was likely to cause significant slurry clay flows during stage 2 reclamation. The extent and depth of the slurry clay flows could be deep, intensive and extensive, if the direct discharge was concentrated on a particular spot. The factor of safety was extremely low for the accumulated lorry load by direct full earth discharge. This method of reclamation was extremely fast. Vast quantity of Grade B material was poured directly into the enclosed water within a very short period as shown in Figure 7, and this caused the massive slurry clay flows at ponds B and A. This method of reclamation was not recorded in the site. The method was interpreted during the period when the inclinometers recorded sudden plunge in the deflection of seawall and the sighting of heaving of clay slurry at pond B on 25 Nov 2011. Chu et al. (2009) reported the slurry clay upheaval in a stringently controlled filling procedure for a slurry pond in Singapore.

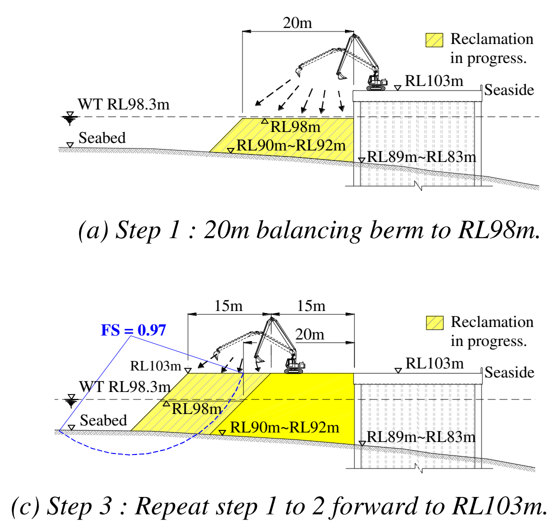
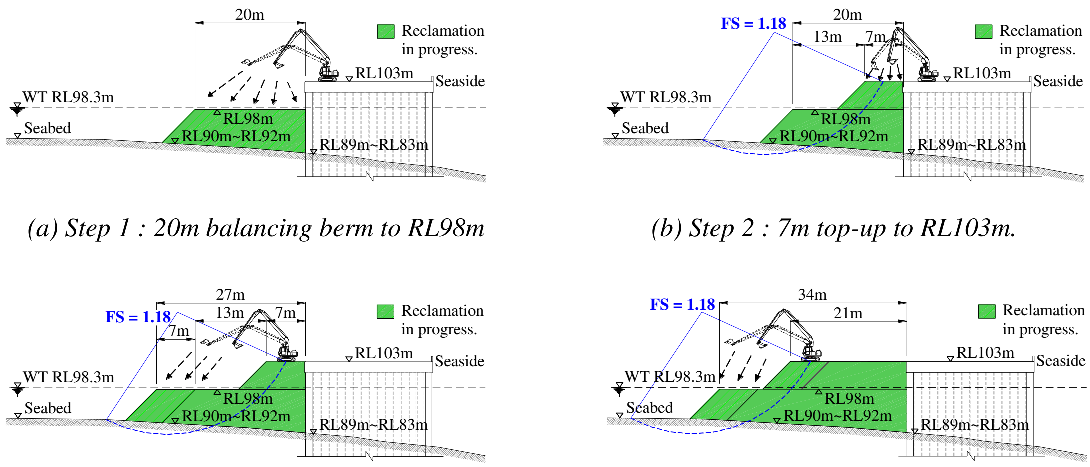


Figure 9. Limited balancing berm for stage 1 reclamation - Method A1.



(a) Step 1 : 20m balancing berm to RL98m

(b) Step 2 : 7m top-up to RL103m.

(c) Step 3 : 7m extension of RL98m berm.

(d) Step 4 : Repeat step 1 to 3 forward to RL103m.

Figure 10. Stringent balancing berm for stage 1 reclamation - Method A2 .

