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Failure of Embankment on Soil-Cement Columns for Thi Vai Port, Vietnam

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FAILURE OF EMBANKMENT ON SOIL-CEMENT COLUMNS FOR THI VAI PORT, VIETNAM

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ABSTRACT

The Thi Vai Container Port is constructed on reclaimed ground along the Thi Vai River in the Mekong delta approximately 90 km southeast of Ho Chi Minh City, Vietnam. The soil profile consists of an about 15 to 23 m thick deposit of soft, normally consolidated, highly compressible clay deposited on dense to compact sand. A soil improvement scheme was instigated aiming to reduce long-term settlement after construction of the facilities and improve the stability of the river bank. The scheme combined wick drains and, along the river bank, soil cement columns and toe revetments. The wick drains were installed at a spacing of about 1.5 m and a staged surcharge was placed to a maximum height of 6 through 6.6 m to bring about the consolidation of the clay. After a surcharge height of 4.7 m had been in place for about three months and the measured settlement was about 1.2 m, a slope failure occurred along about 200 m length of the riverbank. An investigation indicated that the three-month consolidation period had not increased clay undrained shear strength as anticipated and that the slope failure had broken the soil cement columns at about 11 m depth below the original ground surface. Costs to remedy the collapsed and damaged area amounted to about US$10 million. The paper presents the background information, soil failure details, results of bank stability analyses, and the solution chosen for the remedial construction.

INTRODUCTION

The Thi Vai Container Port is built over a 470 m by 600 m area along the bank of Thi Vai River in Mekong delta approximately 90 km southeast of Ho Chi Minh City, Vietnam. The soil profile consists of deltaic sediments of about 15 to 23 m of soft, normally consolidated, highly compressible clay on a thick layer of dense to compact sand. The highest water level is at Elev. +4.0 m. To raise the area above high water level, the area need to be raised to Elev.+5.0 m. In order to accelerate the ensuing consolidation and reduce post-construction settlement, wick drains were installed through the clay to the sand and additional about 3.3 m to 5.0 m of fill was placed to a surcharge elevations ranging from Elevs.+8.3 m and +9.9 m. Moreover, to reduce long-term settlement and improve the stability for the 600 m long river bank, before placing the surcharge fill, the bank was strengthened by constructing soil-cement columns combined with wick drains.

On March 29, 2010, when the final surcharge level was being approached, some lateral displacements were noticed to have occurred, and, on April 5, 2010, cracks appeared on the fill surface about 30 m from the bank along about 100 m length. The cracks are shown on the photograph in Figure 1. In the morning of July 12, 2010, the width of the crack noticeably and progressively increased until, at 07:50h, the river bank failed along an about a 200 m long stretch. Figure 2 shows a photograph of the failure. A significant crack developed parallel to the river about 30 m inland, extending about 400 m along the river bank. All soil-cement columns along that length broke about 11 m below the fill surface. Figure 3 shows an artist's view of the future Port with the failed area marked out.

This paper describes details of preloading and the area of the slope failure, the field measurements and investigations, bank stability analyses, and discusses the solution chosen for the remediation work. The paper compares the results of laboratory tests to in-situ measurements and results of field tests performed before the fill was placed to similar tests performed after the slope had failed. Costs to remedy the collapsed and damaged area (about 9,120 m^2) along the 600 m long river bank amounted to about US$10 million.

Fig. 1 Downstream view of the first cracks that appeared along the riverbank on April 5, 2010

Fig. 2 *Slope failure on July 12, 2010, viewed upstream*

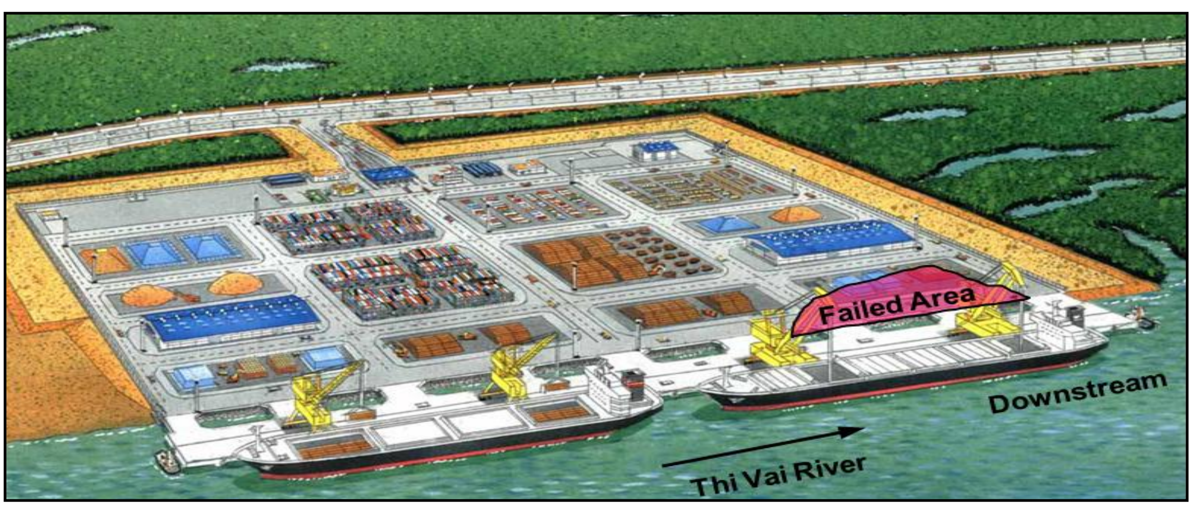

Fig. 3 Artist's view of completed Port with slope failure area overlaid (JICA 2006)

SOIL PROFILE

The soil profile is illustrated in Figures 4 and 5. Figure 4 shows the results of a typical CPTU sounding pushed at the site before construction start. Figure 5 shows the distribution of the basic soil parameters. The natural water content of the clay is 70 to 75 % and the total saturated density is about 1,500 kg/m^3. The density of the sand below the clay is estimated to 1,800 kg/m^3. The field vane shows the clay to be very soft above 10 m depth and soft below. The correlation coefficient, NK, between CPTU cone stress and vane shear stress is about 18.

The groundwater table level varies with tide conditions and seasonally. The average groundwater table lies at the ground surface, Elev. +4.0 m. Pore pressure measurements at 12 and 15 m depths indicate an upward gradient with a hydrostatic distribution from Elev. +5.80 m, 1.8 m above the ground surface.

