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Failure of Gravel Compaction Pile (GCP) Improved Embankment – Case Study

T. Nekasiny, N.H. Priyankara, N.B.G. Imali and M.S. Nilawfer

Abstract: Due to the scarcity of suitable land, it becomes necessary to utilize marshy lands consisting of soft soil for infrastructure development. Soft soil has a problematic nature due to its high moisture content, high compressibility, high void ratio and very low shear strength. Hence, it is a responsibility of the geotechnical engineers to overcome these issues by adopting suitable ground improvement techniques. Gravel Compaction Pile (GCP) is one of the most popular soft ground improvement techniques used in the field. This technique has been successfully applied during the construction of the Colombo-Katunayake Expressway project and the Outer Circular Highway project. However, GCP technique has failed in the Southern Expressway Extension Project from Matara to Beliatta section. Therefore, in this study, the causes of the failure of the GCP improved embankment section were studied based on the field records. The slope stability of the embankment during construction was analysed using Matsuo and Kawamura's method. Back analysis revealed that once shear failure occurred in the subsurface, the shear strength of the soft soil reduces to its residual value and it takes longer time to regain its original strength. Further, it was noted that when the soft soil thickness is greater than 10 - 12 m, it becomes extremely challenging to improve the soft ground without any reinforcement. Therefore, it is strongly recommended to accurately interpret the subsurface characteristics in order to select the most suitable ground improvement technique.

Keywords: Gravel Compaction Pile (GCP), Matsuo and Kawamura method, Slope stability analysis, Very soft peaty clay

1. Introduction

The rapid development and population growth of a country would increase the demand for substantial number of infrastructures by making land very scarce. The scarcity of suitable lands for construction encourages the widespread use of areas underlain by weak soil deposits that are considered either marginal or inappropriate. Due to the scarcity of suitable land, a considerable percentage of the number of infrastructure development projects in Sri Lanka, such as the Colombo – Katunayake Expressway (CKE), Southern Expressway, Outer Circular Highway (OCH) and Central Expressway was constructed on flood plains and marshy terrain made up of extremely soft peat, organic soils, and clay [3][11-15].

Road embankment construction over peat deposits is quite challenging, because of the inherent properties of peat such as high moisture content, high compressibility, high void ratio and very low shear strength [1][3] [12-14]. The primary consolidation of peat is very high with significant secondary compression [11-13]. Madhusanka and Kulathilaka [14] reported that Sri Lankan peaty soil has a high moisture content of about 300 %, low shear strength of about 0.99 kN/m^2, and a compression index (C_c) of 1.51. Further, Karunawardena et al. [11] stated that peaty clay found in OCH has a compression index of 1.95 and undrained shear strength of 7.2–19.0 kPa. As such, peaty soil does not provide favourable conditions for construction. Therefore, geotechnical engineers face many challenges because of peaty soil which cannot support heavy loads, hence construction can result in excessive settlement. In order to overcome these challenges, there are two ways, namely, transfer the structural load to an underlying hard stratum through piles or improve the engineering properties of soft soil.

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However, transferring of load through piles to a hard stratum is not an economical solution for roads occupying a large plan area and moderately loaded buildings. Improving engineering properties of soft soil is the most economical option and it is a responsibility of the geotechnical engineers to find appropriate ground improvement technique/s based on the subsurface soil conditions.

Gravel Compaction Pile (GCP) is one of the most popular soft ground improvement technique that aims to increase load-bearing capacity and reduce settlement by densification of subsoil [1] [11]. The installation of GCP consists of a sequence of routine work as shown in Figure 1. In this method, a 40 cm diameter casing is forced into the ground, down to the required depth, under vibration at a frequency of 10 Hz [16]. Then, casing is retracted stepwise while supplying granular material into the casing and compacted the granular material by casing tip under vertical vibration. As this process is repeated, a well compacted 70 cm diameter GCP is created.

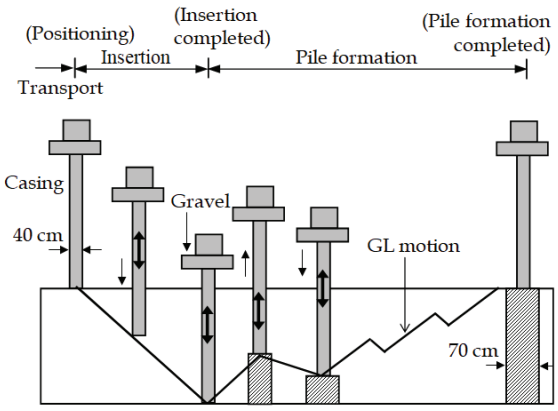


Figure 1 - GCP Construction Procedure

Even though GCP technique has been successfully utilized in the CKE project and OCH project [11][12] to improve soft soils, this technique failed when it was applied to the soft soil improvement in Southern Expressway Extension Project from Matara to Beliatta (Section – 1), causing huge financial loss to the contractor. As such, this paper presents a detailed analysis of the causes of GCP improved embankment failure and lessons that can be learnt from the incident.

2. Details of GCP Trial Embankment

To expand the expressway network in Sri Lanka, the Government of Sri Lanka decided to extend the existing Southern expressway from Matara to Mattala (Figure 2). Section 1 of the Southern Expressway Extension Project from Matara to Beliatta, is mainly going through the Nilwala flood plain which consists of 10.1 km viaducts, 0.6 km bridges/ underpass/drainage box culverts, 15.3 km non-treated area and 4.0 km soft ground treated area [18]. By considering the subsurface soil profile, initially GCP technique had been proposed as the soft ground treatment method. In order to examine the performance of the GCP improved ground, a trial embankment section was done at chainage (Ch.) 7+405 to 7+475. The typical cross section of the GCP improved embankment is shown in Figure 3.

It can be noted that Existing Ground Level (EGL) is about 0.250 m MSL whereas design Road Finished Level (RFL) is about 9.760 m MSL as shown in Figure 3. The side slope of the embankment was planned to maintain as 1:1.5 (1 vertical to 1.5 horizontal) above the berm section and 1:1.8 below the berm. Width of the carriageway is 24.40 m.

2.1 Subsurface Soil Profile

The subsurface soil profile in the trial area was investigated by advancing five boreholes as illustrated in Figure 4. Two boreholes, BH-21 and BH-22, were done in the initial stage of the project at the centre line at Ch. 7+405 and 7+500, respectively, and three boreholes, BH-04, BH-05 and BH-06, were done in the detail design stage at Ch. 7+452 covering the entire cross section.

According to the borehole logs at Ch. 7+405 (BH-21) and Ch. 7+500 (BH-22), soft soil thickness was identified as 17.0 m and 13.0 m, respectively. The subsurface soil profile across Ch. 7+452 can be idealized as shown in Figure 5. It can be seen that soft soil thickness significantly varied from left to right, where soft soil thickness on the Left-Hand Side (LHS) is about 4.0 m whereas that on the Right-Hand Side (RHS) is about 16.0 m.

