A CASE STUDY ON THE DESIGN OF TRANSITION ZONE FOR CEMENT DEEP MIXING FOR A PORT RECLAMATION PROJECT

Patrick K. Wong
Senior Principal, Coffey Geotechnics Pty Ltd, Sydney, Australia

ABSTRACT

This paper describes a large container terminal project constructed in reclaimed land comprising deep soft clay that extends to RL-32 m chart datum. The wharf deck is a piled structure, and a 24 m wide area immediately behind it was constructed in 2008 using a Cement Deep Mixing (CDM) technique to improve the soft soil to provide stability. Post-construction settlement in this area was expected to be less than 35 mm in 20 years. However, the container yard some 30 m behind this area was only treated with prefabricated vertical drains (PVD) and surcharge, and post-construction settlement in this area was expected to be over 300 mm.

The design of the transition zone between the wharf and the container yard to limit differential settlement is described in this paper. In the transition zone, the CDM columns were progressively shortened so that the columns were only partially penetrating to give progressively larger post-construction settlement towards the container yard. The project was complicated by the fact that the surcharge design for the container yard was completed by another consultant and the work carried out by another contractor. In addition, the surface drainage within the area was bi-directional, making the design of the transition zone more challenging. To provide confidence to the client, reliability analyses were carried out to assess confidence levels for the predicted post-construction differential settlement to meet the design objectives.

1 INTRODUCTION

In 2006 to 2007, a reclamation was carried out on a coastal area in south-east Asia for the development of a container terminal which has a design elevation of RL+4.5 m. The subsurface profile at the site comprised over 30 m of soft to firm clay. The client implemented a program of surcharging with PVD for the container yard to limit the post-construction settlement to 300 mm in 20 years, but let the 500 m long wharf construction using a Design and Construct delivery mechanism. The wharf construction contract was awarded in 2007, and the successful contractor elected to use CDM in a 24 m wide zone behind a piled wharf deck structure to provide the necessary stability for a dredging level which varied from RL-5 m at the landward edge of the wharf deck to RL-15.7 m at the seaward edge of the wharf.

The CDM zone immediately behind the wharf was constructed using overlapping 1.6 m diameter columns to form interlocking 24 m wide panels that run normal to the wharf alignment at a spacing of 3.6 m centres, giving an area replacement ratio of 40%. The CDM columns were fully penetrating to RL-35 m into very stiff to hard clay, and the CDM mix provided an average unconfined compressive strength of 1.3 MPa (design strength of 1 MPa). This zone is thus relatively stiff and the post-construction settlement under the design loading of 40 kPa was assessed to be 35 mm. Based on monitoring results of the surcharged container yard, post-construction settlement in 20 years was estimated to be 315 mm.

Therefore, the challenge was to design the transition zone to limit differential settlement to acceptable limits for drainage and pavement performance.

2 DIFFERENTIAL SETTLEMENT DESIGN CRITERIA

The following design criteria were specified by the client with respect to limitations on differential settlement for an applied loading of 40 kPa in the transition area:

- In 20 years, the minimum ground slope shall not be less than 0.7% in order to maintain adequate drainage within the container handling area.
- Within the transition zone, the differential settlement shall not be more than 0.3% change in grade from the general ground slope for satisfactory performance of the pavement.

Another challenge was that there were considerable uncertainties in post-construction settlement predictions particularly for the surcharged PVD area which is beyond the responsibility of the wharf construction contractor, but has direct impact on the differential settlement of the transition area.

3 SUBSURFACE PROFILE AND SOIL PROPERTIES

The soil profile at this site is relatively uniform and comprises a thick soft to firm clay layer that exhibits linearly increasing stiffness and strength with depth. The soft to firm clay is underlain by a medium dense to dense sand followed by a deep sequence of very stiff to hard clays. The adopted parameters for the soft clay layer are summarised in Table 1.

