M. A. Ali, S. M. Seraj and S. Ahmad (eds): ISBN 984-823-002-5

Evaluation of Performance of Rajshahi Town Protection Embankment

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Abstract

The 18 km Rajshahi Town Protection Embankment was threatened during 1998 floods. Seepage, piping, sliding and rain cuts occurred at a number of sections of the embankment. Considerable portions of both the T-Groyne and Shashanghat Closure (I-Groyne) were damaged. The left bank of the Padma eroded severely. A number of protective measures were undertaken to prevent seepage and erosion of the embankment, groynes, spurs and closures which included bamboo pilling, dumping of rock boulders, sand bags, brick crates and brick bats. Polythene cover was used to prevent erosion due to rain cut. Re-sectioning and thorough repairing of the flood embankment had to be done to raise the embankment section up to the design level. In December 1998, Bureau of Research, Testing and Consultation, Bangladesh University of Engineering and Technology, Dhaka carried out field and laboratory investigations. The detailed results of the field and laboratory investigations have been presented in this paper. Based on results obtained from the present investigation, the embankment soils have been found to be in a state of fairly good degree of compaction. Results of stability analyses indicated that despite the significant damages that occurred during the floods, the embankment retained adequate factor of safety against bearing capacity failure. Moreover, timely implementation of protective measures and good in-situ soil parameters played key roles in the overall successful performance of the embankment during the floods.

INTRODUCTION

During 1998 floods significant damages occurred to the Rajshahi Town Protection Embankment. The embankment and its components were threatened during the floods. Seepage, piping, sliding and rain cuts occurred at a number of sections of the embankment. During the occurrence of floods, a number of protective measures were undertaken to prevent seepage and erosion of the embankment. In December 1998, after the floodwater had receded, the Bureau of Research, Testing and Consultation (BRTC), Bangladesh University of Engineering and Technology, Dhaka carried out field and laboratory investigations. The major objectives of the investigations were to assess the performance of the embankment during 1998 floods and to examine the existing condition of the embankment.

This paper presents the extent of damage occurred in the embankment during the floods and the preventive measures taken during the flood to prevent erosion of the embankment and its components. Future plan of protective works has been addressed in this paper. The results of the field and laboratory investigations have been presented in this paper. Finally, using the results of field and laboratory investigations, an attempt has been made to assess the present condition of the embankment with particular emphasis on the existing compaction state and overall stability of the embankment.

RAJSHAHI TOWN PROTECTION EMBANKMENT

Rajshahi Town protection embankment was constructed in 1932 to protect the town from flood. Initially it was 16.9 km long and later increased to a length of about 18 km having 20 single sluice gates and brick mattress in some places. The height and constructed slope of the embankment were 21.3 m (above MSL) and 1: 2 (vertical: horizontal), respectively. Only about 2 km on the upstream side is paved and brick mattress was provided on the slope of riverside. With the development of river training work on the Indian side, the main stream of the Padma started flowing vigorously towards Rajshahi town and the city was threatened. In view of that, 3 groynes and 6 spurs were constructed including brick mattress of 1676 m in 1979-80. A closure was constructed near Shashanghat during 1987-88. The embankment is provided with a number of groynes (T-Groyne and I-Grorne), spurs and closures. T- Groyne is an earthen groyne made of silty clay. Head of the T- Groyne is covered by brick mattressing and boulders are placed mainly on the upstream side. Height of the crest of T-Groyne is 21.3 m (MSL). An I-Groyne is placed about 1.3 km down of dam head, which is also an earthern groyne having a length of approximately 500 m.

Head portion of this groyne is covered by brick mattress. Another I-Groyne was constructed 2 km down of the above mentioned I-Groyne, having a length of 1 km. Construction details of this groyne are the same as the other I- Groyne. The 2B-Spur situated 800 m from the second I-Groyne was constructed with brick box. A total of 220 bricks weighing 750 kg is netted by wire and placed one upon another. The width of the spur is staged. On the top the width is about 10 ft. From here up to the T-Groyne, the riverbank is provided with brick mattress. Other 5 spurs were constructed at the down of T-Groyne in several places having lengths of 150 m to 200 m. These are also earthen spurs with brick mattress at the nose. A closure was constructed at Shashanghat having a length of 500 m with 200 ft brick mattress at the nose.