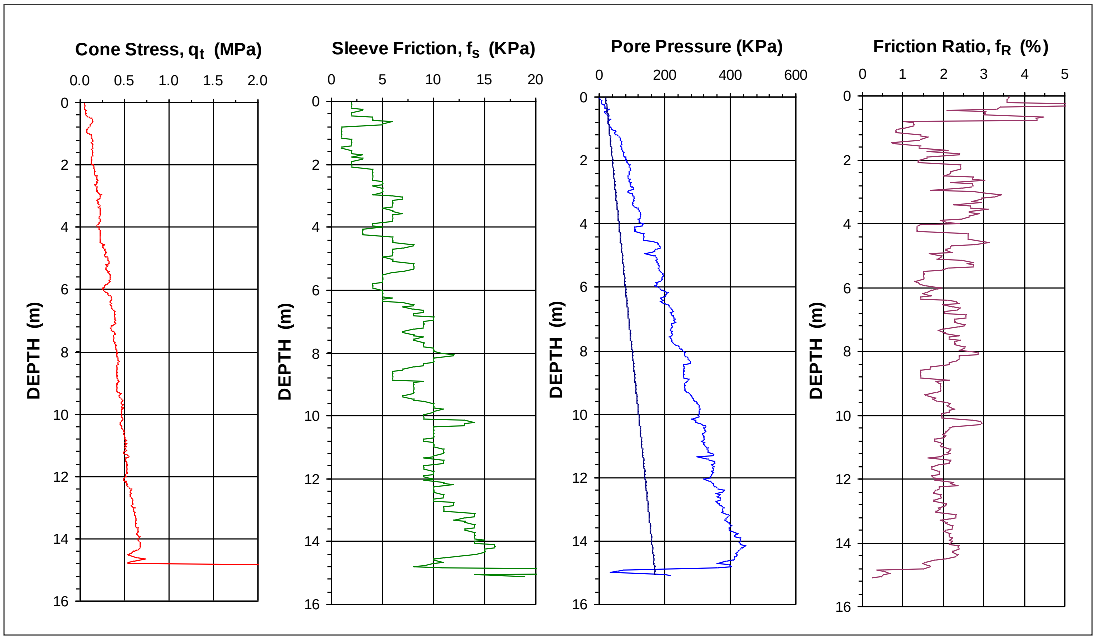

Fig. 4 Diagram of CPTU sounding pushed before construction start

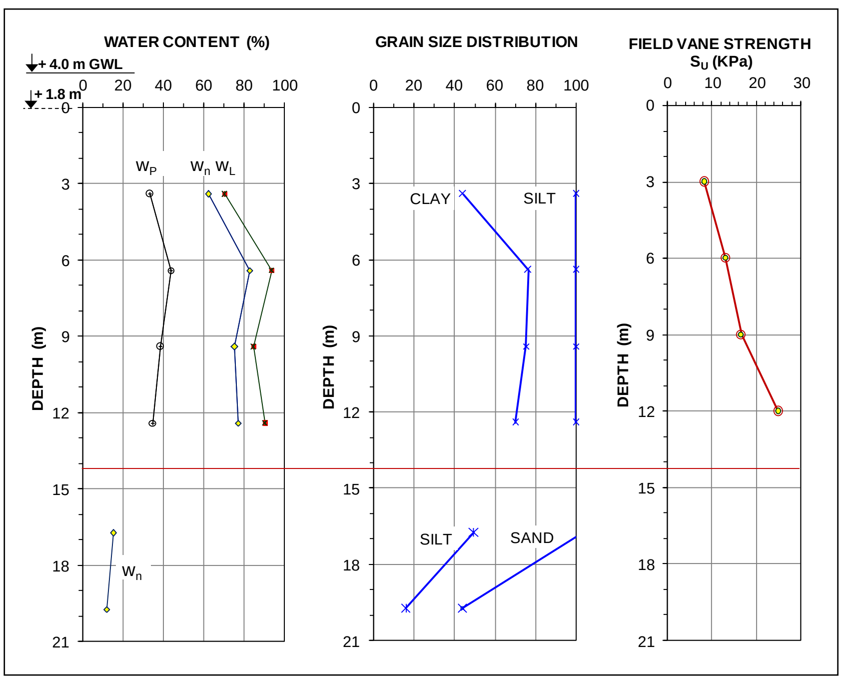

Fig. 5 Diagram of water content and Atterberg Limits, grain size distribution, and field vane strength

EMBANKMENT DESIGN AND SEQUENCE OF EVENTS

The wick drain and lime-cement columns ground-improvement solution was designed according to Technical Standards and Commentaries for Port and Harbor Facilities in Japan (TSCPHF 2002). The stability evaluation of the riverbank after site improvement applied two failure conditions: translational and rotational sliding. The safety factors applied to the short-term and long-term stability analyses were 1.1 and 1.3, respectively.

Figure 6a shows the principle of translational slope stability as applied in the design. The failure mode is based on the horizontal load equilibrium of active and passive earth pressures acting on the side boundaries of the improved area and the shear strength mobilized at the bottom of the improved area of width B. The shaded area is the soil-cement column and wick drain treated ground. The labels WE indicates the weight of the embankment above the treated ground. The labels F_{pS} and F_{aS} stand for passive and active earth stress, respectively acting on the treated ground, and F_{aE} is the active earth stress from the embankment fill.

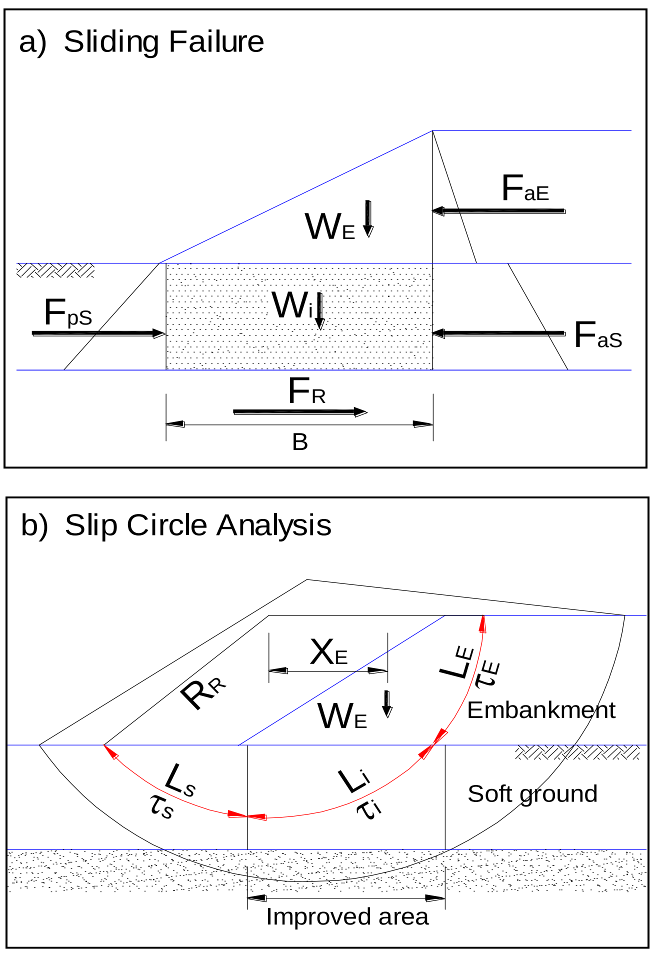

Fig. 6 Typical modes of stability analysis for embankment on soil-cement columns (Technical Standards and Commentaries for Port and Harbor Facilities in Japan 2002)