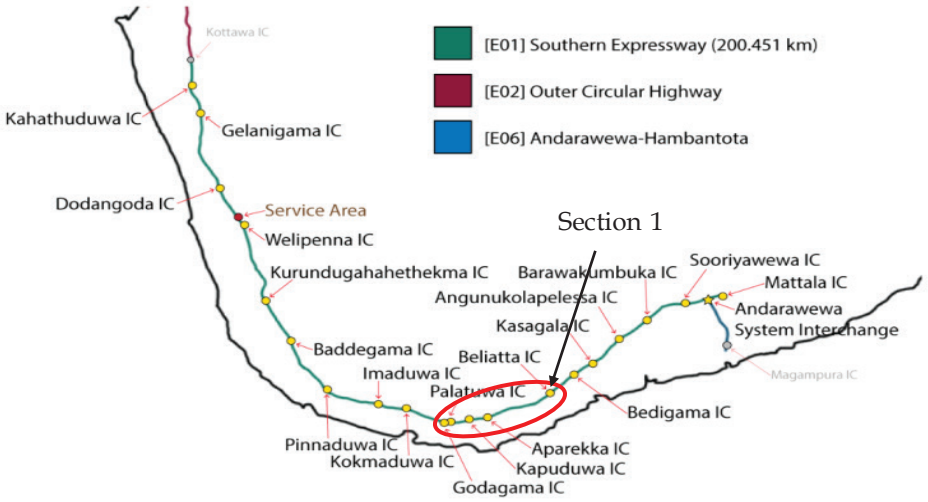


Figure 2 - Project location – Southern Expressway Extension Project (Section 1)

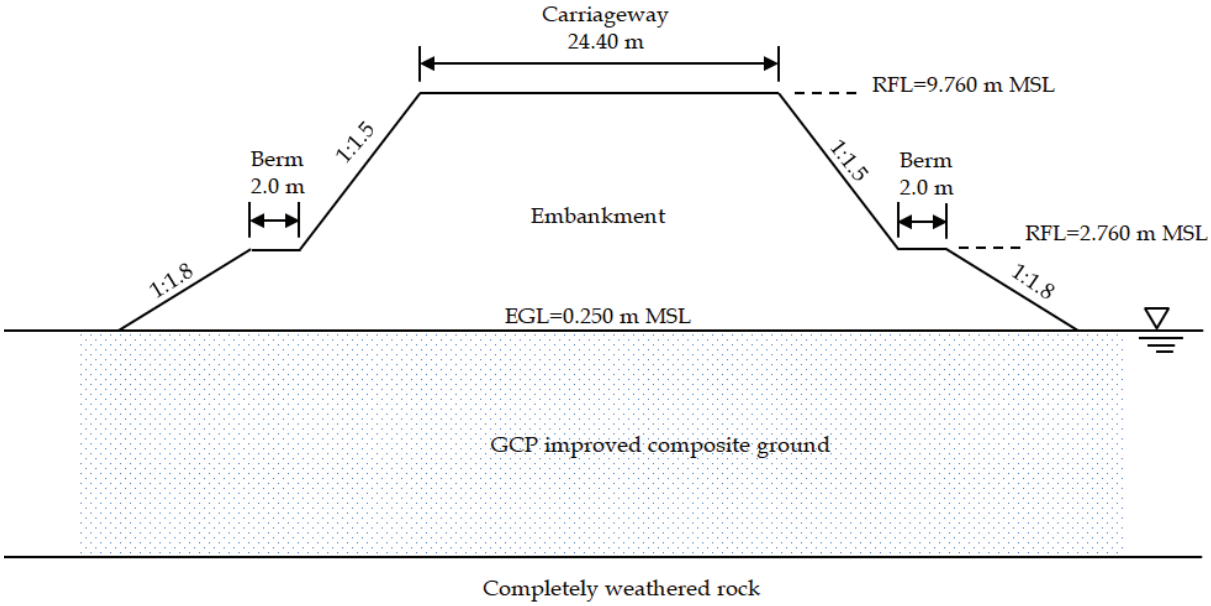


Figure 3 - Typical Embankment at Ch. 7+405 – 7+475 (Not to Scale)

2.2 Geotechnical Parameters of Subsurface Soil

The physical and engineering properties of the subsurface soil based on field and lab test data are summarized in Table 1.

Based on the data presented in Table 1, it can be noted that subsurface mainly consists of very soft peaty clay followed by completely weathered rock layer. The peaty clay has very high moisture content with very high void ratio. Both laboratory triaxial tests and insitu vane shear test results indicated that the peaty clay has very low shear strength. Furthermore, laboratory oedometer test results indicated that the peaty clay has high compression index with very high modified compression index of about 0.3. The average coefficient of consolidation in vertical direction (C_v) and modified secondary compression index were found as 0.8 m²/year and 0.1, respectively. For a conservative design, coefficient of consolidation in horizontal direction (C_h) was assumed as 0.8 m²/year.

2.3 Gravel Compaction Pile (GCP) Installation

Before installation of GCP, 1.5 m high soil fill had been placed on the existing ground as the working platform for the movement of GCP installation machine. Based on the geotechnical parameters presented in Table 1, GCPs were installed at 1.3 m spacing on the Right-Hand Side (RHS) and at 1.6 m spacing on the Left-Hand Side (LHS) in square pattern as there is a considerable variation of the soft soil thickness from LHS to RHS as shown in Figure 5. The spacing of the GCP was taken as 1.6 m when the soft soil thickness is less than 10 m.

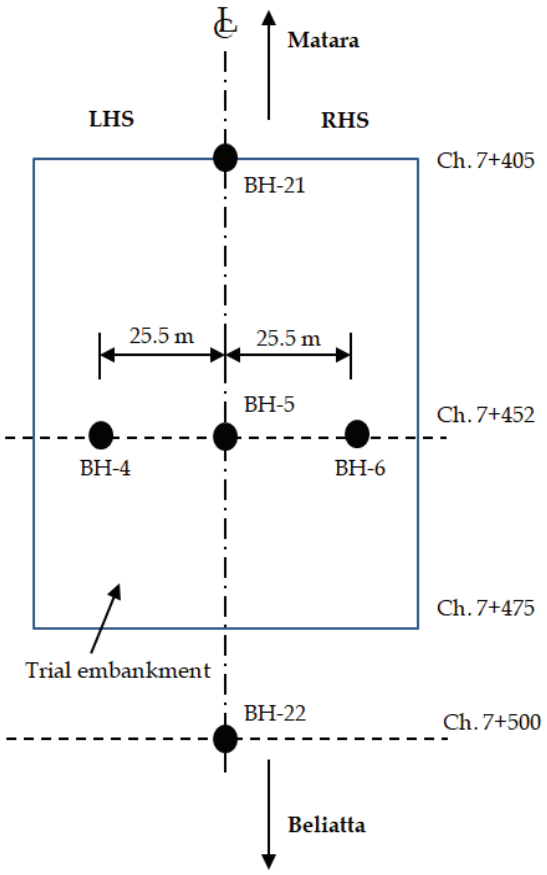


Figure 4 - Borehole Locations within the GCP Trial Embankment Section

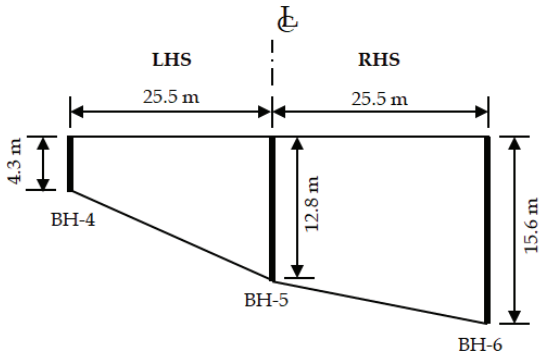


Figure 5 - Variation of Soft Soil Thickness Across Ch. 7+452

With the help of vertical vibrating excitation of the vibro-hammer, casing was penetrated through the soft soil to the hard stratum. When the applied current (generally 35A) of the GCP machine to penetrate the casing was significantly increased (up to about 80A), driving of the casing was terminated as depicted in Figure 6. When the required current to penetrate the casing is significantly increased, that implies casing has reached the hard stratum.