DAMAGES OCCURRED DURING THE 1998 FLOOD

The embankment was threatened during the 1998 flood with seepage, piping, sliding, and rain cuts occurring at different sections of the embankment. The velocity of water current during the 1998 floods was recorded as approximately 18 m/s. The highest water level recorded during the floods was 19.68 m, the highest level ever recorded and this water level was more than 1 m above the danger level. The major thrust took place at the T-Groyne. Initially on 18th August 1998, piping on the nose took place and the nose settled. Later, on 30th August 1998, downstream nose and shank were attacked and damaged badly. About 200 m^2 of downstream shank was washed away. The Shashanghat Closure (I-Groyne) near Panchabate was also threatened during the 1998 flood and considerable portion from the head was damaged. The other groynes and spurs were not attacked by the flood significantly, except the 2B-Spur. The netting of brick blocks was tore off which needed major repairing. The current took away a 150-ft section of the nose of closure. The left bank of the Padma eroded severely, specially, at a location 2 km downstream of the T-Groyne and in some places in upstream. This area shall be protected using brick mattress or other suitable type of bank revetment measures. Re-sectioning and repairing of the flood embankment and its components has to be done thoroughly to revive the section up to the design level.

PROTECTIVE WORKS UNDERTAKEN

Several types of protective works were undertaken for the protection of the embankment from erosion due to flood. Photographs of various protective measures are shown in Figs. 1 to 3. The following protective works were undertaken: (i) Bamboo Pilling: Extensive bamboo piling was carried out for the protection of embankment against erosion; (ii) Dumping of Boulders: Boulders were dumped on the upstream and downstream of groynes and closure for their protection; (iii) Dumping of Sand Bags: Gunny bags filled with local sand were dumped in order to prevent seepage, erosion of groynes, spurs and closure. Approximately 1,35,000 sand bags were dumped, of which 85000 bags were dumped at the T-Groyne; (iv) Percupine: This is a method in which bricks were encased in bamboo boxes. These boxed were placed at the locations where boulders and sand were found to be washed away; (v) Brick Crates: Bricks were placed in a netted wire and placed on embankment slopes for their protection; (vi) Dumping of Brick Bats: Gunny bags filled with brick bats were dumped at several places of the embankment for erosion protection; (vii) Polythene Cover: This was used to prevent erosion due to rain cut. The whole embankment (from upstream toe to the downstream toe) was covered with polythene when heavy rainfall took place. A total of 1200 m polythene cover having width of 30 ft was used.

With the implementation of the various protective measures during the flood, it was possible to protect the embankment from catastrophic failure. The Surface Water Modelling Center (SWMC), Dhaka has been requested to conduct a mathematical model study on the 1998 floods. After the completion of mathematical and physical model studies, the type of appropriate protective measures could be ascertained and permanent protective measures could be undertaken. Meanwhile overall repair of the damaged T-Groyne and other infrastructure including the flood embankment should be undertaken as soon as possible. For the 1998-99 financial year, Bangladesh Water Development Board has taken up the Rajshahi Town Flood Protection Embankment (5th Term) Project which includes: (i) Repair works of groynes: Taka 290 lac; (ii) Repair and strengthening of foot of 2B-Spur: Taka 50 lac; (iii) Brick mattress in upstream and downstream of groynes and cement concrete blocks at the foot (1100 m): Taka 580 lac; (iv) Brick mattress work at Char Kazla (250 m): Taka 50 lac; (v) Resectioning of embankment (5 km): Taka 50 lac; and (vi) Repair of brick mattress (3 km) : Taka 100 lac.