Figure 6b shows the principle of a rotational cylinder—slip-circle—slide failure (TSCPHF 2002) as applied in the design. The improved ground is assumed to be a composite material with an average and equal shear strength along the slip circle arc. The labels L_E, L_i, and L_s indicate length of circular arc in embankment, improved and original soft ground, respectively. The τ_E, τ_i and τ_S indicate shear strength of embankment, improved and original soft ground, respectively. W_E is the weight of the embankment and X_E is horizontal distance of embankment from center of slip circle. The R_R is the radius of the slip circle. The more shallow slip circle assumes linear increase in undrained shear strength of the soft ground with depth. The deeper slip circle assumes that the undrained shear strength is constant in the soft clay.

Figure 7 shows a section of the river bank with the treated ground before slope failure. The soil-cement columns were constructed through the soft clay using the wet deep mixing method designed to have an unconfined compressive strength of 500 KPa and, therefore, an undrained shear strength of 250 KPa. The column diameter was 1,300 mm. One group of columns was constructed with each overlapping the next by 0.1 m. A second group was constructed as similarly overlapping pairs with open spaces between pairs of 1.3 m, 1.9 m, and 2.6 m. The shear strength of the original soft soil was not considered to contribute to the stability. For use in the stability analysis, the average shear strength of improved ground was estimated to be 200 KPa. About 26 m^2 of 50 to 70 mm stone and 21 m^2 of core stones with weight in range of 10 to 50 kg were placed on the soil-cement columns to form a revetment for protecting the toe of the slope along the river bank from erosion (total area of about 47 m^2).

Wick drains were installed behind the soil-cement columns at a 1.5 m spacing through the clay and into the surface of the sand layer. The wick drains were not installed in the soil-cement column area as it was expected that the soil-cement columns would act as vertical drains.

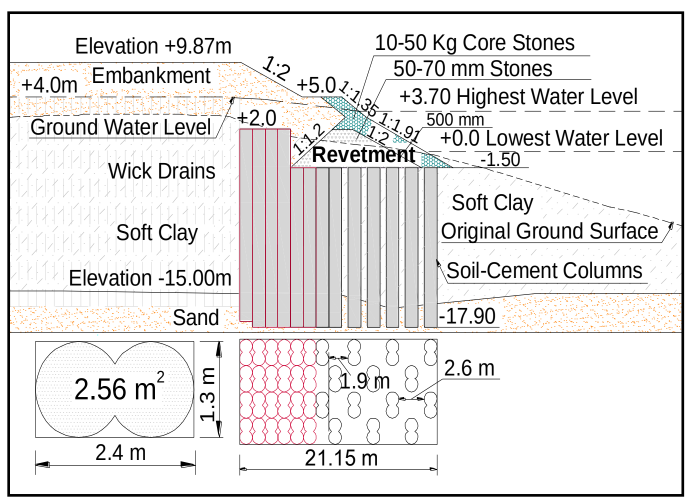

Fig. 7 Typical Cross Section of Embankment on Soil-Cement Columns

The design assumed that a consolidation ratio of 80 % would be reached within 12 months and the settlement at that time would amount to 1.65 m. The fill in excess of the final grade would be removed, and remaining settlement from the surface at Elev.+5.0 m would be limited to secondary compression.

At locations indicated in Figure 8, before start of placing fill on the ground, settlement monitoring plates, SS-plates (SS-1, SS-2, SS-3, SS-30, SS-31, and SS-32) were installed on the original ground surface. Two piezometers (P1 and P2) were installed for measuring pore pressure in two locations at depths of 6.5 m and 14 m, and 6.5 m and 17 m, respectively. Two extensometer gages (E1 and E2) were installed at the same two locations for measuring settlements occurring below depths of 0.2 m, 6.6 m, and 10.0 m, and 14.0 m and 20 m, respectively. Lateral displacement was measured by one inclinometer (I-2) installed to 28 m depth.

After the original ground surface had been raised from its original elevation at Elev. +2.7 m, to the final level at Elev.+5.0 m, the soil-cement columns and toe revetment were constructed. The surcharged area along the riverbank was divided into three parts: Area I-1 and I-2, where placing fill started on January 30 and February 8, 2010, respectively, after building temporary dikes along riverbank about 10 m away from the each area. The purpose of the dikes was to divert the water originating from the fill as it was imported by hydraulic pumping from barges. The surcharge fill in Areas I-1 and I-2 was placed in a total of 12 to 13 lifts each about 0.5 m high to Elevs.+8.3 m and 9.9 m, respectively. The first readings of SS 1, SS 2, SS 3, SS 30, SS 31, and SS 32 were taken on October 28, September 14, August 29, November 23, September 14, and November 7, 2009, respectively.

When on April 5, 2010, cracks appeared on the fill surface, the soil was unloaded by removing about 0.5 m of the fill over an area of 20 by 30 m and later, on May 7, 2010, about 1.0 m of the fill was removed from an 80 by 30 m strip in Area I 1, as delineated in the figure.

Placing fill in the cracked area was resumed on July 2, 2010, when the pore pressure measurements indicated reducing trend. On June 6 and 11, 2010, when the surcharge elevation was at Elev. +7.20 m to +7.62 m, a gradual crack widening trend was noticed. On July 12, 2010, at 20:40h, the slope toward the river failed.

SLIDE INVESTIGATION

After the failure, the shear strength of the soil was investigated. The investigation included cone penetrometer soundings, CPTU, and boreholes at locations shown in Figure 9. No new FVTs were included. Surveying observations indicated that toe revetment material and surcharge fill had moved about 70 m out into the river.

Figure 10 indicates the slide surface starting about 30 m from the river bank and sloping down at 1(V):4(H) toward the soil-cement columns at a depth of about 11 m below the original ground surface, breaking the columns. The lowest location of sliding surface was at Elev. 3.4 m, and the fill and ground surface after slope failure was lower than groundwater level (Elev. +4.0 m). The columns failed along an approximately horizontal plane, which suggests that the type of failure was by translational sliding and wedge.