Table 1: Geotechnical Model Adopted

Layer	**Elevation RL (m)**	**Thickness (m)**	**CR = $C_c/(1+e_o)$**	**CRR = $C_r/(1+e_o)$**	**$C_{\alpha\varepsilon}$**	**σ_{vo}' (kPa)**	**OCR**	**S_u (kPa)**
1	+2 to +0	2	0.280	0.045	0.0112	13.22	4.9	11
2	+0 to -3	3	0.469	0.047	0.0188	28.28	3.5	15
3	-3 to -8	5	0.573	0.101	0.0229	43.18	1.9	19
4	-8 to -12	4	0.512	0.091	0.0205	66.69	1.6	25
5	-12 to -18	6	0.807	0.103	0.0323	98.17	1.7	33
6	-18 to -22	4	0.675	0.107	0.0270	122.68	1.8	49
7	-22 to -25	3	0.541	0.116	0.0217	139.16	1.6	56
8	-25 to -28	3	0.490	0.102	0.0196	156.52	1.6	63
9	-28 to -32	4	0.379	0.105	0.0152	174.47	1.5	72

where: C_c = Compression Index
C_r = Recompression Index
e_o = Initial void ratio
$C_{\alpha\varepsilon}$ = Creep strain rate
σ_{vo}' = Initial effective vertical stress
S_u = Undrained shear strength
OCR = over-consolidation ratio

4 ADOPTED SOLUTION

The strategy adopted for the transition zone to meet the differential settlement design criteria was as follows:

- Surcharge the transition zone (5 m surcharge height), which was carried out over a period of only 3 months due to time limitations. Only 0.65 m of settlement was achieved in the surcharged transition area compared to about 3 m in the surcharged PVD area.
- Provide an initial ground slope of 2.1% at the transition zone, sloping down from the edge of the surcharged PVD area to the edge of the wharf CDM area. This slope was chosen on the basis of the maximum slope at which the container handling over-head gantry will be able to operate. It is expected that this slope will reduce with time as the surcharged PVD area will settle more than the transition zone.
- Provide stepped CDM ground improvement in the 30 m wide transition zone to provide a gradual increase of settlement towards the surcharged PVD area.

The CDM in the transition zone comprised twin 1.6 m diameter columns at 4.8 m lateral spacing and 3.5 m longitudinal spacing, giving an area replacement ratio of 22.8%. An average of 8 rows of twin CDM columns was used across the transition area. The adopted strategy in the transition zone is illustrated in Figure 1 below.