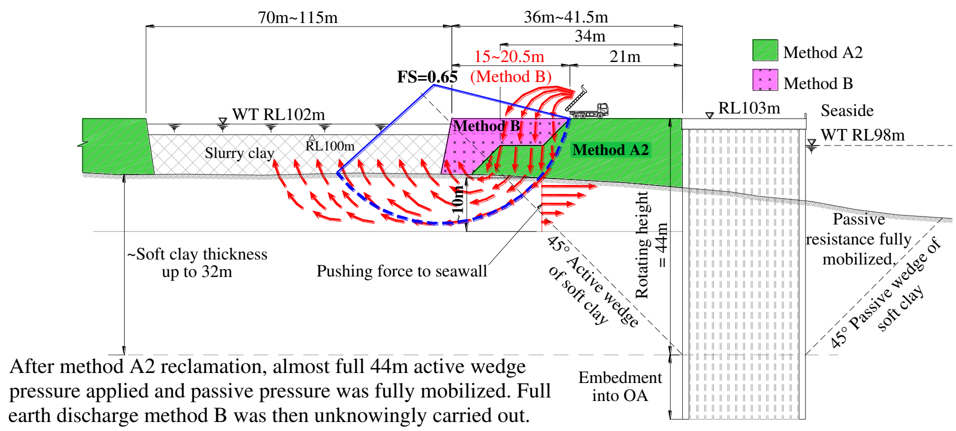


Figure 11. Full earth discharge method B for stage 2 reclamation at section 3.

Reclamation by Controlled Filling Method C

Reclamation by controlled filling method C for filling into ponds B, A and C is shown in Figure 12. This control filling of 0.5 m thickness with Grade A material was enforced for the clay slurry ponds A, B and C. For pond A, a grid pattern of 10 m x 10 m was set up for the filling that was carried out in progression. At each step of progression, the fill thickness was

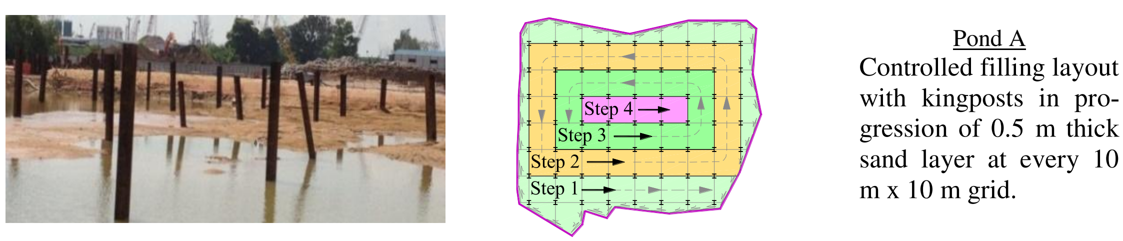
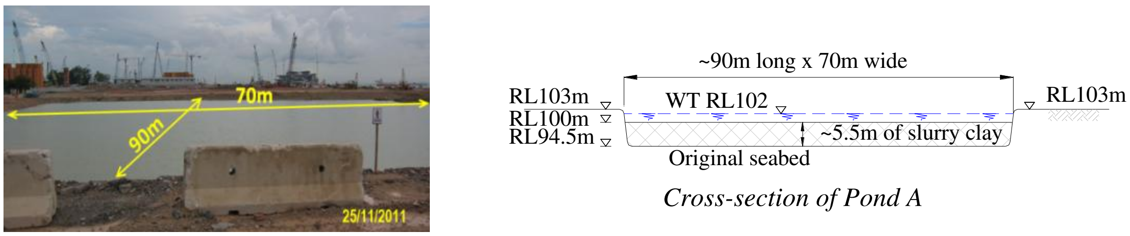


Figure 12. Photograph showing controlled filling method C for stage 3 reclamation.