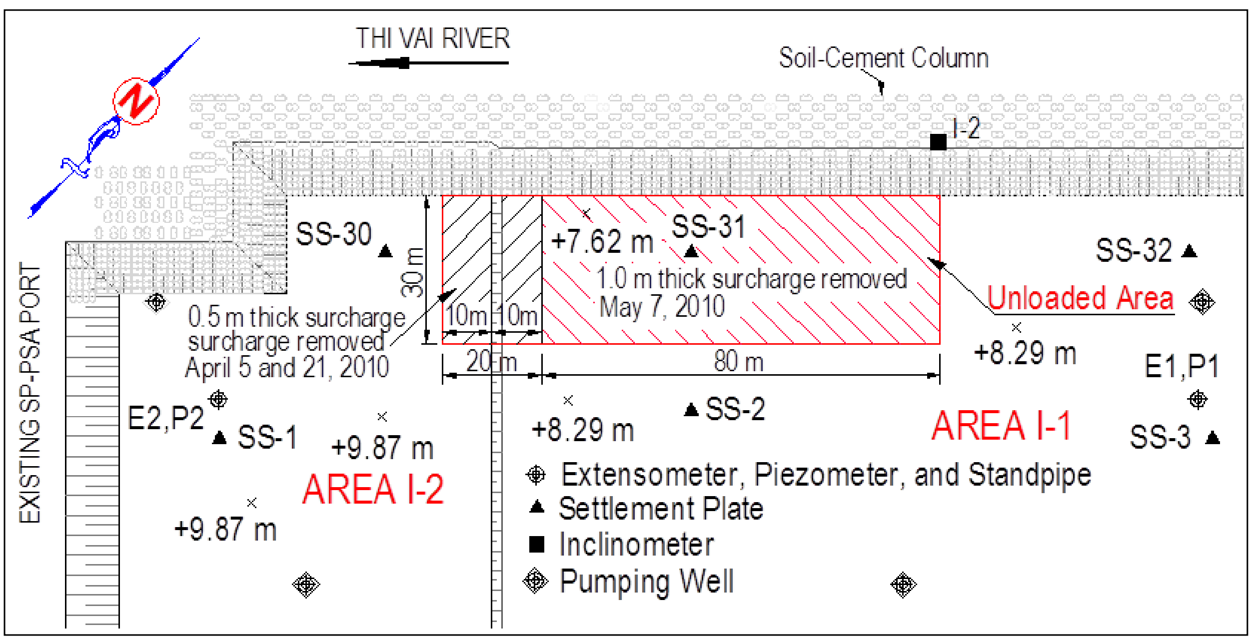

Fig. 9 Locations of boreholes and CPTUs in the failure

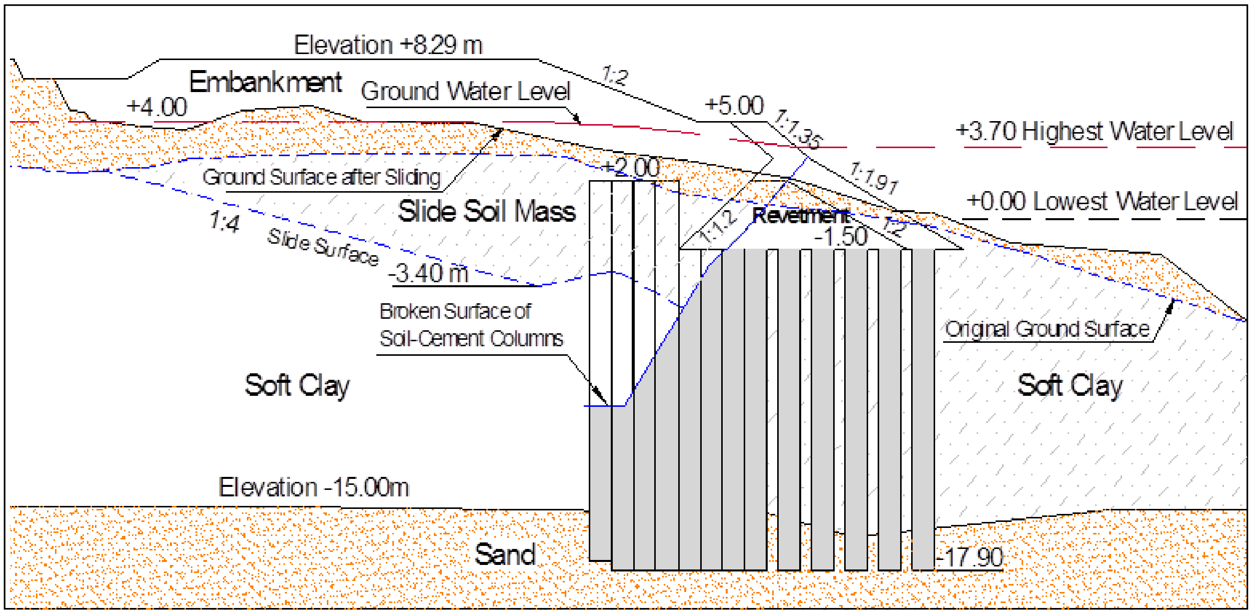

Fig. 10 Cross section of failed embankment

Figure 11 indicates the distribution of settlement versus depth at monitoring stations SS 1, P2 at Area I 1 and SS 3, P1 at Area I 2 just outside the failure area. The records were taken at the occasion of the completion of the placing of each, approximately equal, fill lift at the monitoring point. The recorded settlements are also indicated by the figure showing the records at the gradually increasing depth of each particular settlement anchor point.

The settlement readings in extensometer stations E1 and E2 in Areas I-1 and I-2 started on October 21, 2009 and February 2, 2010, respectively. The final set of readings (the red curve) is from July 12, 2010, the day of the slope failure. The records show increasing settlement between March 23 through May 21, 2010, in Area I-1, and March 30 through May 29, 2010, in Area I-2 respectively. This increase coincided with lateral displacements observed in inclinometer measurements taken during the surcharge lift to Elev.+8.1 m in Area I 1 and to Elev.+8.9 m in Area I 2.