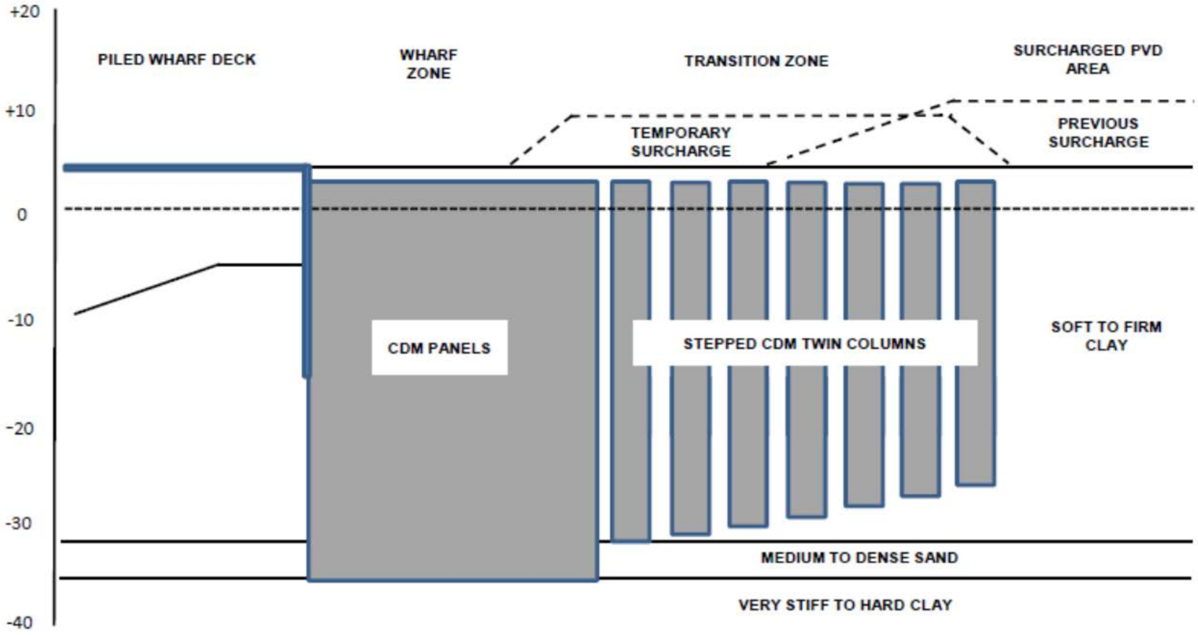

Figure 1: Illustration of Ground Improvement Adopted in Transition Zone.

5 ANALYSIS METHODOLOGY AND RESULTS

The analysis method adopted for the design comprised primarily of simple one-dimensional consolidation analysis based on the method described in SGF (1997). The elastic modulus of the CDM column material was assessed to be 153 MPa with a standard deviation of 28 MPa based on 14 batches of tests (over 80 samples). The soil compressibility values given in Table 1 were converted to constrained modulus values based on initial and final stress levels, and OCR of the soil layers. An equivalent constrained modulus of the soil was then calculated in the CDM treated zone based on the area replacement ratio in accordance with SGF (1997). In the untreated zone below the toe of the stepped CDM columns, the soil compressibility is unchanged. Post-construction creep following preloading was assessed using the method described in Wong (2007).

After the design was approved by the client, numerical analyses were carried out using the commercially available finite element analysis software package PLAXIS. However, this paper will focus on the reliability assessment of the results rather than the analytical and numerical settlement analysis procedures.

By progressively lifting the toes of the twin CDM columns by 0.97 m increments for each row moving landwards, the post-construction settlement in 20 years was estimated to range from about 60mm to 250 mm, with increasing settlement towards the surcharged PVD area. With the initial ground slope set at 2.1%, the estimated settlement will reduce the initial slope to 1.4% in 20 years and this allows for uncertainty in predictions in meeting the minimum gradient of 0.7% for drainage requirements.

The numerical analysis showed that because there is at least 2.5 m of compacted fill over the soft clay, the differential settlement between CDM columns is well within the design limit of 0.3% change from the general ground slope. Geogrid reinforcement in the fill was used to provide additional safety to even out settlement.

6 RELIABILITY ASSESSMENT

To assess the confidence level of the post-construction settlement estimate, a reliability assessment based on the procedure described by Duncan (2000) was carried out. The steps involved in the reliability assessment procedure is summarised briefly as follows:

1) Assess the most likely values (MLV) for each parameter.
2) Using the MLV for all parameters, calculate the most likely settlement estimate S_{MLV}.
3) Assess the standard deviation (SD) of each parameter that involves uncertainty. In the absence of adequate statistical data, the standard deviation may be estimated in two ways (a) by using the three-sigma rule - estimating the highest conceivable value (HCV) and the lowest conceivable value (LCV)

and estimating the standard deviation as (HCV – LCV)/6, and (b) by using published literature on the coefficient of variation (CV) and estimating the standard deviation as CV x MLV.

4) Compute the settlement with each parameter increased by one SD and then decreased by one SD, while maintaining all other parameters at the MLV.

5) Calculate the difference in settlement (ΔS_i) between (MLV + SD) and (MLV – SD) for each of the parameters, and use Taylor's series to calculate the combined standard deviation of S_{MLV} as follows:

$$SD_{MLV} = \sqrt{\left(\frac{\Delta S_1}{2}\right)^2 + \left(\frac{\Delta S_2}{2}\right)^2 + \left(\frac{\Delta S_3}{2}\right)^2 + \ldots\ldots\ldots\ldots} \qquad \text{Eq. [1]}$$

6) Calculate the coefficient of variation of S_{MLV} as $CV_{MLV} = SD_{MLV}/\ S_{MLV}$