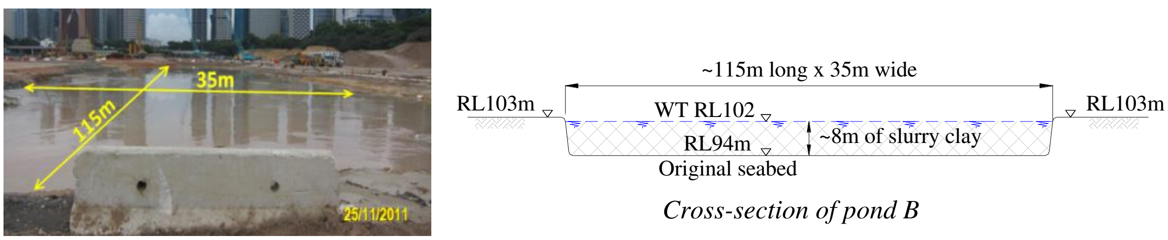
not exceeding 0.5 m. Even with such stringent control filling, some of the kingposts were noted to tilt. The method of reclamation was extremely slow and may cause clay squeezing of thickness less than 0.5 m. Similar layering reclamation filling in 0.5 m thickness was carried out for ponds B and C.

CREATION OF CLAY SLURRY PONDS

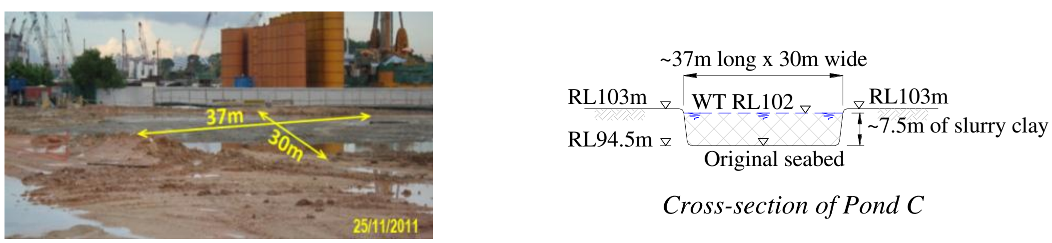
The clay slurry ponds B and C were observed on 25 Nov 2011 as shown in Figure 13, when the clay slurry was pushed sideways and upward to the pond water level RL102.0 m. Pond A was still submerged in water and the clay slurry thickness was determined by chain sounding.



(a) Photograph of pond A (90 m x 70 m x 5.5 m) with slurry clay heaved from RL94.5 m to RL100 m.



(b) Photograph of pond B (115 m x 35 m x 8 m) with slurry clay heaved from RL94 m to RL102 m.



(c) Photograph of pond C. (37 m x 30 m x 7.5 m) with slurry clay heaved from RL94.5 m to RL102 m.

(d) Photograph of pond D (42 m x 32 m x 5.5 m) with slurry clay heaved from RL96.5 m to RL102 m.

Figure 13. Slurry clay ponds A, B, C and D.

RATE OF RECLAMATION

Estimation of rate of reclamation as shown in Table 3 is based on the extent of the surface land reclamation records and the extent of the volume of clay slurry ponds as of 29 Nov 2011. It is noted that the reclamation rate by method B is 14 times faster than method A or method C. The two slurry clay ponds B and A show that a vast quantity of earth fill was used to push the clay below the seabed towards the surface.

Table 3. Slurry clay flow due to various reclamation methods.

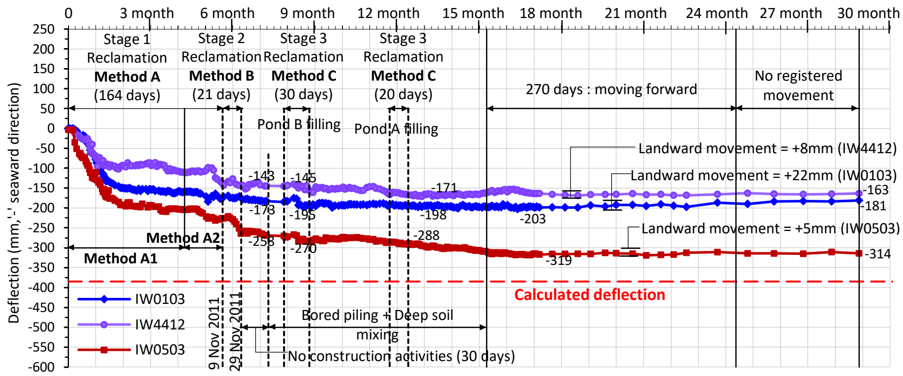
Stages of reclamation fill	Fill method	Approx. rate of reclamation	Period		No of days	Inclinometer deflection	Slurry clay flow
			From	To			
Stage 1	Method A1+A2	0.26 m^3/day	20 May 2011	09 Nov 2011	164	Within design expectation.	Limited slurry clay flow below seabed.
Stage 2	Method B	3.44 m^3/day	10 Nov 2011	29 Nov 2011	21	Sudden plunge.	Massive slurry clay flow below seabed.
Stage 3	Method C	0.22 m^3/day	Pond C			Negligible deflection.	No slurry clay flow below seabed.
			17 Dec 2012	19 Dec 2012	3		
			Pond B				
			16 Jan 2012	13Feb 2012	30		
			Pond A				
			11 May 2012	31 May 2012	20		