POST FLOOD GEOTECHNICAL INVESTIGATION

A detailed field investigation was carried out at the Rajshahi Flood Protection Embankment after the 1998 flood. Laboratory tests were also conducted on disturbed and undisturbed soil samples collected from the embankment. The major objectives of the soil investigation program were as follows: (i) Determination of stratigraphic sequences of sub-soil strata by drilling of boreholes; (ii) Evaluation of consistency and relative density of the sub-soil by carrying out Standard Penetration Test (SPT); (iii) Identification

Figure 1: Photograph showing bamboo piling for erosion protection

Figure 2: Photograph showing dumped rock boulders on groyne of the embankment

Figure 3: Photograph showing dumped sand bags and brick crates on the embankment slope

and classification of the embankment soil and the foundation soil by carrying out index property tests in the laboratory; (iv) Evaluation of the existing compaction state of the embankment soils by carrying out *in-situ* density test and Standard Compaction Test in the laboratory; and (v) Assessment of the strength, compressibility and permeability properties of the soil by carrying out consolidated undrained direct shear test, unconfined compression test and one-dimensional incremental loading consolidation test.

FIELD INVESTIGATIONS AT THE EMBANKMENT

Five boreholes were drilled vertically at this site using wash boring technique. Wash borings of small diameter (approximately 100 mm) were drilled by water flush aided by chiselling. The depth of boreholes below the surface varied from 30 ft to 50 ft. The density and stiffness characteristics of the sub-soil layers in the boreholes were measured by performing Standard Penetration Test (SPT) at 1.5 m (5 ft) intervals by means of standard 50.8 mm outside diameter split-spoon sampler. Disturbed and undisturbed samples were collected from the boreholes. A split-spoon sampler was used to obtain the disturbed samples. Undisturbed samples were also retrieved from cohesive layers of the boreholes by pushing conventional 76 mm external diameter thin-walled Shelby tubes following the procedure outlined in ASTM D1587 (ASTM, 1989). Siddique et al. (1999) reported summary of the sub-soil stratification for these boreholes.

Density of embankment soils in place was determined at ten locations by the Sand Cone Method as outlined in ASTM D1556 (ASTM, 1989). In order to assess the degree of field compaction, maximum dry density of four categories of the embankment soils collected from four different locations of the embankment was determined in the laboratory.

LABORATORY INVESTIGATIONS

Index Properties of Soil Samples

The results obtained from index property tests on different samples are presented in Table 1. The natural moisture contents for the samples obtained from boreholes BH-1, BH-2, BH-3, BH-4 and BH-5 varied from 21.9 % to 42.4 %, 23.9 % to 32.2 %, 19.5 % to 28.1 %, 21.5 % to 27.9 % and 18.1 % to 36.4 %, respectively. The values of liquid limit, plastic limit and plasticity index of the cohesive samples varied from 35 to 46, 20 to 28 and 8 to 26, respectively. From the particle size distribution curves, percent clay (< 0.002 mm), percent silt (0.002 mm to 0.06 mm) and percent sand (0.06 mm to 2 mm) were determined using MIT Texural Classification System. Percent finer than number 200 sieve and fractions of sand, silt and clay of the samples tested are presented in Table 1. Using the results of index property tests, soil samples obtained from the embankment have been classified according to Unified Soil Classification System (USCS) as outlined in ASTM D2487 (ASTM, 1989). Classifications of the cohesive and non-cohesive soil samples are also shown in Table 1. The cohesive samples obtained from the boreholes are typically inorganic clays and silts of low to medium plasticity (USCS Symbols are CL and ML) while the sandy samples are either SM or SP-SM.

Moisture-Density Relations and Assessment of *In-situ* Compaction of the Embankment

Moisture-density relationships were determined for four samples obtained from four selected locations of the embankment, namely, slope of T-groyne, Buattala, Noboganga and Shashanghat. Each test was carried out following the procedure outlined in ASTM D698 (ASTM, 1989). The moisture-density relations of the four samples are presented in Fig. 4. From the moisture-density curves, optimum moisture content (w_{opt}) and the corresponding maximum dry density for the soil samples were estimated. The values of the w_{opt} and maximum dry density of the samples varied from 15.1 % to 19.5 % and 100.5 lb/ft^3 to 110.4 lb/ft^3, respectively. In-place density of the embankment soil was determined at ten spots of the above-mentioned four locations of the embankment, following Sand Cone Method. A summary of the field density test results is shown in Table 2. It can be seen from Table 2 that the *in-situ* dry density and compaction of the soil samples at four locations of the embankment varied from 84.3 lb/ft^3 to 105.1 lb/ft^3 and 79.5 % to 95.2 %, respectively. The average value of the degree of field compaction at the ten spots was found to be 85.3 %. Although the embankment suffered significant erosion and that the embankment soils were not compacted after the floodwater had receded, an average compaction of 85.3 % indicates that the embankment soils are still in a state of fairly good degree of compaction.