Table 1 – Properties of Subsurface Soil

Property	Value
Soil type	Blackish grey very soft peaty clay
Natural moisture content (%)	107 – 150
Unit weight (kN/m³)	11.38 – 12.85
Coefficient of consolidation (C_v) (m²/year)	0.8 – 1.5
Compression Index (C_c)	0.997 – 1.140
Initial void ratio (e_0)	2.40 – 2.62
Modified Compression Index $C_c' = \frac{C_c}{1+e_0}$	0.28 – 0.33
Modified Secondary Compression Index (C_α')	0.08 – 0.1
Undrained shear strength (C_u) (kPa)	5.75
Undrained friction angle (ϕ_u) (°)	0
Liquid Limit (%)	109 - 159
Plastic Limit (%)	51 - 72

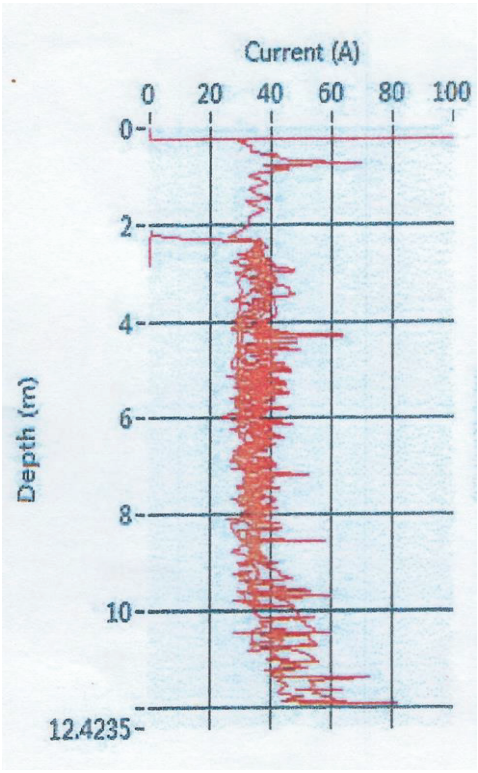


Figure 6 – GCP Termination Criteria Based on the Electric Current Variation at Ch. 7+470

Further, based on the GCP installation records, variation of soft soil thickness along a particular cross section can be identified as shown in Figure 7. By combining borehole logs and GCP installation records, variation of soft soil thickness within the trial embankment section can be idealized as shown in Figure 8.

Based on this idealization, it can be seen that soft soil thickness in RHS varied from 12-18 m whereas in the LHS, it is about 4-10 m. Particularly between Ch. 7+405 and 7+420, the thickness of soft soil varied from 16 to 18 m even on the LHS. This clearly illustrates the highly fluctuating nature of the subsurface soil profile.

2.4 Quality Assurance of GCP

The quality of the GCP was investigated at thirteen (13) GCP locations by conducting Dynamic Cone Penetration Tests (DCPT) at the center of the GCP [4][5][17]. Then, the DCPT-Measured (DCPT–M) values were converted into SPT–N values using the correlation as shown in Equation 1 [4].

$$SPT - N = \frac{DCPT - M}{1.5} \quad \ldots (1)$$

The variation of SPT-N values over depth at different GCP locations are presented in Figure 9.

It can be clearly seen that the SPT–N values gradually increased with depth due to overburden pressure. Even though SPT-N values increased with depth, when the overburden correction was applied to estimate the SPT-N value, the corrected SPT-N values are around 40–50 irrespective of the depth. This implies GCP was properly compacted.

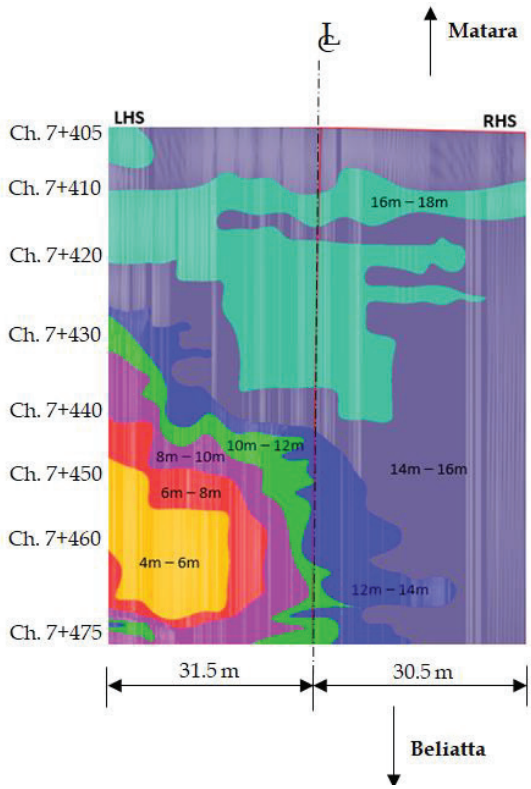


Figure 8 - Soft Soil Thickness Contour Map within the Trial Embankment Section

Then friction angle of the compacted GCP materials (ϕ) was estimated using Equation 2 [6], where N is the field measured SPT-N value. By taking the average SPT-N value of 16 as indicated in Figure 9, it can be calculated that the friction angle of the GCP material (ϕ) is about 32^0.