DEFLECTION OF SEAWALL BY VARIOUS RECLAMATION METHODS

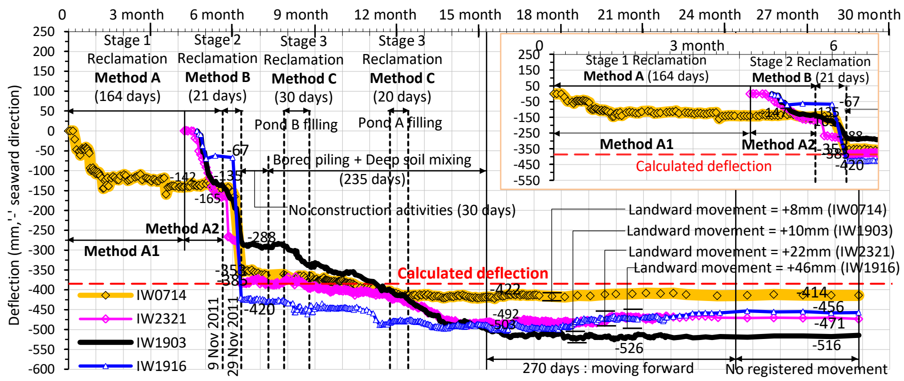
Figure 14 shows the comparison of deflections of the sections 2 and 3 of the seawall. Section 2 is from box no. 1/41 to box no. 6/46, and represented by inclinometers IW0103, IW4412and IW0503, whereas section 3 is from box no. 7/47 to box no. 28 and deflection is from inclinometers IW0714, IW1903, IW1916and IW2321. The influence of the various methods of reclamation is examined. Method A1, A2, and C reclamations were supervised and recorded on site. There was no record of supervision for method B reclamation.

After the reclamation filling by method A1 and A2, the full active pressure wedge was on section 2 and an almost full 44 m active pressure wedge on section 3. At this stage the passive resistance of soft clay was fully mobilized. Deflection of both sections was lesser than the 60% of the calculated value. Method B was in the proximity of section 3 and caused a plunge in 3 inclinometers IW0714, IW1916 and IW2321, with a maximum value of -353 mm or 0.80% of H as shown in Figure 14b. The plunge was caused by the concentration of the generated lateral force of the slurry clay flow beyond the 44 m active wedge, using the constructed seawall as a reaction. Inclinometer IW1916 suggested that this slurry clay force might be higher than the 44 m active wedge force. This was a transient force which dissipated after the creation of the massive slurry clay ponds B and A.

