ESTIMATION OF THE STABILITY OF EMBANKMENT SLOPES USING FIELD MONITORING DATA

Thiloththama Hiranya Kumari Nawarathna

118814X

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Department of Civil Engineering

University of Moratuwa
Sri Lanka

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Dr. L.I.N.De Silva

B.Sc. Eng (Moratuwa), M.Eng (Tokyo), Ph.D. (Tokyo)

Senior Lecturer,

Department of Civil Engineering,

University of Moratuwa,

Moratuwa,

Sri Lanka.

ABSTRACT

Estimation of the Stability of Embankment Slopes Using Field Monitoring Data

In Sri Lanka lands underlain by soft, weak and problematic soil are being use for various constructions due to rapid development and the lack of suitable lands. Construction of an embankment over soft soil is challenging due to its low shear strength and high compressibility nature. As a result of that embankment can be subjected to a shear failure or excessive settlement. Assessment of the stability of the embankment is vital to ensure a safe embankment and stability can be evaluated by using available analytical methods or field monitoring data. Use of field monitoring data to evaluate the stability is easy and more practicable. In Sri Lanka Matsuo chart which is based on the field monitoring data was used for the prediction of stability of the embankments in the Colombo Katuanayaka highway project. However applicability of Matsuo chart for various embankment conditions is still questionable. This study was carried out to investigate the applicability of Matsuo chart for various embankment conditions using advanced numerical tools. Two test embankments published in the literature and three embankments, which belong to the Colombo - Katunayaka expressway were analyzed by using Finite Element Method, Limit Equilibrium Method and Matsuo chart and compared with the field data. Further, the effect of embankment width, height and the subsoil parameters on the prediction of stability using Matsuo method was investigated. Research finding verify that the Finite Element Method, Limit Equilibrium Method and Matsuo chart can predict the stability of the embankment accurately and Finite Element Method can be used to predict the deformation characteristics. Stability of the embankment is directly proportional to the un-drained shear strength of the sub soil and Factor of Safety values decrease with the increase of the embankment height. However it was found that the embankment width has little influence on the factor of safety.

Key words: embankment, Matsuo chart, finite element method, limit equilibrium method, soft soil

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LIST OF ABBREVIATIONS

Abbreviation	Description
CKE	Colombo Katunayaka Expressway
FEM	Finite Element Method
FOS	Factor of Safety
LEM	Limit Equilibrium Method
MC	Mohr-Coulomb
NC	Normally Consolidated
OC	Over Consolidated
SS	Soft Soil
SSC	Soft Soil Creep

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CHAPTER 1 INTRODUCTION

1.1 Introduction to Research

Embankment is the one of most important type of structure, which is required in the construction of highways and railways, dams and retention dikes, harbor installations and airports. Due to urbanization and rapid growth of population lands available for the infrastructures and other developments have been decreasing. As a result of that, weak and problematic subsoil are also used in constructing the earth structures.

Design, construction and maintenance of an embankment over a low strength and high compressible sub soil is an engineering challenge due to associated low bearing capacity and the excessive settlement. Short-term stability of embankment over soft soil is critical than the long term stability because, the consolidation of the sub soil under loading tends to increase the strength of the sub soil with time. Further evaluation of the magnitude and the rate of settlement of the sub soil is also important, in order to design a safe embankment.

Improvement of the sub soil and assessment of the stability of the embankment is vital due to all the facts mentioned before. Many researches have been attempted to determine the factor of safety before the construction of embankments on soft sub soil. Stability of the embankment during the construction and after the construction can be evaluated by using available analytical methods or field monitoring data. Most sections of the Highway network in Sri Lanka have been constructed on weak soft soil deposits. Generally failure of the embankment will occur when the progress of the shear deformation is faster than that of consolidation settlement.

As mentioned earlier stability of the embankment can be evaluated by using analytical methods or field monitoring data. Analytical methods include the Limit Equilibrium method and Finite Element method. In limit equilibrium method, appropriate failure mechanism is assumed and the factor of safety is determined. However it doesn’t give any idea about the stress and deformation within the soil mass. In Finite Element method soil mass is divided into number of small elements and appropriate stress strain behavior is assumed for the soil element. If different construction stages and material behavior are simulated correctly and accurately in to the analysis, FEM can provide good prediction of the soil structure interaction problem.