7) Assess the probability of the actual settlement being greater than SR x S_{MLV} where SR (settlement ratio) = actual settlement/S_{MLV} using a lognormal reliability index, β_{LN} as follows:

$$\beta_{LN} = \frac{\ln\{(SR)\sqrt{1+CV_{MLV}{}^2}\}}{\sqrt{\ln(1+CV_{MLV}{}^2)}} \qquad \text{Eq. [2]}$$

8) Using the built-in function NORMDIST in Excel, the probability that the settlement ratio SR may be exceeded is {1 – NORMDIST(β_{LN})}

For the row of CDM columns on the side of the PVD area which are to be installed with their toes at about mid-depth of Layer 8, there are 16 parameters that involve uncertainties which will affect the estimated settlement. The adopted MLV, CV, SD, (MLV + SD) and (MLV – SD) for these parameters are presented in Table 2.

Table 2 – Parameters Adopted for Reliability Assessment

Description	**Symbol**	**CV**	**SD**	**MLV**	**MLV+SD**	**MLD – SD**
Applied stress	Δp	0.14	10	70.3	80.3	60.3
No of log time cycles for creep calculation	Ncreep	0.20	0.2	1	1.2	0.8
CDM treated zone	D'eq	0.20	7.3	36.5	43.8	29.2
	H(CDM)	0.02	0.5	28.5	29	28
Untreated bottom 1.5m thickness of Layer 8	CR	0.20	0.0980	0.4900	0.5880	0.3920
	CRR	0.20	0.0204	0.1020	0.1224	0.0816
	Cαε	0.20	0.0039	0.0196	0.0235	0.0157
	σvo'	0.05	8	160.1	168.1	152.1
	σp'	0.20	50.5	252.4	302.9	201.9
	H(clay)	0.00	0	1.5	1.5	1.5
Untreated clay layer 9	CR	0.20	0.0760	0.3790	0.4550	0.3030
	CRR	0.20	0.0210	0.1050	0.1260	0.0840
	Cαε	0.20	0.0030	0.0152	0.0182	0.0122
	σvo'	0.05	8.75	174.5	183.25	165.75
	σp'	0.20	52.8	264	316.8	211.2
	H(clay)	0.10	0.4	4	4.4	3.6

The adopted MLV and SD values were evaluated from an extensive set of testing results, together with back-analysis results of settlement from both the surcharge PVD area and the wharf CDM area.

The results from Step (4) of the reliability assessment procedure are presented in Table 3.

Table 3: Computed Settlement from MLV + SD and MLV – SD for various Parameters

Description	**Symbol**	**Settlement (m)**			
		S(MLV + SD)	**S(MLV - SD)**	**$\Delta S/2$**	**$(\Delta S/2)^2$**
Applied stress	Δp	0.260	0.235	0.012	0.000154
No of log time cycles for creep calculation	Ncreep	0.266	0.230	0.018	0.000325
CDM treated zone Untreated bottom 1.5 m thickness of Layer 8	D'eq	0.238	0.261	0.011	0.000131
	H(CDM)	0.249	0.247	0.001	0.000001
	CR	0.250	0.246	0.002	0.000004
	CRR	0.251	0.244	0.004	0.000014
	$C\alpha\varepsilon$	0.253	0.242	0.006	0.000035
	$\sigma vo'$	0.254	0.240	0.007	0.000049
	$\sigma p'$	0.243	0.304	0.031	0.000939
	H_c	0.248	0.248	0.000	0.000000
Untreated clay layer 9	CR	0.253	0.242	0.005	0.000030
	CRR	0.257	0.238	0.009	0.000086
	$C\alpha\varepsilon$	0.260	0.235	0.012	0.000148
	$\sigma vo'$	0.261	0.234	0.013	0.000173
	$\sigma p'$	0.236	0.354	0.059	0.003475
	H_c	0.261	0.234	0.013	0.000180
				Standard Deviation	**0.076**

The combined SD on S_{MLV} calculated using Equation 2 is 0.076 m (i.e. 76 mm), giving a coefficient of variation CV_{MLV} of 76/250 = 0.3 (or 30%).