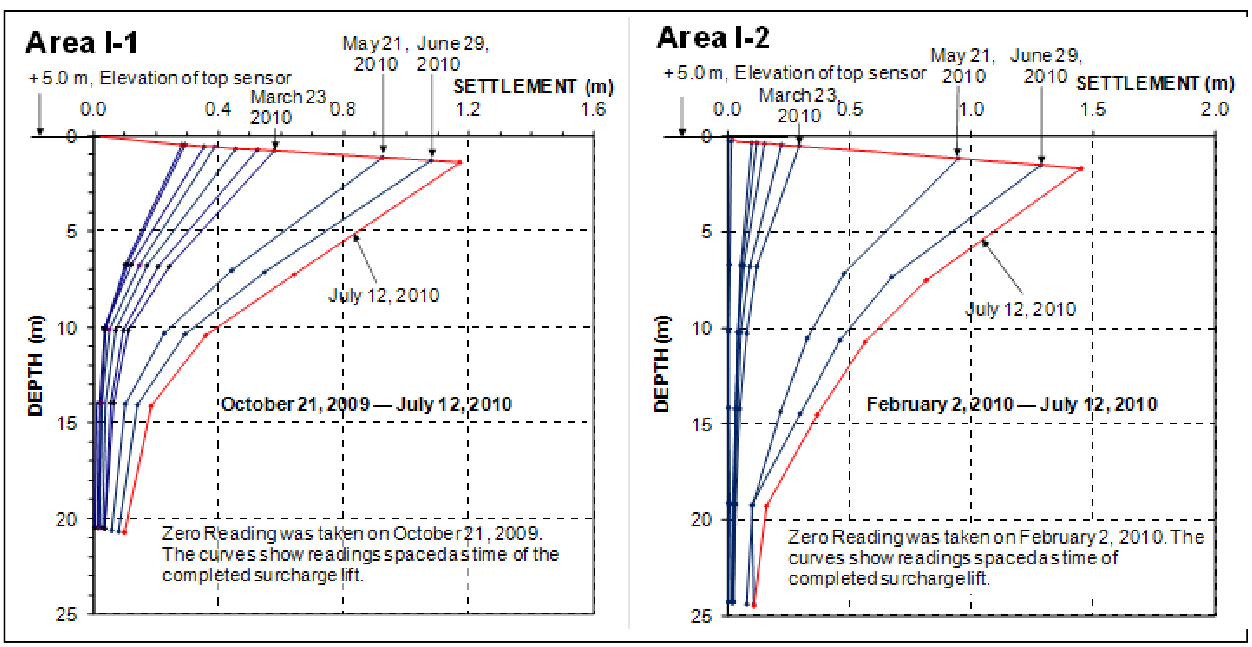

Fig. 11 Distributions of settlement with depth below original ground surface at Areas I-1 and I-2

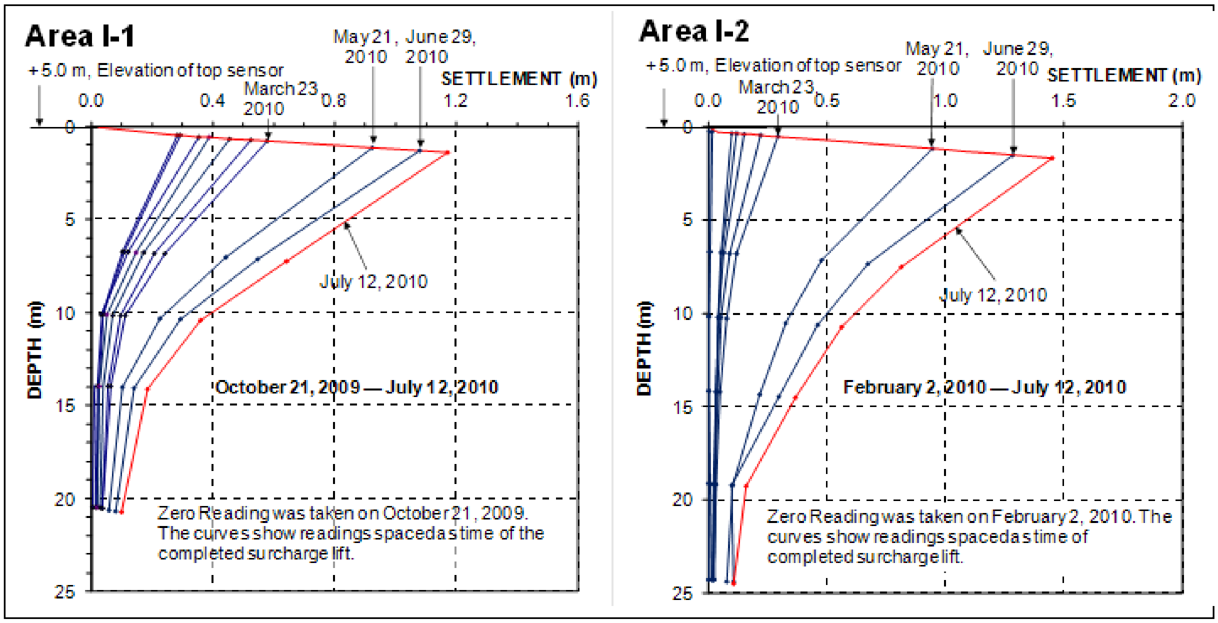

Fig. 11 Distributions of settlement with depth below original ground surface at Areas I-1 and I-2

Figure 12 shows the measured settlement as a function of the fill height at the failure area, Areas I 1 and I 2. Stations SS-30 and SS-31 are inside the failure area. The settlements were monitored from August 29, 2009 through July 12, 2010, and October 28, 2009 through July 12, 2010, respectively. The graphs indicate that when the fill height of SS 30 in Area I 2 was increased to about 10.6 m (to Elev.+9.9 m), the settlement measured was smaller than that measured at SS 31, where the fill height was about 5.5 m (to Elev.+6.6 m).

The figure shows that at Station SS 31, where the about 1 m of fill was removed (May 7 through July 2), the settlements continued to increase, which is considered to be a consequence of the fact that the soil mass below SS-31 was moving laterally toward the river.

Figure 13 shows the measurements of pore water pressure at piezometers P1 (Area I 1) at Elevs. 1.5 m and -9.0 m, and at piezometer P2 (Area I 2) at Elev. 1.5 m and -12.0 m, both immediately outside the failure zone. The pore water pressures were monitored from October 21, 2009 through July 12, 2010, and February 2, 2010 through July 12, 2010, respectively (placing of fill started on January 30 and February 8, 2010, respectively). The dashed horizontal lines are the zero phreatic pore pressures at the indicated elevations.