$$\varphi_s = 0.3N + 27 \quad \ldots(2)$$

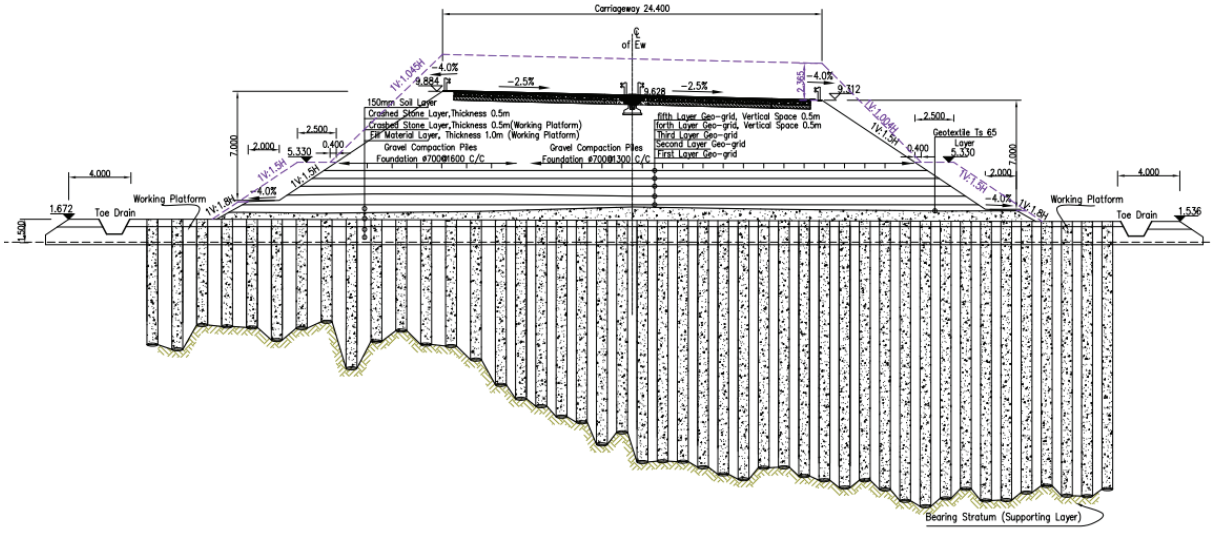


Figure 7 - Typical GCP Installation Records at Ch. 7+460

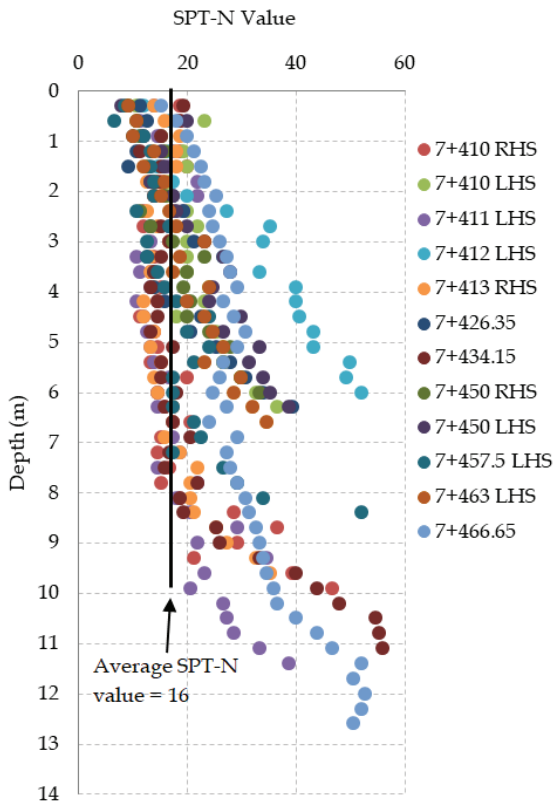


Figure 9 - SPT N Value over Depth at GCP Locations

2.5 Embankment Construction and Field Instrumentation

Soon after the GCP installation, a 0.5 m thick gravel mat was placed as a drainage layer. After that, a geotextile was laid as a separator and embankment filling commenced. The performance of the embankment during construction was evaluated based on the field instrumentation and monitoring data. During the construction of the embankment, the field behaviour was monitored using 16 settlement plates, 5 vibrating wire piezometers, 2 inclinometers, and 38 surface stakes. The field instrumentation arrangement is shown in Figure 10.

Settlement plates were used to measure the settlement of the soft soil below the embankment, while piezometers were used to measure the variation of the pore water pressure. Surface stakes were installed near the toe of the embankment to measure the lateral movement of the subsoil, close to the ground surface (not varying with depth) and to check the stability during construction. Lateral displacement measurements obtained from inclinometers indicated the continuous horizontal movement of the subsoils with depth under the embankment.

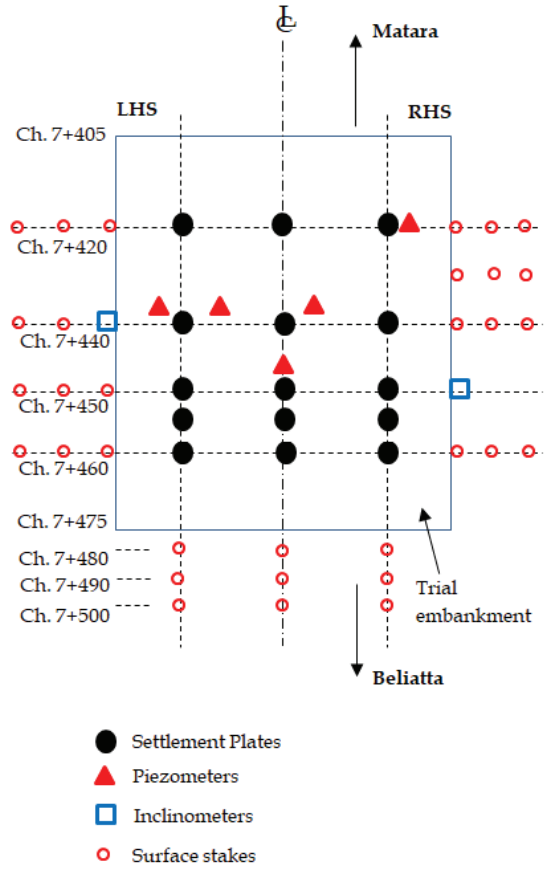


Figure 10 – Instrumentations in the GCP Trial Section

3. Field Observations

The embankment was constructed in four phases, namely, Embankment construction-Stage 1, Waiting period, Berm construction and Embankment construction–Stage 2. The summary of the trial embankment construction is illustrated in Table 2. The graphical presentation of the four phases of the embankment construction is shown in Figure 11.

It can be noted that, due to upheaving of the ground during GCP installation, the working platform elevation before start of the embankment filling was about 2.0 m MSL. The settlement of the subsurface soft ground due to embankment construction is presented in Figure 12. It can be seen that settlement on the RHS is much higher than that of LHS. Settlement after 277 days at RHS and LHS were about 1.35 m and 0.3 m, respectively. The variation of lateral displacement in RHS of the embankment over depth at different time intervals is illustrated in Figure 13. The variation of rate of lateral displacement over time at RHS of the embankment at the critical depths is shown in Figure 14.

Table 2 – Summary of Trial Embankment Construction

Phase	Details	Duration (Days)
0	Platform construction and GCP installation	-
1	Embankment construction – Stage 1	100
2	Waiting period	89
3	Berm construction	40
4	Embankment construction – Stage 2	48

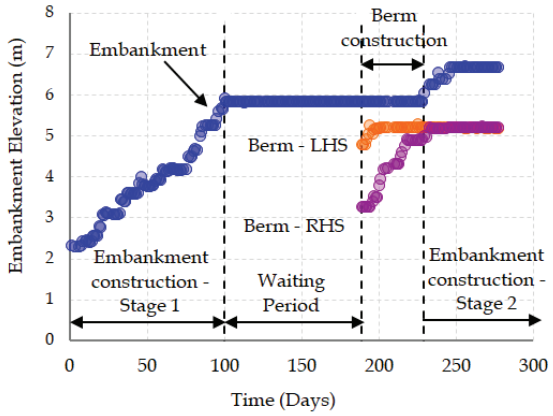


Figure 11 – Variation of Embankment Elevation Over Time at Ch. 7+455

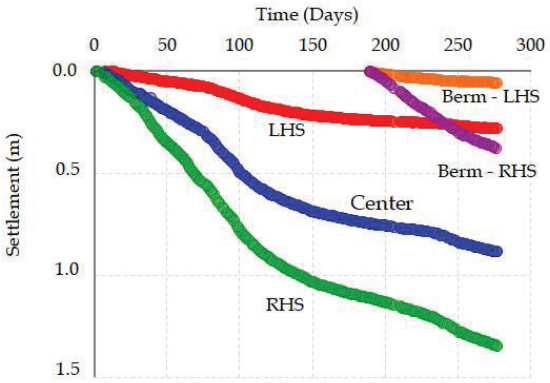


Figure 12 – Settlement Over Time at Ch. 7+455

To monitor the excess pore water pressure dissipation, it is necessary to install the piezometers before starting the embankment construction. According to the information provided, three (3) piezometers were installed soon after the GCP installation. However, none of the piezometers were under working condition during the embankment construction - Stage 1. As such, two additional piezometers were installed about 60 days after the initiation of embankment construction built to a height of 4.0 m MSL elevation.