Table 1: Summary of index properties of soil samples

Borehole No./ Sample No.	Depth (ft)	LL	PL	PI	USCS Symbol	Grain Size Distribution			
						% Finer No. 200 Sieve	Sand (%)	Silt (%)	Clay (%)
BH-1 /UD-1	8.0 to 9.5	36	28	8	ML	91.5	12	76	12
BH-1 / D-1	5.0	46	26	20	CL	99.0	3	70	23
BH-2 / D-2 and D-3	10.0 & 15.0	40	28	12	ML	98.0	2	85	13
BH-2 / D-7 and D-8	35.0 & 40.0	-	-	-	-	41.2	64	32	4
BH-3 / UD-1	8.0 to 9.5	37	20	17	CL	87.4	14	72	14
BH-3 / D-3 and D-4	15.0 & 20.0	35	21	14	CL	88.0	13	75	12
BH-3 / D-6	30.0	36	22	14	CL	73.7	26	60	14
BH-4 / UD-1	8.0 to 9.5	38	24	14	CL	93.2	7	80	13
BH-4 / UD-2	13.0 to 14.5	-	-	-	SP-SM	7.5	-	-	-
BH-4 / D-4 and D-5	20.0 & 25.0	46	20	26	CL	92.7	10	61	29
BH-5 / UD-1	8.0 to 9.5	-	-	-	SM	16.8	-	-	-
BH-5 / UD-2	13.0 to 14.5	-	-	-	SP-SM	5.9	-	-	-
BH-5 / D-7	35.0	-	-	-	-	42.5	62	35	3

Direct Shear Test Results

Five consolidated undrained direct shear tests were carried out on the undisturbed samples obtained from the boreholes. In each test, cylindrical samples of 63.5

mm diameter by 25 mm height were initially consolidated using three different normal loads and subsequently sheared under undrained condition. From the failure envelopes of the samples, the values of cohesion (c) and angle of internal friction (ϕ) of the samples have been determined. A summary of the direct shear test results is presented in Table 3. Comparing the values of angle of internal friction obtained for the samples with those reported by Terzaghi and Peck (1967), it can be concluded that the relative density of samples tested from boreholes BH-2, BH-3 and BH-4 are loose while that for the two samples tested from borehole BH-5 are dense.

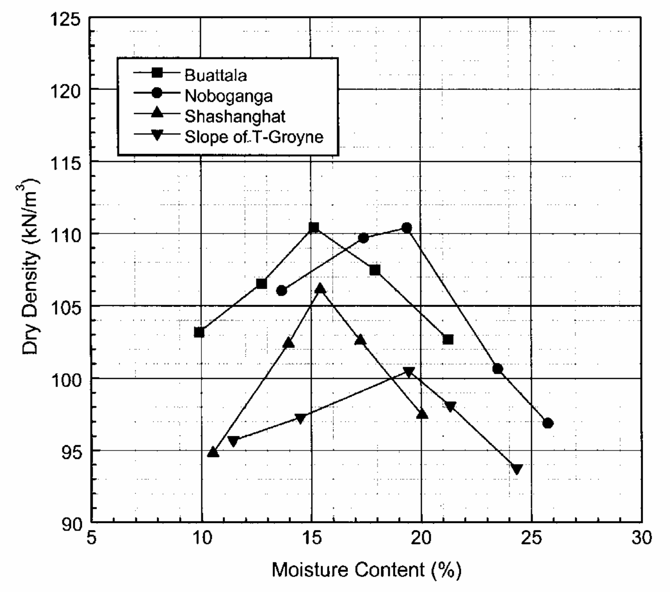

Figure 4: Moisture-density relations of selected embankment soil samples

Unconfined Compressive Strength of Soil Samples

Unconfined compressive strength tests were carried out on two undisturbed samples, one from borehole BH-1 and the other from borehole BH-3. From the stress-strain data, unconfined compressive strength (q_u), and axial strain at failure (ε_f) were determined. A summary of the unconfined compression test results is presented in Table 4. On the basis of the value of undrained shear strength, which is half of the unconfined compressive strength for clays, the samples UD-1 of BH-1 and UD-1 of BH-3 are soft and firm, respectively.