In FEM, stress and deformation within the soil mass can also be obtained. Today more sophisticated computer programs are available to conduct these analyses. As a result applying FEM is more comfortable and easy. However obtaining of field monitoring data to predict the stability of the embankment is more practical.

Matsuo and Kawamura (1975) proposed a diagram (Matsuo chart) for prediction of failure of soft ground by observing the settlement at the center of the embankment and the lateral displacement at the toe of the embankment. This is an observational method and basic parameters incorporate with the Matsuo chart are easy to monitor. Vertical settlement at the center of the embankment represents the consolidation settlement and the horizontal displacement at the toe of the embankment represents the shear deformation.

In Sri Lanka Matsuo chart was used in the prediction of stability of the embankments in the Colombo Katuanayaka highway project. Handling the Matsuo chart is easy and more practicable, but applicability of Matsuo chart for various embankment conditions is still questionable. Furthermore despite the advantages of FEM approach over LEM approach Limit equilibrium method is still widely used in the evaluation of stability of slopes.

Therefore this study is focused on investigating the applicability of Matsuo chart, LEM and FEM methods in predicting the stability of embankment slopes.

1.2 Objectives

The objectives of the present study are as follows,

1) Conduct a thorough literature review on the area of study.

2) Investigate the applicability of Finite Element Method (FEM) and Limit Equilibrium Method (LEM) in predicting the stability of embankment slopes.

3) Investigate the effects of embankment width, height and sub soil parameters in predicting the stability of embankment slopes using field monitoring data.

CHAPTER 2 LITREATURE REVIEW

Limited number of researches had been carried out earlier to investigate the applicability of the FEM, LEM and the Matsuo chart on the prediction of the stability of the embankment slopes.

Indraratna, Balasubramaniam & Balachandran (1992) studied about the observed and predicted performance of a full scale trial embankment which was constructed on soft Malaysian marine clay. Deformation of the sub soil, critical fill height and corresponding failure surface were predicted by using modified cam-clay theory (CRISP) and hyperbolic stress-strain model (ISBILD) and compared with the field measurements. Purely drained, purely un-drained and couple consolidated analyses were conducted to predict the behavior of the embankment.

Sub surface profile under the Muar plain and the corresponding shear strength and consolidation parameters are shown in the Figure 2.1.

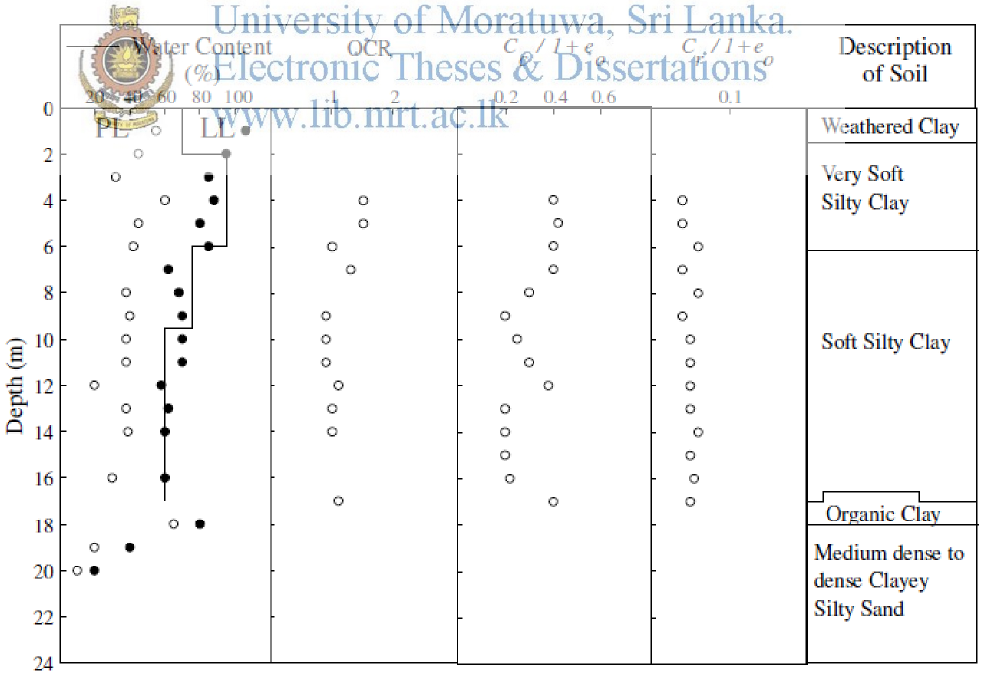


Figure 2.1: Sub soil profile of the Muar plain

Source : Indraratna et.al, 1992

Soil parameters used for modified cam-clay theory (CRISP) and hyperbolic stress-strain model (ISBILD) are given in the Table 2.1 and Table 2.2.