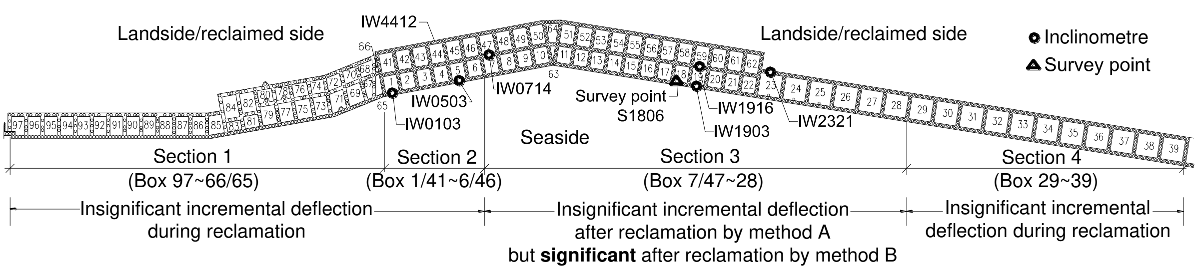
The average deflection of the stringent balancing berm method A2 was -122 mm at section 3 (Rotating height = 40 m to 44 m) and this was lower than the limited balancing berm method A1 with an average deflection of -156 mm at section 2 (Rotating height = 25 m to 40 m). This lower deflection at section 3 showed that the stringent balancing berm method A2 had the minimum seawall deflection, as shown in Table 4.



(a) Deflection of capping beam of seawall for section 2.



(b) Deflection of capping beam of seawall for section 3.



(c) Inclinometers at various sections of seawall.

Figure 14. Comparison of deflection of capping beam of section 2 and 3 of seawall.

Table 4. Deflection by various reclamation methods.

Section	Inclinometer	Rotating height, H*	Incremental deflection				Total deflection (Method: A + B + C)	
			Method A		Method B	Method C		
			A1	A2				
2	IW0103	31.5m	-158mm	-2mm	-12mm	-9mm	-181mm	0.57% of H
	IW4412	37.2m	-109mm	-2mm	-29mm	-10mm	-150mm	0.40% of H
	IW0503	38m	-202mm	-23mm	-35mm	-10mm	-270mm	0.71% of H
3	IW0714	41m	-142mm	-5mm	-206mm	-30mm	-383mm	0.93% of H
	IW1903	44m	-	-135mm	-153mm	-72mm	-360mm	0.82% of H
	IW1916	44m	-	-67mm	**-353mm**	-20mm	-440mm	1.00% of H
	IW2321	44m	-	-165mm	-220mm	-29mm	-414mm	0.94% of H

* H= inclinometer rotating height; (-) Seaward; (+) Landward.

DEFLECTION OF SEAWALL DUE TO CONSTRUCTION WITHIN THE 44-M ACTIVE WEDGE

It was noted that high deflection occurred when construction activities were carried out near the seawall and within the 44 m active wedge as shown in Figure 15, with the incremental magnitude in Table 5. A deflection magnitude varying from -17 mm to -166 mm was recorded.

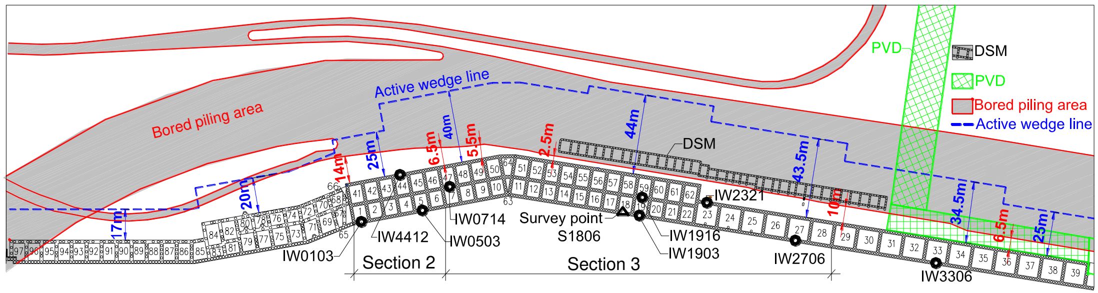


Figure 15. Construction works within 44 m active wedge.

Table 5. Incremental deflection by inclinometer at seawall capping beam by construction activities.