Table 2: In-place dry density and % compaction of soil samples

Sample Location	Optimum Moisture Content (%)	Maximum γ_d in Laboratory (lb/ft^3)	Sample No.	In Place Water Content (%)	In Place Dry Density (lb/ft^3)	*In-situ* Compaction (%)
Slope of T-Groyne	19.5	100.5	1	35.6	87.6	87.2
			2	36.4	86.6	86.2
			3	31.3	90.8	90.3
Buattala	15.1	110.4	1	17.1	90.1	81.6
			2	23.7	94.0	85.1
Noboganga	19.4	110.4	1	18.7	95.7	86.7
			2	16.3	105.1	95.2
			3	17.1	89.6	81.2
Shashanghat	15.4	106.1	1	21.3	84.3	79.5
			2	20.2	85.0	80.1

Table 3: Summary of direct shear test results

Borehole No.	Sample No.	Depth (ft)	Average Water Content (%)	Average Dry Density (kN/m^3)	Cohesion (kN/m^2)	Angle of Internal Friction (Degree)
BH-2	UD-1	8.0 to 9.5	22.1	16.15	20	27
BH-3	UD-1	8.0 to 9.5	18.6	15.89	22.5	27
BH-4	UD-1	8.0 to 9.5	25.2	15.28	20	29
BH-5	UD-1	8.0 to 9.5	27.5	14.45	0	41
BH-5	UD-2	13.0 to 14.5	28.9	14.66	0	39

Table 4: Summary of unconfined compression test results

Borehole No.	Sample No.	Depth (m)	Water Content (%)	Dry Density (kN/m^3)	Value of q_u (kPa)	Value of ε_f (%)
BH-1	UD-1	8.0 to 9.5 ft	21.2	15.24	72.6	6
BH-3	UD-1	8.0 to 9.5 ft	18.1	16.88	146.9	11

Compressibility and Permeability Properties

Compressibility and permeability properties of one sample obtained from borehole BH-1 were determined from incremental loading one-dimensional consolidation tests. Void ratio versus logarithm of effective vertical stress plots and coefficient of consolidation versus logarithm of effective vertical stress plots for the sample are presented in Fig 5. Compression index (C_c) of the sample, determined from the slopes of the loading portion of the void ratio versus logarithmic of pressure curve shown in Fig. 5, was found to be 0.17. The initial void ratio (e_0) of the sample was found to be 0.69. Low values of C_c and e_0 indicate that the compressibility of the sample is low. Depending on the stress range, the values of coefficient of consolidation and coefficient of volume compressibility of the soil sample have been found to be in the range of 1.49 x 10^{-3} to 3.71 x 10^{-3} cm^2/sec and 0.88 x 10^{-4} to 9.8 x 10^{-4} m^2/kN, respectively.