The measurements show the pore pressures to rise as the placing of fill commenced. However, after about April 6, 2010, and March 4, 2010, in Areas I-1 and I-2, respectively, no further increase of pore pressure was measured. The measurements indicated excess pore pressures elevations at piezometer tip depth Elev.-1.5 m were at Elevs.+9 m to Elev.+13 m about 4 to 8 m above the original pore pressure phreatic height at that depth. The maximum phreatic elevations for the deeper down piezometers, P1 at Elev.-9 m

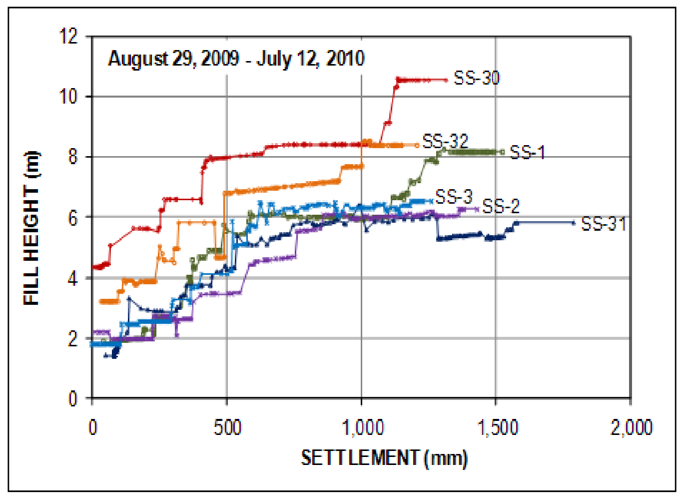

Fig. 12 Fill height vs. settlement

and P2 at Elev.-12 m, were Elev.+8 m and Elev.+11 m, respectively, about 3 to 6 m above the original pore pressure phreatic height at the piezometer tip depths. The excess phreatic heights correspond to a range of excess pore pressure of about 30 through 80 KPa. In comparison, the increase of total stress due the fill was about 150 KPa. It was expected that the wick drains and soil-cement columns wold be effective in dissipating the increase of pore pressure due to the placing of the fill. However, it is likely that the horizontal shear movements developed pore pressures which counteracted the dissipation from the consolidation.

The variation of measured pore pressure makes it difficult to use the pore pressures in assessing the consolidation progress along the shore line. It is unfortunate that the construction control included this few piezometers.

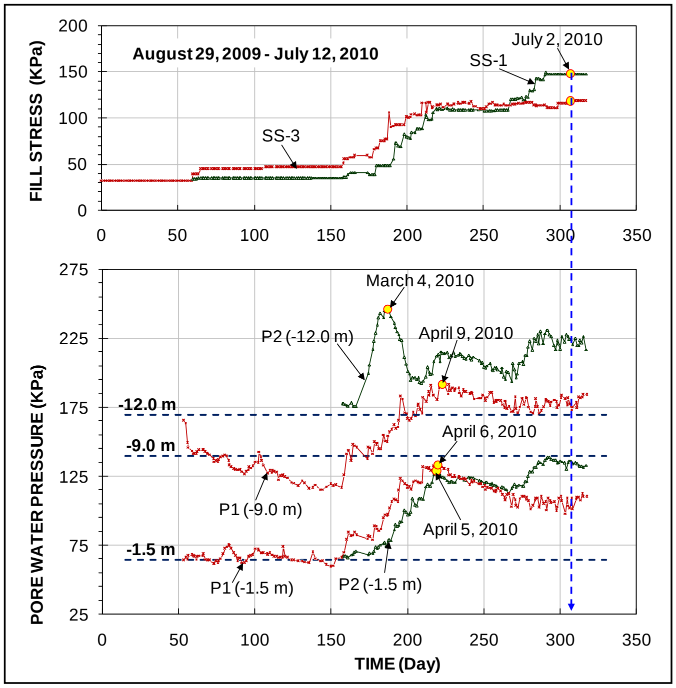

Fig. 13 Measured pore water pressure vs. applied stress

Figure 14 shows fill height versus horizontal displacement at all relevant monitoring stations. When the fill height at Area I-1 (SS-2, SS-3, SS-31, and SS-32) reached a height of about 5 m to about Elev.+7.30 m, corresponding to a stress increase of 100 KPa, the settlement increased significantly. In Area I-2 (SS 1 and SS 30), the similar increase occurred at a fill height of about 8 m (at about Elev.+8.0 m; stress increase of 150 KPa).

Figure 15 shows the horizontal displacement versus settlement obtained from inclinometer measurements. Until May 5, 2010, the horizontal displacements and settlements were about equal. However, thereafter, the horizontal displacement became about 2 to 3 times larger than the settlement. The dashed red lines in the figure show the average slopes of displacement to settlement of about 0.8 and 2.8, respectively.

Figure 16 presents the measurements of horizontal displacement versus depth from October 10, 2009, through July 12, 2010, at the two inclinometer stations. The blue and dark green curves show the readings after completion of each surcharge lift at Areas I 1 and I-2. The lines connecting the top of each curve shows the fill surface level below the Elev.+5.0 m line on the date of the measurements. The measurements show that the onset of the sliding occurred after March 30, 2010, and that translation soil mass movement dominated down to Elev. 3.0 m, about 5 m below the original ground surface (Elevs.+2 to +3 m) and 8 m below the fill surface, with shear zone movements below and to Elev.-12.5 m, about 15 m below the original ground surface. The key zone for the analysis of the slope failure is at about Elev.-3 m, where soil shear can be assumed as fully mobilized by the slide.