Inclinometer	Rotating height, H*	Perforated vertical drain (PVD)	Deep soil mixing (DSM)	Bored piling (BP)	Total deflection (PVD+DSM+BP)	
IW0103	31.5m	-	-	-17mm	-17mm	0.05% of H
IW4412	37.2m	-	-	-20mm	-20mm	0.05% of H
IW0503	38m	-	-	-36mm	-36mm	0.10% of H
IW0714	41m	-	-	-39mm	-39mm	0.10% of H
IW1903	44m	-	-51mm	-115mm	-166mm	0.38% of H
IW1916	44m	-	-15mm	-48mm	-63mm	0.14% of H
IW2321	44m	-	-19mm	-57mm	-76mm	0.17% of H
IW2706	33.5m	-	-20mm	-55mm	-75mm	0.22% of H
IW3306	35m	-35mm	-	-40mm	-75mm	0.16% of H

* H=Inclinometer rotating height; (-) Seaward; (+) Landward.

DEFLECTION OF SEAWALL DUE TO TIDAL FLUCTUATION

It was observed that the seawall oscillated with an amplitude of ± 8 mm as shown in Figure 16 during tidal fluctuation of 2.55 m. This observation was for a period of approximately 7 months, after which the deflection fluctuation gradually tapered off. A possible explanation of the deflection lies in the mechanism of the very loose partially saturated reclaimed fills. The reclaimed fill was haphazardly and loosely deposited into the enclosed water. While settling under its own weight, slight tension force was created below and higher tension force above the natural water level. The tensile pore pressure will gradually dissipate during each tidal cycle. This would result in higher bulk density and reduction of tension force of the fill within the tidal zone. As the active wedge is 44 m wide, this significant fluctuation in lateral pressure oscillated the wall up to a magnitude of ±8 mm. After a period of seven months this active force came to an equilibrium within the tidal levels and the oscillation movements tapered off.

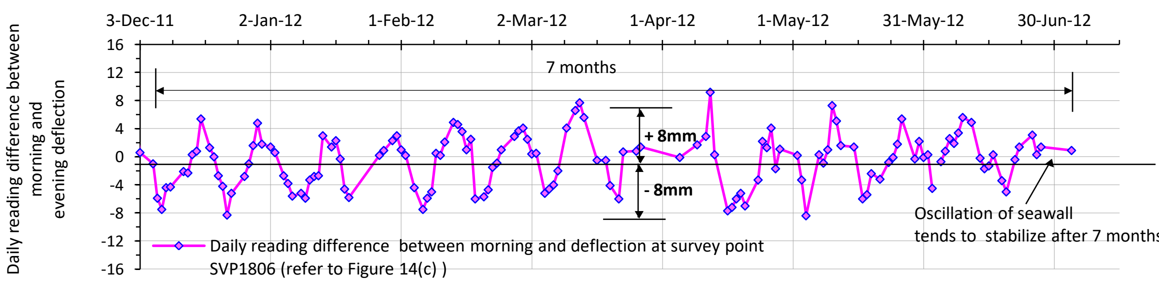


Figure 16. Difference in seawall deflection registered between morning and evening reading for survey point 1806.

STABILITY OF SEAWALL AT DESIGN DEFLECTION

The safety of seawall is examined for its observed maximum deflection of 1.20% of H which is higher than the calculated or design value of 0.89% of H and exceeding the deflection of 0.5% of H specified in BS 8002. At a deflection of 1.20% of H, the passive resistance of the seawall above the rotating height is fully mobilized. Therefore, the seawall is dependent on the structural and geotechnical components or the geo-structural turning back bending capacity for its long term stability as shown in Figure 4. The safety of the geo-structural capacity of the seawall is interpreted from the inclinometer deflection profile.

At the design deflection of -385 mm or 0.89% of H, the structural and geotechnical components of the strong boxes have the following surplus factor of safety, i.e.