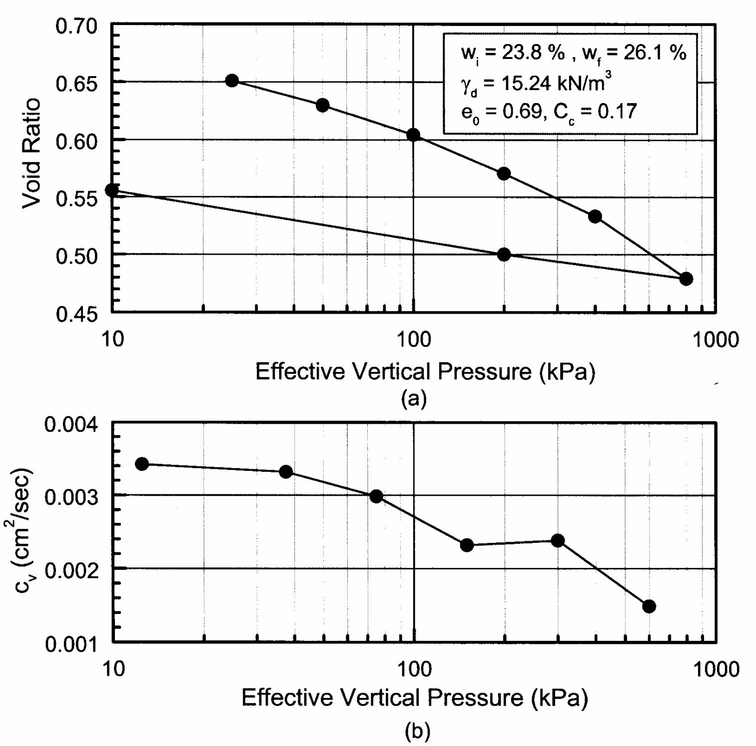

Figure 5: Compressibility plots of a sample: (a) void ratio versus log of effective vertical pressure, (b) c_v versus log of effective vertical pressure

Coefficient of permeability of the samples was determined indirectly from one-dimensional consolidation tests. Depending on the void ratio, the values of coefficient of permeability of the soil samples varied from 1.29 x 10^{-10} m/sec to 3.29 x 10^{-9} m/sec. Void ratio versus logarithm of coefficient of permeability plot has been presented in Fig. 6. It can be seen from Fig. 6 that, the relationship between void ratio and permeability is approximately linear. The average slope of the relationship, which is termed the permeability change index (Tavenas et. al., 1983), C_k for this sample is 0.13. The ratio of compression index to permeability change index, i.e., C_c/C_k for the sample is 1.3. Berry and Wilkinson (1969) reported that for many soils C_c/C_k often lies within the limits of 0.5 and 2.0, while Mesri and Rokhshar (1974) observed that the experimental values of C_c/C_k were found to vary between 0.5 and 5.0.

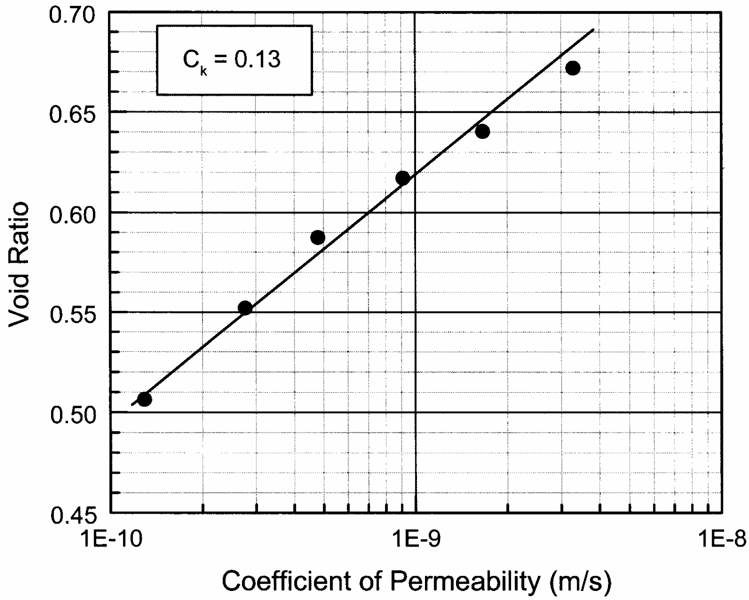

Figure 6: Void ratio versus coefficient of permeability plot for a sample

STABILITY ANALYSIS OF THE EMBANKMENT

The overall stability failure mechanism is development of slip circles resulting in a deep sliding surface. This is a conventional soil mechanics stability problem. Pre-existing slip planes within the soil, or lenses and bends of cracker material can have a significant effect on slope stability. Stability analyses were carried out to evaluate the factor of safety against bearing capacity failure of the embankment for a number of conditions. The following four conditions have been considered: (i) deep stability of the embankment on the river side during high water period; (ii) deep stability of the embankment on the river side during low water period; (iii) deep stability of the embankment on the country side during high water period; and (iv) deep stability of the embankment on the country side during low water period