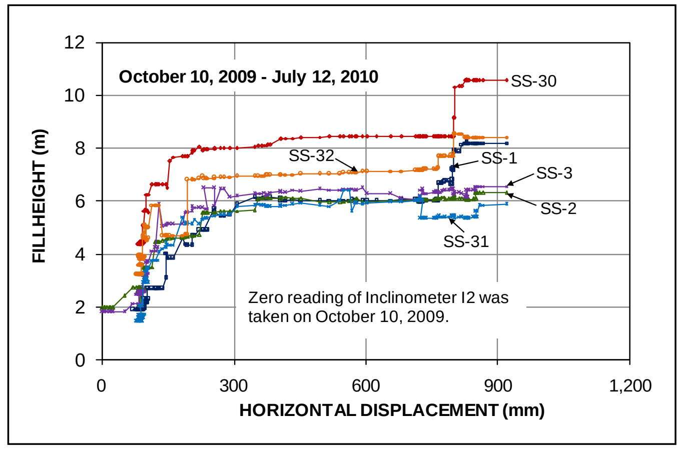

Fig. 14 Fill height vs. horizontal displacement

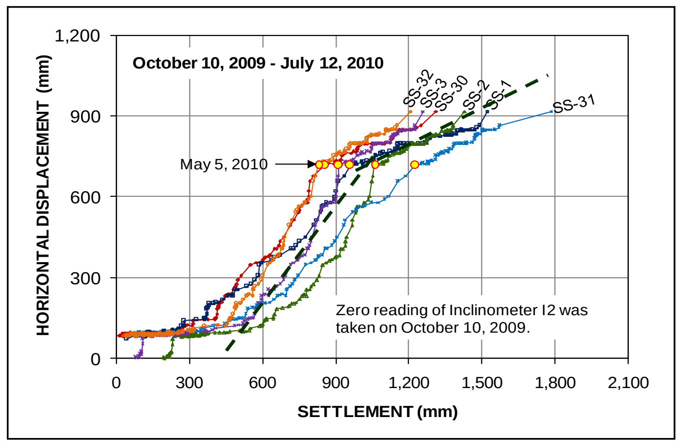

Fig. 15 Settlement vs. horizontal displacement

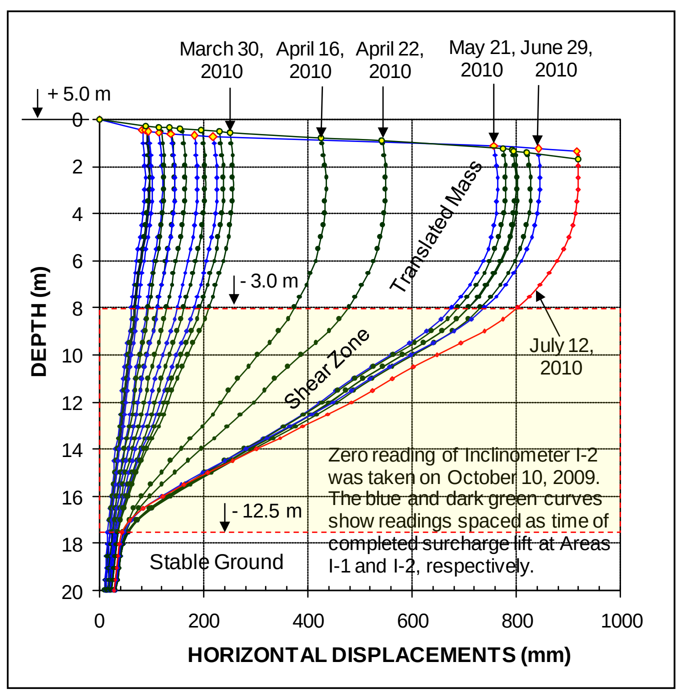

Fig. 16 Horizontal displacement versus depth

Figure 17 indicates a comparison between the distributions of cone stress from the CPTU soundings performed before and after the slope failure (correlated to elevation), suggesting little or no change between the cone stress for before and after the slide. An increase of shear strength would have resulted in an increase of cone stress. Instead, the cone stress from Area I-2 below Elev.-3 m even showed a decrease for the sounding after the failure.

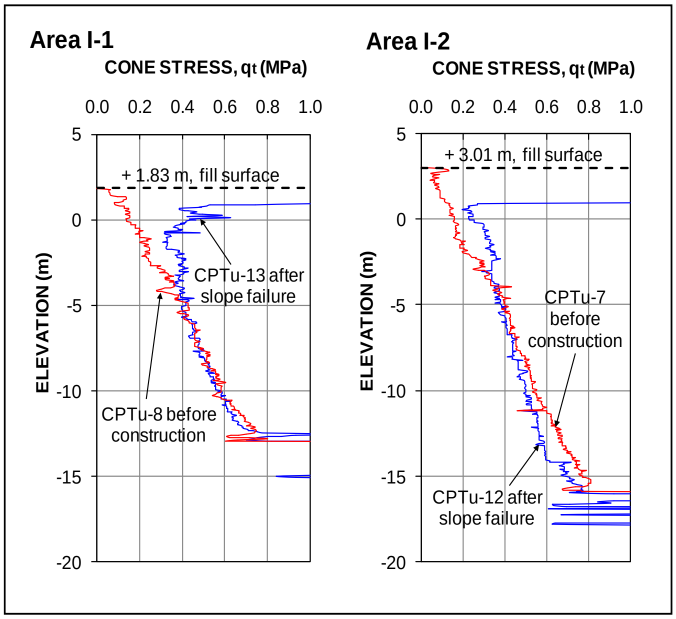

Fig. 17 Cone stresses, q_t, versus depth in Area I-1 and I-2

RIVER BANK STABILITY ANALYSIS

The design stage slope stability analyses of revetment along the Thi Vai River Bank (TSCPHF 2002) assumed the lowest water level in the Thi Vai River to be at Elev.+ 0.6 m, and the fill height to be at Elev.+10.6 m, imposing a stress of 140 KPa. The design was total stress analysis applying undrained shear strength of 15 KPa and that this value would increase during the consolidation. The unconfined compressive strength of the soil-cement columns, 500 KPa, was included in the analysis. The calculations resulted in a factor of safety of 1.20 and 1.27 for translational and rotational slide analysis, respectively, at the end of construction.

However, at the time of slope failure, the low-tide water level in the Thi Vai River was at Elev.-0.2 m and the actual fill stress at Area I-1 and I-2 were about 120 KPa and 150 KPa, respectively.

New stability analyses were carried out for the conditions existing just before the slide. The analyses ignored the contribution of strength of the soil-cement columns and the soil strength was assumed not to have increased beyond the original strength. The analyses showed that the actual safety factor was about 0.8. In hindsight, the slope failure was quite obvious.

SELECTED REMEDIAL SOLUTION

Because of the instability of the shoreline demonstrated by the slope failure and stability analyses, a scheme of remedial construction for the shore line became necessary. It was decided to carry out the following remedial construction.

1. Constructing a piled deck platform along the shoreline
2. Lowering the revetment slope from 1(V):2(H) to 1(V):4(H)
3. Constructing a series of 1.3 m diameter soil-cement columns (called the Advanced Low Improvement Cement Columns, ALICC) behind the damaged soil-cement columns

Figure 18 shows the layout in plan of the remedial area. The soil-cement columns were constructed as overlapping pairs and the free distance between the pairs is 1.5 m. To reduce the differential settlement in the improved area, a 1.5 m thick soil-cement layer was placed directly on the column heads as a precautionary solution. The cement columns were constructed to the sand layer at about 20 m below the deck surface (Elev.-15 m), as shown in Figure 19. The unconfined compression strength of the columns was determined to be 600 to 800 KPa, which was considered satisfactory for the deck loads. The average shear strength of the cement-column reinforced clay was assumed to be 70 KPa.

Stability analyses of the remedial design indicated that the area and the deck would be stable for a surcharge fill behind the constructed ALICC columns to a height of 6.6 m. Settlement analyses indicated that over a period of 20 years the settlement would be smaller than 300 mm.