1. Bending capacity of pipe piles.
   a. Only 4 front piles are at the limit elastic state of FS = 1.53 as defined by BS5950-1:2000.
   b. Cross, intermediate and back pipe piles are under-stressed with respective factors of safety of 2.06, 2.30 and 2.32.
2. Structural forces of capping beam.
   Maximum structural forces during filling of the box and maximum reclamation height with surcharge loadings are designed in accordance with BS8110-1997.
3. The tensile force of the P-P interlock.
   The tensile force of the P-P interlock maintains the box action of the pipe piles by more efficient distribution of the vertical bending capacity of the pipe piles. Factor of safety is high in the region of 2.5.
4. The concrete plug connects the pipe pile and capping beam as a rigid joint.
   The concrete plug or the reinforcing bars linking the pipe piles to the capping beam is almost equivalent to the cross-sectional area of the pipe piles.
5. The geotechnical depth of embedment into the Old Alluvium.
   The optimum embedment depth of the box into the Old Alluvium maintains its external equilibrium and has the geotechnical factor of safety ranging from 1.7 to 1.8.

Therefore, the geo-structural turning back capacity of the strong box is independent of the passive resistance of the soft clay in front of the seawall and is structurally and geotechnically adequate for higher deflection than 0.5% of H or design deflection of 0.89% of H.

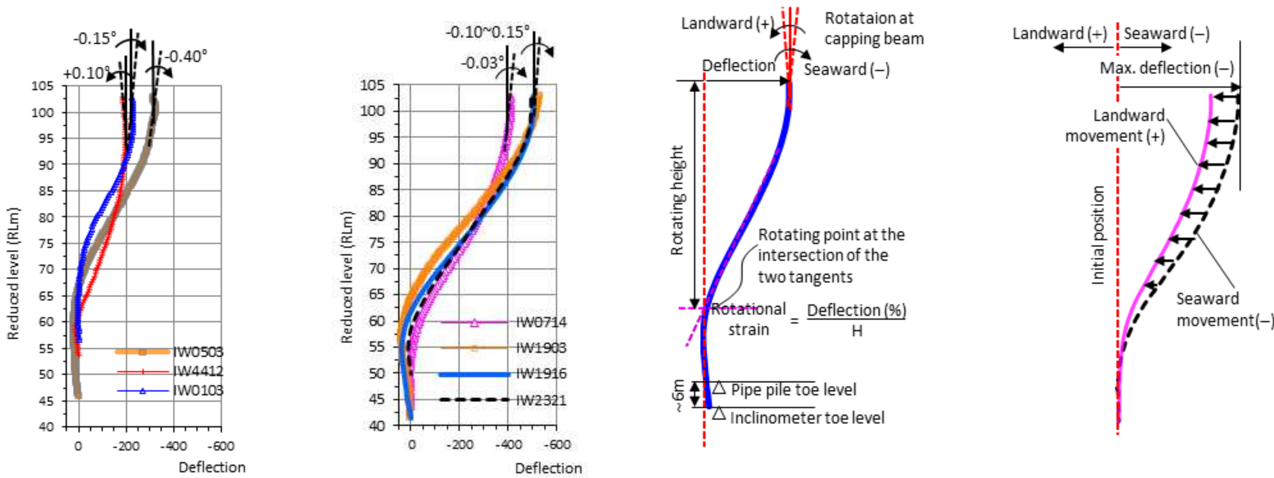
LONG TERM STABILITY OF INCLINOMETER DEFLECTION PROFILES

Based on the inclinometer installed inside the hollow pipe piles, the deflection of the seawall for section 2 and 3 are shown in Figure 17. As the cantilever structure rotates just above the embedment into OA, the rotating point is the intersection of the tangent line into OA and above OA. The rotating height is from the rotating point to top of capping beam. The deflection profile of the inclinometers are shown in Table 6. The long-term stability of the seawall is demonstrated by the inclinometer deflection profiles.

Table 6. Deflection profile of inclinometers (seaward and then landward).