XSTABL program has been used for the stability analysis. XSTABL performs a two-dimensional limit equilibrium analysis to evaluate the factor of safety for a layered slope using the Simplified Bishop Method or the Janbu Method. Based on the data of the present investigation, stability analyses of the embankment were performed for two different sections of the embankment with heights of 20 ft and 10 ft on the riverside. The height of the embankment on the countryside for both the sections was taken as 8 ft with berm. Soil properties were also varied for the embankment sections. Two types of soil properties for the embankment soil were considered while the properties of the foundation soil were kept the same for all the analyses.

Table 5: Summary of the results of stability analyses

Soil Properties		Analyses No.	Embankment Height on River Side	Height of Water on River Side	Position of Sliding Surface	Factor of Safety
Foundation	Embankment					
c = 0 ϕ = 25° γ_m=15.5kN/m^3 γ_s=17 kN/m^3	c = 20 kN/m^2 ϕ = 28° γ_m=17.0kN/m^3 γ_s=18.5 kN/m^3	1	20 ft	15 ft	R/S	1.88
		2	20 ft	0 ft	R/S	2.10
		3	20 ft	15 ft	C/S	4.98
		4	20 ft	0 ft	C/S	4.97
		5	10 ft	7.5 ft	R/S	2.51
		6	10 ft	0 ft	R/S	2.77
		7	10 ft	7.5 ft	C/S	5.26
		8	10 ft	0 ft	C/S	5.26
	c = 0 ϕ = 40° γ_m=16.0kN/m^3 γ_s=17.5 kN/m^3	9	20 ft	15 ft	R/S	1.56
		10	20 ft	0 ft	R/S	1.72
		11	20 ft	15 ft	C/S	2.19
		12	20 ft	0 ft	C/S	2.19
		13	10 ft	7.5 ft	R/S	1.68
		14	10 ft	0 ft	R/S	1.73
		15	10 ft	7.5 ft	C/S	2.19
		16	10 ft	0 ft	C/S	2.19

Note : γ_m = Moist unit weight of soil; γ_s = saturated unit weight of soil;
R/S = River side; C/S = Country side

A two-layer soil model has been used. The stability of the embankment was evaluated for a degree of consolidation equal to zero. Circular failure surfaces were assumed for all the eight cases of analyses as mentioned above. All together sixteen analyses were carried out. Table 5 shows the values of factor of safety of the embankment for the analyses performed. It can be seen from Table 5 that the factor of safety of the embankment on the riverside for heights of 20 ft and 10 ft varied from 1.56 to 2.10 and 1.68 to 2.77, respectively. However, the factor of safety of the embankment on the countryside has been found to be higher than those obtained for the riverside. This has been attributed to low height of the embankment on the countryside and also due to the presence of berm on the countryside. The factor of safety of the embankment on the countryside has been found to vary from 2.19 to 5.26. The results of stability analyses, therefore, indicate that that the embankment has adequate factor of safety against bearing capacity failure and that the embankment has adequate overall stability.

CONCLUSIONS

During 1998 floods considerable damages occurred to the Rajshahi Town Protection Embankment. The 18 km embankment and its components were threatened during the 1998 floods. Seepage, piping, sliding and rain cuts occurred at a number of sections of the embankment during the occurrence of flood. Major damages took place at the T-Groyne and the Shashanghat Closure (I-Groyne). About 200 m^2 of downstream shank of the T-Groyne and a considerable portion from the head of the I-Groyne were damaged. The netting of brick blocks was teared off which needed major repairs. The current took about 150 ft section of the nose of closure away. The left bank of the Padma eroded severely, specially, at 2 km downstream of the T-Groyne and in some places in the upstream.

A number of protective measures were undertaken to prevent seepage and erosion of the embankment, groynes, spurs and closures which included bamboo pilling, dumping of rock boulders, sand bags, brick crates and brick bats. Polythene cover was used to prevent erosion due to rain cut. With the implementation of the various protective measures, it was possible to protect the embankment from catastrophic failure. Due to considerable damage of the embankment and its components, re-sectioning and thorough repairing of the flood embankment had to be done to raise the embankment section up to the design level.