According to the data obtained, the variation of pore water pressure over time was drawn as shown in Figure 15 for Ch. 7+450 and Ch. 7+435 at depths of 3.75 m and 8.45 m, respectively. It is clearly seen that pore water pressure is gradually increasing with time during all the phases. Even during the waiting period, without embankment filling, the excess pore water pressure was not dissipated.

4. Analysis

4.1 Slope Stability Based on Matsuo and Kawamura Method

It has been widely recognized that the failure of soft ground is closely related to the magnitude and history of the deformation which had taken place before final failure. It uses information from practical measurements in the field to control embankment construction to be safe and efficient. When the soft ground is under loading, in addition to the consolidation, there is a possibility for horizontal soil flow (shear deformation). This fact makes it difficult to theoretically distinguish the displacement and the failure of soft ground. It is obvious that failure occurs when the progress of shear deformation is faster than the consolidation settlement. Therefore, the graphical method proposed by Matsuo and Kawamura [7] is commonly used to estimate the stability of the embankments constructed on soft ground based on the field monitoring data as shown in Figure 16.

Figure 16 indicates a relationship between settlement and ratio of lateral displacement to settlement, and each curve corresponds to a different Factor of Safety (FOS) value, varying from 1.0 to 1.67. Then, based on the obtained field monitoring data, embankment settlement versus ratio of lateral displacement to settlement was plotted in Matsuo and Kawamura's diagram as shown in Figure 17. This clearly indicates the variation of FOS at different depths of the soft soil against slope failure of embankment at different phases of construction.

It can be observed that FOS gradually decreases with the increment of embankment height at Stage 1 (Figure 17) due to huge lateral displacement of about 200 mm on RHS of the embankment as shown in Figure 13.

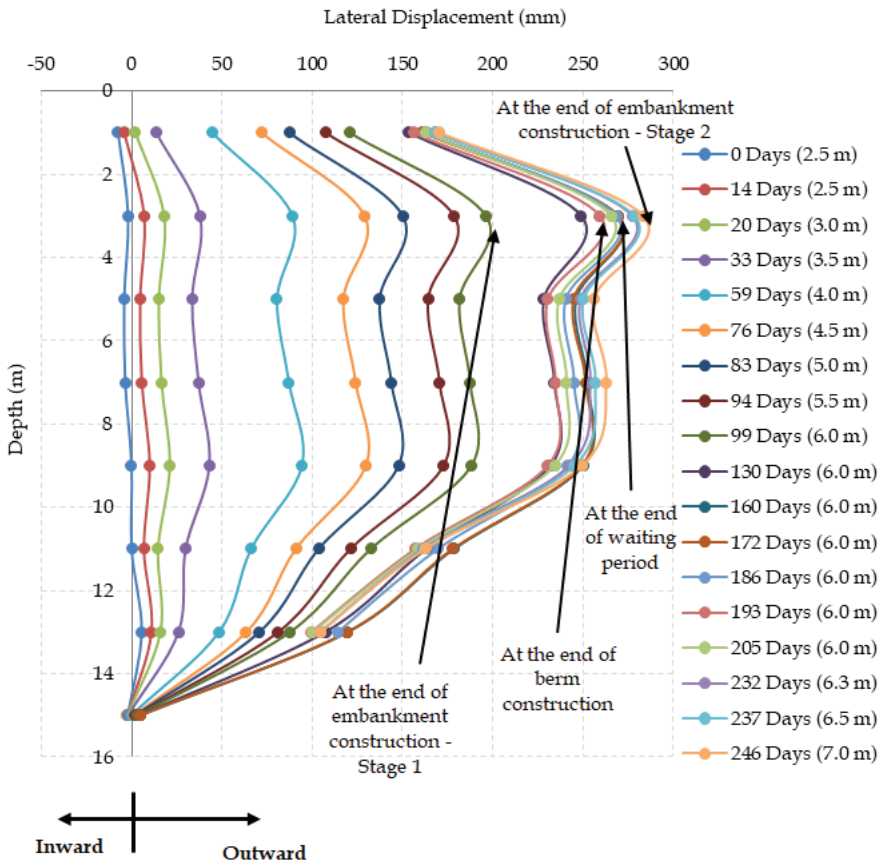


Figure 13 – Variation of Lateral Displacement Over Depth at RHS of the Trial Embankment

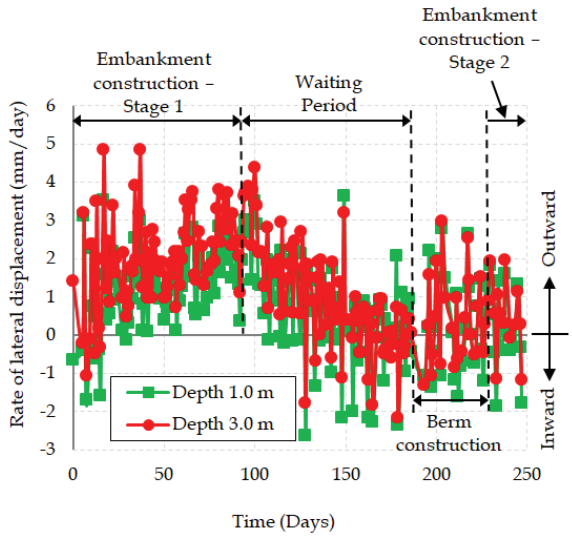


Figure 14 – Variation of Rate of Lateral Displacement Over Time for RHS of Embankment at Ch. 7+455

The significant outward lateral movement can be observed down to a depth of 12.0 m at the end of Stage 1 of embankment construction. The rate of lateral displacement per day has increased up to 4-5 mm/day as indicated in Figure 14.

According to the field practice norms, if the rate of lateral displacement is between 4 to 5 mm/day, frequent monitoring is required. Further, FOS at the end of Stage 1 of embankment construction can be estimated as 1.24. Therefore, the embankment filling was stopped as indicated in Figure 11 (waiting period) expecting an improvement in FOS.

The continuous increase of pore water pressure (Figure 15) during embankment construction – Stage 1 clearly indicated the unsafe nature of the embankment at that phase. As such, embankment construction was stopped for a period of 89 days (waiting period) expecting the dissipation of excess pore water pressure to improve the stability.

Although the excess pore water pressure was not dissipated during the waiting period, the rate of increment of pore water pressure significantly decreased at shallow depth as illustrated in Figure 15(a). However, at greater depth [Figure 15(b)], pore water pressure has increased even during the waiting period. This behaviour clearly indicates that during Stage 1 of the embankment construction, the existing