CONCLUSIONS

The case history presented on the failure of the soil-cement columns reinforced shore line at the Thi Vai Port is an example of soil improvement construction, which ordinarily would be carried out in accordance with a well planned and executed observational method. The following summary conclusions are presented.

1. The average settlement at the slide area, Areas I-1 and I-2, measured over a the about 9 months of placing fill was about 1.4 m. Consolidation analysis indicated that about half of calculated soil consolidation settlement had developed when the slope failure occurred on July 12, 2010.
2. The inclinometer measurements indicated that the slide involved translation movement above Elev. 3.0 m and a shear zone below. The increase of horizontal movements which occurred when the fill was raised to Elev.+8.0 m in Area I-1 and Elev.+8.9 m in Area I-2, coincided with increased settlements.

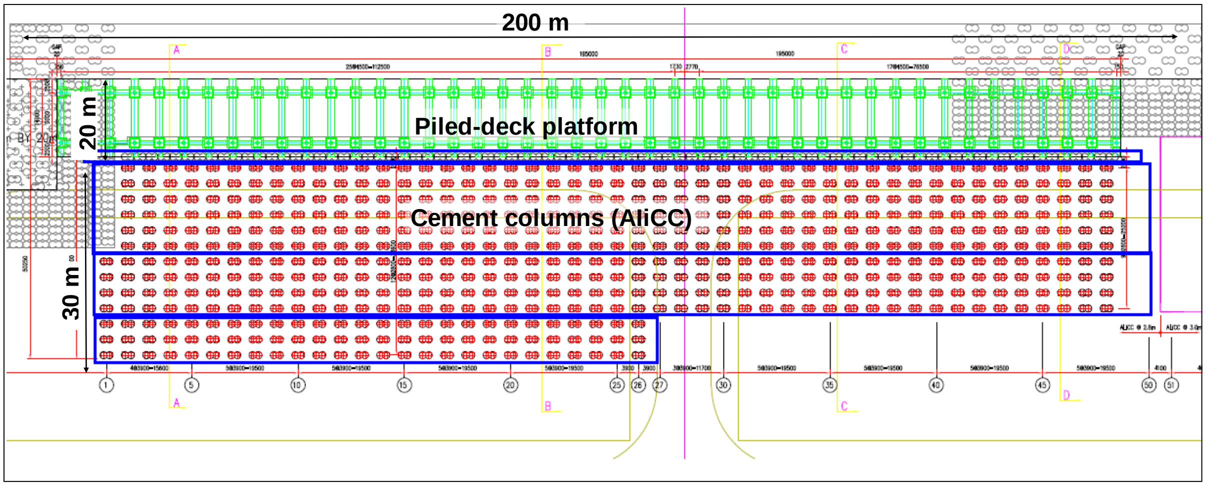

Fig. 18 Plan view of remedial area

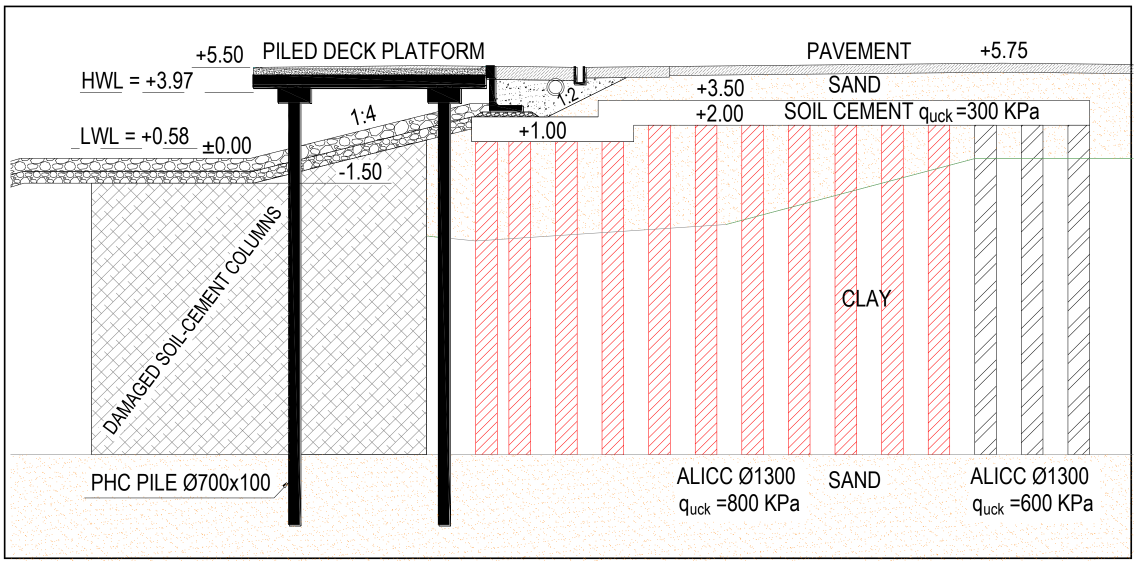

Fig. 19 Cross section of remedial area

3. Up to placing the last lift before failure occurred, the ratio between horizontal displacement and settlement was 0.8. After placing the last lift, significant horizontal movements occurred, and the ratio increased about 2.5.

4. The horizontal shear movements generated pore pressures at about the same rate as the pore pressures caused by the placing of the fill reduced due to the consolidation.

5. The CPTU soundings before the start of placing the surcharge and after the slope failure showed about equal distribution of cone stress, which suggested that no increase of clay shear strength occurred during the consolidation as opposed to what was assumed in the design. The CPTU soundings before the start of placing the surcharge and after the slope failure showed about equal distribution of cone stress, which suggested that no increase of clay shear strength occurred during the consolidation as opposed to what was assumed in the design.

6. The design analyses assumed a slightly smaller imposed surcharge stress than the actual value, 140 KPa versus 150 KPa. The design was total stress analysis applying undrained shear strength of 15 KPa and that this value would increase during the consolidation. However, in the presence of excess pore pressures at the site, effective stress analysis would have been more reliable.

7. It appears obvious that the stability analyses were not representative for the site conditions and, moreover, when the cracks and horizontal movement indicating instability occurred, they were not taken seriously enough to warrant re-assessment of the overall stability along the shore line that could have prevented the slide.

8. The field instrumentation, notably the extensometer and piezometer stations were too few to be fully constructive; not enough to sound a warning before the slide occurred, not useful in the assessment of the reasons for the failure and not supportive in deciding on a remedial solution.

9. Soil improvement designs require incorporation of the observational method in the construction, and, for such use, an adequate redundancy in instruments is necessary, which was not the case for the subject project.
10. The remedial solution stabilized the shore line and no further cracking or excessive soil movements have been noticed at the site.

ACKNOWLEDGEMENTS

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