In December 1998, Bureau of Research, Testing and Consultation (BRTC), Bangladesh University of Engineering and Technology, Dhaka carried out field and laboratory investigations. The field investigations at the embankment consisted of drilling of boreholes, performing Standard Penetration Test (SPT), collection of sufficient numbers of disturbed and undisturbed tube samples, and performance of in place density tests. A total of five boreholes were drilled. A detailed laboratory investigation was carried out on soil samples collected from the boreholes. The cohesive samples obtained from the boreholes are typically inorganic clays and silts of low to medium plasticity (USCS Symbols are CL and ML) while the sandy samples are either SM or SP-SM. The values of the optimum moisture content and maximum dry density of the samples collected from four selected locations of the embankment varied from 15.1 % to 19.5 % and 100.5 lb/ft^3 to 110.4 lb/ft^3, respectively. In-place density of the embankment soil was determined at ten spots of the above-mentioned four locations of the embankment. The *in-situ* dry density and compaction of the soil samples at four locations of the embankment varied from 84.3 lb/ft^3 to 105.1 lb/ft^3 and 79.5 % to 95.2 %, respectively. The average value of the degree of field compaction at the ten spots was found to be 85.3 %, indicating that the embankment soils are still in a state of fairly good degree of compaction. Direct shear tests conducted on five undisturbed samples indicated that the angle of internal friction varied from 27° to 41°. On the basis of the value of undrained shear strength, two samples tested from two boreholes were found to be soft and firm, respectively. The values of compression index (C_c) and initial void ratio (e_0) of a sample has been found to be 0.17 and 0.69, respectively. Low values of C_c and e_0 indicate that the compressibility of the sample is low. Depending on the stress range, the values of coefficient of consolidation and coefficient of volume compressibility of the soil sample have been found to be in the range of 1.49 x 10^{-3} to 3.71 x 10^{-3} cm^2/sec and 0.88 x 10^{-4} to 9.8 x 10^{-4} m^2/kN, respectively. Depending on the void ratio, the values of coefficient of permeability of the soil sample varied from 1.29 x 10^{-10} m/sec to 3.29 x 10^{-9} m/sec. The relationship between void ratio and permeability has been found to be approximately linear. The average slope of the relationship, termed as permeability change index, C_k is 0.13 for this sample and the ratio of compression index to permeability change index (C_c/C_k) for the sample is 1.3.

Stability analyses were carried out to evaluate the factor of safety against bearing capacity failure of the embankment for a number of cases. Deep stability of the embankment on the river side and country side during both high water and low water periods have been investigated using XSTABL slope stability program. A total of sixteen analyses were carried out. The factor of safety of the embankment on the riverside for heights of 20 ft and 10 ft varied from 1.56 to 2.10 and 1.68 to 2.77, respectively. The factor of safety of the embankment on the countryside has been found to vary from 2.19 to 5.26, which are substantially higher than those on the riverside. This is due to low height of the embankment and also due to presence of berm on the countryside. The results of stability analyses indicate that although the embankment suffered considerable damages during the 1998 floods, the existing factor of safety against bearing capacity failure of the embankment is adequate.

The present investigations clearly demonstrate that although the embankment showed distresses, e.g., local erosion, rain cuts, sloughing, etc. at various sections, the overall performance of the embankment has been satisfactory. The satisfactory values of soil parameters, adequate *in-situ* density and timely undertaking of protective measures played key roles in the overall successful performance of the embankment during the 1998 floods.

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NOTATION

c	cohesion
c_v	coefficient of consolidation
C_c	compression index
C_k	permeability change index
e	void ratio
LL	liquid limit
m_v	coefficient of volume compressibility
PL	plastic limit
PI	plasticity index
q_u	unconfined compressive strength
w_{opt}	optimum moisture content
γ_d	dry density of soil
γ_m	moist unit weight of soil
γ_s	saturated unit weight of soil
ϕ	angle of internal friction
ε_f	axial strain at failure