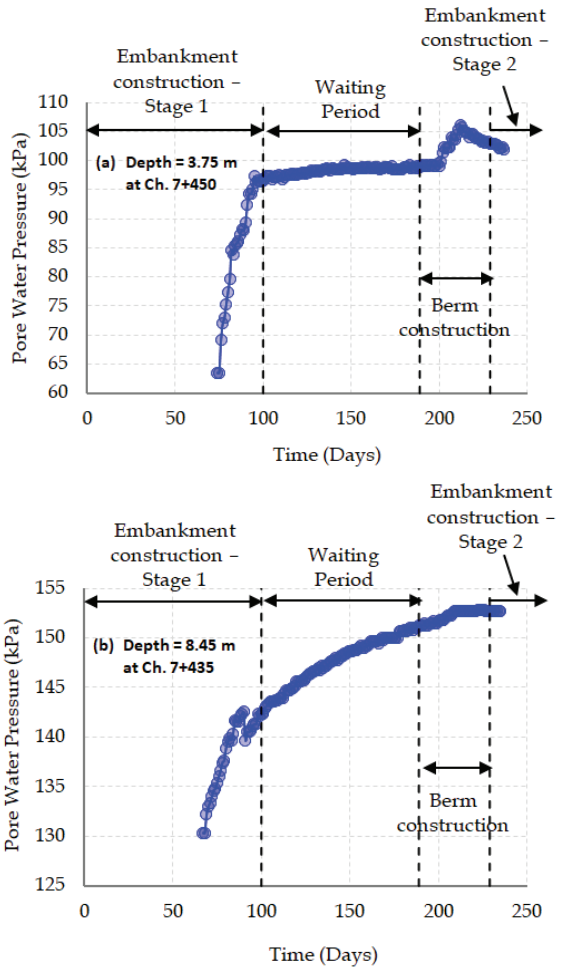


Figure 15 – Variation of Pore Water Pressure with Time

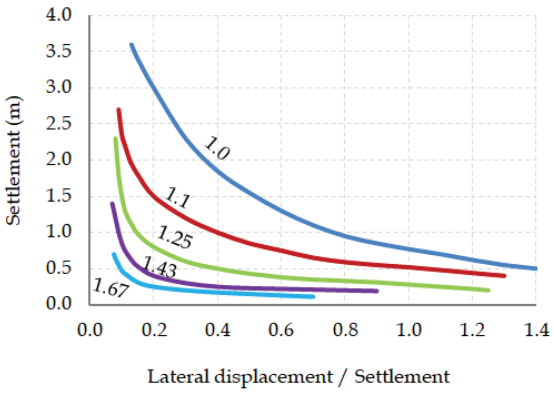


Figure 16 - Matsuo and Kawamura's Diagram [7]

GCPs were ineffective in helping to transfer the pore water at greater depth to the ground surface [10]. However, during the design stage, generally it is assumed that pore water in the soft ground flow towards the GCP due to higher horizontal permeability of the soft soil and collected pore water transfer to the ground surface through the GCP. Based on field monitoring data, it seems that vertical drainage path in the GCP has been significantly reduced due to smear effect and/ or higher horizontal permeability of soft soil has been diminished due to disturbance during GCP installation. As a result, excess pore water pressure has not dissipated during the embankment construction – Stage 1 and even during the waiting period. Moreover, it can be noted that if length of the GCP was greater, the vertical drainage through the GCP due to capillary action may not have been effective. Even though rate of lateral displacement has been significantly reduced during the waiting period, additional 70 mm outward lateral movement can be observed during the waiting period. As such, this behaviour further reduces the FOS against embankment slope failure down to 1.20 even during the waiting period.

Then, a berm has been introduced up to the elevation of 5.33 m MSL in both RHS and LHS of the embankment to reduce the lateral displacement. The newly introduced berm and revised design cross section is shown as a dotted line in Figure 7. It is well known that a berm provides an additional lateral support against outward movement of the embankment. Berm construction process and settlement due to berm construction are depicted in Figure 11 and Figure 12, respectively.

It can be noted that settlement due to berm construction in RHS is about 0.4 m whereas in LHS is about 0.05 m. This clearly indicates the effects of variation of soft soil thickness across the trial embankment area. By providing the berm, embankment moved about 10 mm in the inward direction as illustrated in Figure 13. Further, rate of lateral displacement has changed to 1-2 mm/day in the inward direction as depicted in Figure 14. However, there is no reduction in pore water pressure due to berm construction. It can be clearly seen that settlement versus lateral displacement to settlement ratio graphs in the Matsuo and Kawamura's [7] plot have been moved to the left, indicating a slight improvement in stability. This behaviour is clearly illustrated in the enlarge view of the variation of FOS at the depth of 3.0 m at different phases of embankment construction in the same figure.

Once the FOS has increased and indicated a slight improvement in stability, embankment construction – Stage 2 has been started as shown in Figure 11.

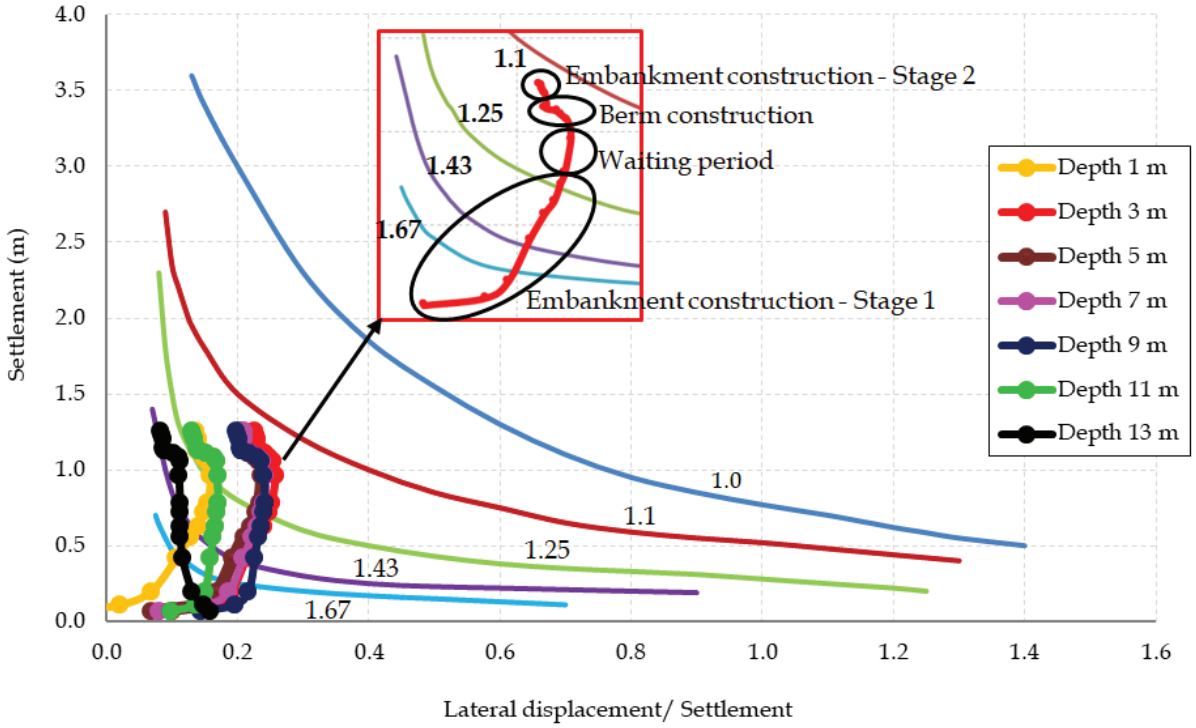


Figure 17 - Variation of FOS in RHS of the Embankment using Matsuo and Kawamura Method [7] at Ch. 7+455

When the embankment elevation increased to 7.0 m MSL, embankment has again started to move in the outward direction as shown in Figure 13. Even though the rate of lateral displacement is about 2.0 mm/day, the FOS curves in the Matsuo and Kawamura's [7] plot moved in the vertical direction, indicating further reduction in FOS. Reduced FOS values are about 1.13 – 1.17 at depths 3 m to 7 m which is less than the required minimum FOS of 1.20 for short term stability [2]. Therefore, it is clearly indicated that the embankment is unstable at this stage.