Location	Inclinometer	As-built depth into OA		Rotating height, H**	Rotation of capping beam	Max. deflection of capping beam (seaward)		Landward movement
		Pipe pile	Inclinometer					
Section 2	IW0103	12.2m	15.5m	31.5m	-0.15°	-203mm	0.64% of H	+22mm
	IW4412	7m*	12.2m	37.2m	+0.10°	-171mm	0.46% of H	+8mm
	IW0503	12.9m	18.9m	38m	-0.40°	-319mm	0.84% of H	+5mm
Section 3	IW0714	12m	18.1m	41m	-0.03°	-422mm	1.03% of H	+8mm
	IW1903	12.6m	17.5m	44m	-0.20°	-526mm	1.20% of H	+10mm
	IW1916	12.9m	17.8m	44m	-0.15°	-503mm	1.14% of H	+46mm
	IW2321	12m	9.2m***	44m	-0.10°	-492mm	1.12% of H	+22mm

* Sandstone encountered; ** Inclinometer rotating height; *** Inclinometer toe level was below pipe pile toe level by 6 m except IW2321; Deflection: (+) Landward; (-) Seaward; Rotation: (+) counterclockwise toward landside from vertical; (-) clockwise toward seaside from vertical.



Section 2 inclinometer | *Section 3 inclinometer* | *Rotating height and strain* | *Seaward and then landward movement*

Figure 17. Inclinometer profiles at maximum deflection, seaward and subsequent landward movement.

Depth of embedment into OA by inclinometer readings is higher than design construction of pipe pile embedment. This higher passive resistance of the OA indicates no loss of design passive resistance despite the high seawall movement of 1.20% of H. The bottom of the strong box is rigidly held together. The profile of the inclinometer deflection indicated robustness of the integrated strong box. The robustness was derived from (a) the optimization of the vertical bending capacity of the pipe pies, (b) capping the top of the pipe piles as a box, (c) utilizing the P-P interlock to produce the box action within the rotating height. The deflection profile of the inclinometer was at the elastic state.

The deflection seaward (-ve) was the accumulation of lateral forces from reclamation within the 44 m active wedge, other construction activities within the 44 m active wedge and the creation of the slurry clay pond B and A beyond the 44 m active wedge. The maximum seaward (-ve) of -353 mm or 0.80% of H was caused by the slurry clay pond B and A. After the lateral push of these forces stabilized, the seawall then moved in the reverse direction towards the landside (+ve) as shown in Figure 17. The landward movement was monitored for a period of 13 months and the maximum magnitude is +46 mm as shown in Table 6. The landward movement suggested that the seaward (-ve) deflection of 1.20% of H is still within the elastic state of the geo-structural turning back moment capacity of the strong box. The landward movement was caused by the vertical consolidation of the 44 m active wedge.

CONCLUSION

The building of the seawall is constrained by the 35 m wide construction corridor. Because of its 44 m rotating height, it is necessary to design an integrated strong box to resist the very high active force. This integrated strong box must have the adequate geo-structural turning back capacity above the rotating point for internal equilibrium. Below the rotating point, there must be adequate passive resistance to satisfy external equilibrium.

Despite the intended construction control to enforce the reclamation methods that would cause the limited clay slurry flows below the seabed, extensive clay slurry ponds were unknowingly created during the short period of 21 days after the successful stage 1 reclamation. The slurry clay flows exerted additional force and deflection to the 44 m active wedge of the seawall. This resulted in an additional maximum incremental deflection or plunge of -353 mm or 0.80% of H to the 44 m active wedge deflection of inclinometer IW1916. The additional maximum deflection or plunge of -353 mm or 0.80% of H suggests that the pushing seaward force of the slurry clay flows is higher than the 44 m active wedge pressure.

At the design deflection of -385 mm or 0.89% of H, the structural and geotechnical factors of safety of the strong box have the following redundancies:

a. The bending capacity of the 4 front piles are at the FS=1.53 at only one point along the vertical length, and the intermediate, cross and back pipe piles are understressed.
b. Structural capping beam, P-P interlock and concrete plug are understressed.
c. The geotechnical factor of safety for external equilibrium is ranging between 1.70 to 1.80.

Hence there is redundancy in the turning back capacity of the strong box at the design deflection of -385 mm or 0.89% of H. At a maximum deflection of -526 mm or 1.20% of H, as shown in inclinometer IW1903, the seawall is noted to be stable. This is confirmed by the seawall movement landwards by a magnitude of +46 mm. In general, all the inclinometers move landward (+ve) after attaining the maximum seaward deflection (-ve).

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