From Steps 7 and 8 of the reliability assessment procedure, the probability of exceeding a particular multiple of S_{MLV} (i.e. Settlement Ratio, SR) has been computed and shown in Figure 2 below:

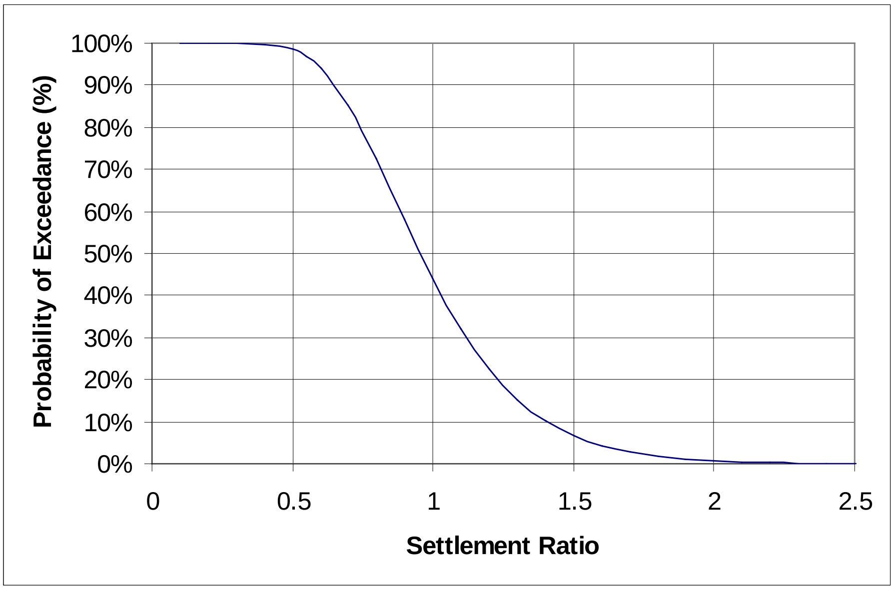

Figure 2: Computed Probability Distribution of Settlement Estimate

For ground improvement design to limit settlement and differential settlement of civil infrastructures, a confidence level of 95% is generally considered to be a stringent design requirement. From Graph 1, it can be seen that there is only a 5% probability that the actual settlement would exceed the most likely estimate of 250 mm by more than 1.6 times.

Assuming that the wharf CDM treated area would settle 60 mm as estimated, and the settlement could be as much as 400 mm (i.e. 1.6 x 250) in 20 years, the ground slope would reduce from the initial slope of 2.1% to about 1% which meets the specified minimum slope of 0.7% for drainage. Even if the wharf CDM treated area does not settle, a minimum slope of 0.7% in 20 years would be met.

7 POST-CONSTRUCTION PERFORMANCE

Unfortunately, the author was unable to obtain any monitoring data from our client on this project post-construction. However, verbal information from our client is that the container terminal is performing satisfactorily to date. Plate 1 shows the pavement condition during operation of the container terminal.

Plate 1

Finished pavement surface during container handling operation

8 CONCLUSION

Different ground stiffness between the main container storage yard and the wharf area of this container terminal project was caused by different ground treatment adopted. The main container storage yard was treated using surcharge with PVD while the wharf area was treated using CDM with fully penetrating columns to RL-35m. This situation presented a significant challenge in the design of the transition zone between these two areas to meet the differential settlement criteria for serviceability of the container handling equipment and surface drainage.

A 30 m wide stepped CDM zone together with setting the initial ground slope upwards to the landside provided a satisfactory solution to the challenge. The use of the simple reliability assessment procedure described by Duncan (2000) provided a useful quantification of possible uncertainties, and provided confidence to the client that the adopted solution is sound and has enough built-in safety to cater for potential uncertainties in material properties and design assumptions.

9 REFERENCES

Duncan, J.M. (2000) Factors of Safety and Reliability in Geotechnical Engineering, Jnl of Geotechnical and Geoenvironmental Engineering, April 2000.

SGF Report 4:95E (1997) Lime and Lime Cement Columns, Guide for Project Planning, Construction and Inspection, Swedish Geotechnical Society, February 1997.

Wong (2007) Preload Design to Reduce Post-construction Creep Settlement ANZ Geomechanics Soft Soil Workshop (Brisbane, 25 October 2007).