4.2 Determination of Shear Strength Parameters Based on Back Analysis

The above embankment conditions were numerically modelled using GEOSLOPE SLOPE/W software as shown in Figure 18. Using the back-analysis technique, shear strength parameters of GCP improved composite ground at different phases of the embankment construction were estimated. Spencer's method was used as the constitutive model while "Entry and Exit" method was used to generate the slip surfaces. Since the occurrence of soft soil is critical on RHS of the embankment, only RHS of the trial embankment section was considered for the slope stability analysis. In the model, it was assumed that only the soft soil portion below the embankment was improved with GCP and the water table was maintained at the existing ground surface. According to the field observations, only three GCPs were installed beyond the toe of the embankment. In addition, 200/200 geogrids (tension in both transverse and longitudinal direction is 200 kN/m^2) were placed in the embankment to enhance the slope stability.

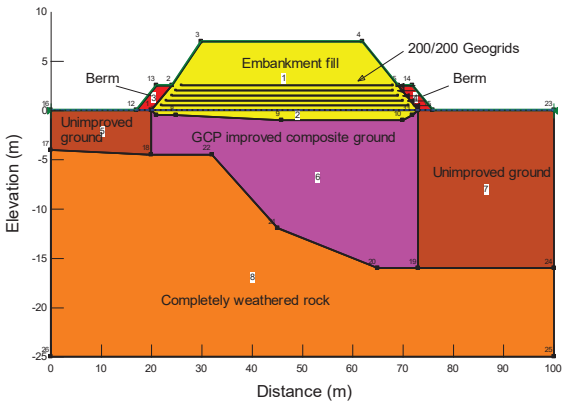


Figure 18 – Numerical Model used for the Back Analysis

The geotechnical parameters of other materials (except GCP improved composite ground) used for the analysis are shown in Table 3. Effective shear strength parameters (c' and ϕ') were used for embankment fill material and completely weathered rock while undrained shear strength parameters (c_u and ϕ_u) were used for unimproved ground for the analysis. Based on the GCP spacing and diameter, area

replacement ratio (a_s) was estimated as 0.228. Further, it was assumed that unit weight of the GCP improved composite ground (γ_{avg}) does not vary with the embankment fill height, and average shear strength method was used to compute the unit weight of GCP improved composite ground [1] as presented in Equation 3. By taking unit weight of GCP material as 22 kN/m^2, the average unit weight of GCP improved composite ground can be estimated as 14.28 kN/m^3.

$$\gamma_{avg} = \gamma_{GCP} a_s + \gamma_{clay}(1 - a_s) \quad \ldots (3)$$

In this analysis, assuming that FOS obtained through Matsuo and Kawamura method [7] is correct, shear strength parameters of the GCP improved composite ground were estimated from trial and error method. Since friction angle of the GCP improved composite ground is governed mainly by the GCP material, it was assumed that friction angle of GCP improved composite ground is not varying with the embankment height. As such, the average friction angle (ϕ_{avg}) of GCP improved composite ground is estimated using Equation 4 [1], where μ_s is the stress ratio in GCP. By taking stress concentration ratio (n) as 3, μ_s was estimated to be 2.06.

$$tan\varphi_{avg} = \mu_s a_s tan\varphi_s \quad \ldots (4)$$

The shear strength parameters obtained through back analysis are shown in Table 4 and critical failure surfaces at different stages of embankment construction are illustrated in Figure 19. Based on above explanation, the friction angle of the GCP improved composite ground can be computed as 16.4^0.

Table 3 – Material Parameters used for SLOPE/W Analysis

Material	γ (kN/m^3)	C (kPa)	ϕ (^{o})
Embankment Fill	20	5	32
Completely Weathered Rock	21	10	38
Unimproved ground	12	5.75	0
GCP improved ground	14.28	-	-

Based on the results presented in Table 4, it can be noted that undrained cohesion (c_u) gradually decreases with the increment of embankment height. According to Skempton and Bjerrum [9], undrained shear strength gain depends on the applied external load and degree of consolidation of the soft soil. Phase 1 to phase 3 of the embankment constructions is under the same applied external vertical load (embankment height = 6.0 m). Even though, 229 days have been spent for the construction from beginning to phase 3, there is no improvement in shear strength gain during this period due to less dissipation of excess pore water pressure. Poor dissipation of excess pore water pressure may result in the very slow primary consolidation. Further, introduction of a berm has not had any influence on the shear strength gain even though it reduced the outward movement of the embankment.

As such, it is very clear that there is no shear strength gain of soft soil due to GCP installation and stage construction of the embankment. The continuous reduction of FOS and outward lateral movement of embankment led to abandoning of the GCP ground improvement technique in the project and huge financial loss has occurred to the contractor.

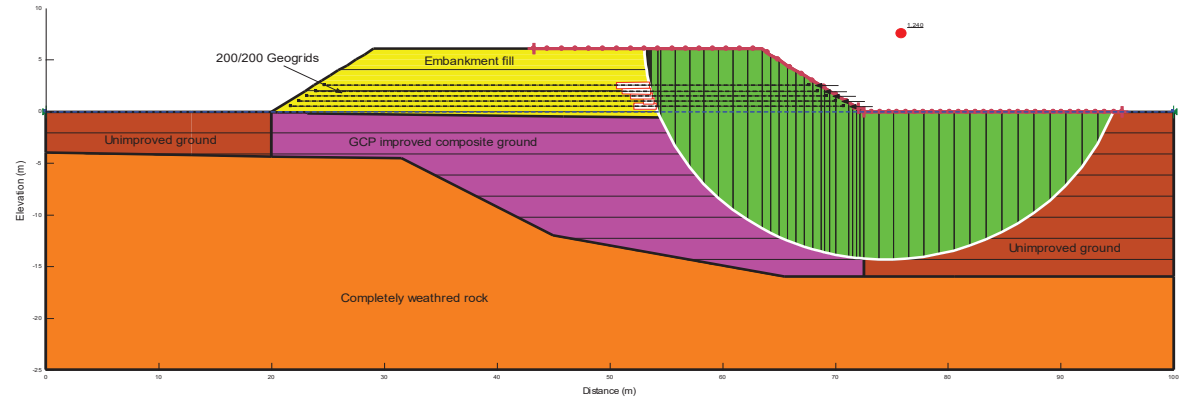
Table 4 – Back Analysis Results

Construction phase	FOS		Shear strength parameters	
	Matsuo and Kawamura	SLOPE/W	Cohesion c_u (kPa)	Friction angle ϕ_u (^{o})
Stage 1	1.240	1.240	17.3	16.4
Waiting period	1.200	1.203	15.7	16.4
After berm construction	1.205	1.209	12.0	16.4
Stage 2	1.130	1.131	12.0	16.4

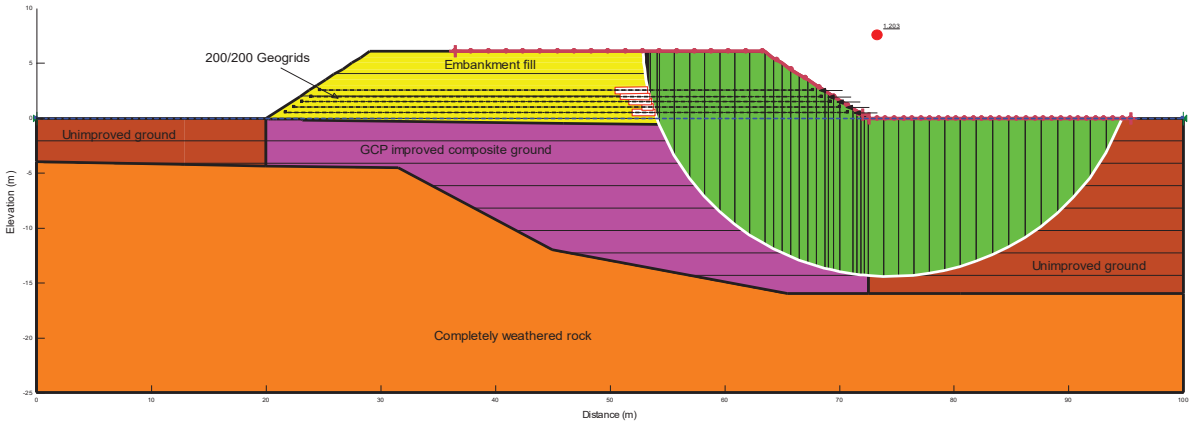
5. Conclusions and Recommendations

Based on this research study, the following conclusions can be drawn:

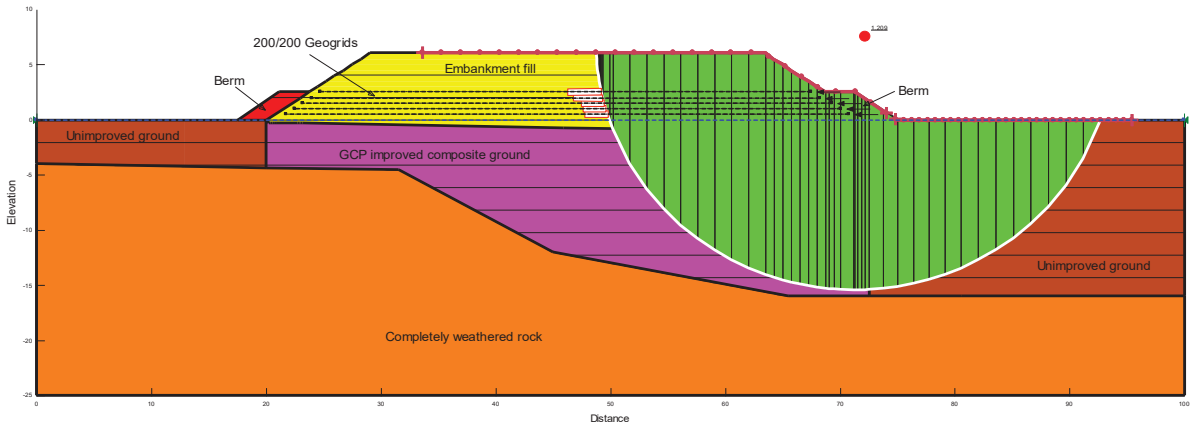
1. According to limited borehole investigations, soft soil thickness in the RHS was identified as 15.6 m and GCP spacing was decided by taking the average soft soil thickness as 13.7 m. However, during GCP installation, it was realized that average thickness of soft soil on the RHS is about 16 m and, at some locations, it may be around 18 m. As such, it can be concluded that GCP spacing was decided by wrong interpretation of the soft soil thickness. This clearly indicates the importance of proper site investigation prior



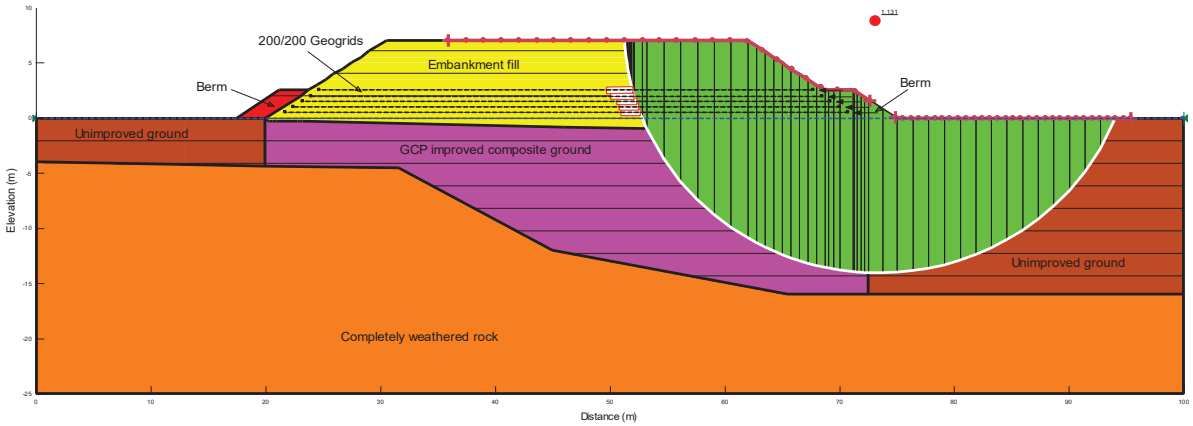
(a) At the end of embankment construction – Stage 1



(b) At the end of waiting period



(c) At the end of berm construction



(d) At the end of embankment construction – Stage 2

Figure 19 – Critical Failure Surfaces at Different Stages of Embankment Construction

to start of the GCP installation.

2. As per depth measurements carried out during the GCP installation, soft soil thickness was higher than the value considered for the GCP design. However, no precautions have been taken to revise the original design or no countermeasures have been implemented before start of the embankment filling. Hence steps should have been taken to revise the original design if the field records indicated a somewhat different soil profile than the soil profile used for the original design, and, if required, install additional GCPs in between already installed GCPs.

3. It was observed that only three GCPs were installed beyond the toe of the embankment at a particular row. However, it was realized that three GCPs are insufficient to provide toe support to the embankment due to high thickness of the soft soil layer. If the thickness of the soft soil is greater, adequate stability could have been achieved against slope failure by installing precast concrete piles as toe support.

4. The construction of the trial embankment started immediately after the GCP installation. However, it is a well-known fact that the strength and stiffness of the surrounding soft soil are reduced as a result of disturbance during GCP installation. Hence, it is necessary to allow adequate time to stabilize the surrounding soft ground to recover its strength. The waiting period can be decided based on the pore pressure measurements using piezometers.

5. It can be noted that, in the trial embankment section, piezometers were installed after starting the embankment construction. This clearly indicates that embankment filling was started without proper prior monitoring of pore water pressure. Hence, it is recommended to establish a proper field monitoring system, prior to starting embankment construction.

6. Based on the field observations and data analysis, it can be concluded that, when the soft soil thickness is more than 10 – 12 m, it is really difficult to improve the soft ground without any reinforcement. Hence it is strongly recommended to select the most suitable ground improvement technique by accurately interpreting the soft soil thickness.

7. The slow rate of dissipation of excess pore water pressure after embankment loading is due to the shear failure of the soft soil. Due to disturbance of the soft soil during GCP installation and other field activities, horizontal permeability of soft soil reduces and the smear zone surrounding GCP increases. As a result, the vertical drainage path within the GCP reduces, causing reduction in dissipation of pore water pressure. This behaviour clearly indicates the importance of proper maintenance of field activities without disturbing the subsurface during GCP ground improvement.

8. Once shear failure has occurred, the strength of the soft soil reduces to its residual value and it takes a longer time to regain its original strength [16]. After the shear failure, even constructing a berm to provide lateral support will not be successful.

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