Table 2.1: soil parameters for modified cam-clay model (CRISP)

Depth	k	λ	M	v	e_{cs}	Kw $*10^4$ (cm^2/s)	γ (kN/m)	k_h*10^{-9} (m/s)	k_v*10^{-9} (m/s)
0-2.0	0.05	0.13	1.19	0.3	3.07	4.4	16.5	1.5	0.8
2.0-8.5	0.05	0.13	1.19	0.3	3.07	1.1	15.5	1.5	0.8
8.5-18	0.08	0.11	1.07	0.3	1.61	22.7	15.5	1.1	0.6
18-22	0.10	0.10	1.04	0.3	1.55	26.6	16.1	1.1	0.6

Source: Indraratne et al., 1992

Table 2.2 : soil parameters for hyperbolic stress-strain model (ISBILD)

Depth	k	C_u (kPa)	K_{ur}	C' (kPa)	Φ' (degree)	γ (kN/m)
0-2.0	350	15.4	438	8	6.5	16.5
2.0-8.5	280	13.4	350	22	13.5	15.5
8.5-18	354	19.5	443	16	17.0	15.5
18-22	401	25.9	502	14	21.5	16.1

Source : Indraratne et al., 1992

Embankment was constructed with the rate of 0.4m/week. Following parameters were used for the embankment surcharge,

Table 2.3 : Soil parameters for embankment surcharge

E	v	γ	c'	Φ'
5100kPa	0.3	20.5kN/m3	19kPa	26^0

Source: Indraratne et al., 1992

Variation of the excess pore water pressure beneath the embankment and the lateral and vertical settlement of the embankment were measured through the monitoring instruments. Locations of the monitoring instruments are shown in the Figure 2.2.

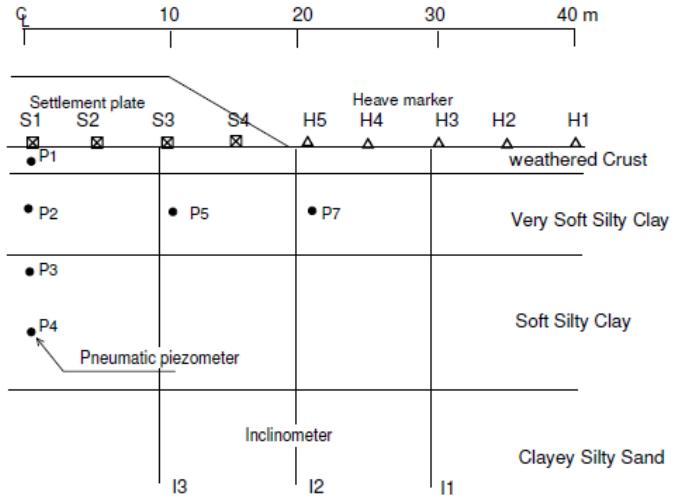


Figure 2.2 : Cross section of the Muar embankment with monitoring equipment

Source: Indraratne et al., 1992

Comparison between the predicted and measured surface settlement for various fill heights are shown in Figure 2.3. When the fill height is 2m, the un-drained analysis by ISBILD agrees well with the observed values. But for greater fill heights, coupled consolidation analysis provides better results.

Figure 2.4 shows the Variation of lateral displacement at the critical height of the embankment. According to the Figure 2.4 maximum lateral displacement is appear in the upper very soft clay layer. Modified cam-clay theory (CRISP) agrees well with observed details for the upper soft clay layer, but at greater depths it deviate from the observed values.

Indraratne et al. (1992) concluded that finite element method can be used to predict the correct mode of failure by using incremental displacement vectors and mobilized shear stress contours. Figure 2.5 shows the predicted shear band based on the maximum incremental displacement.

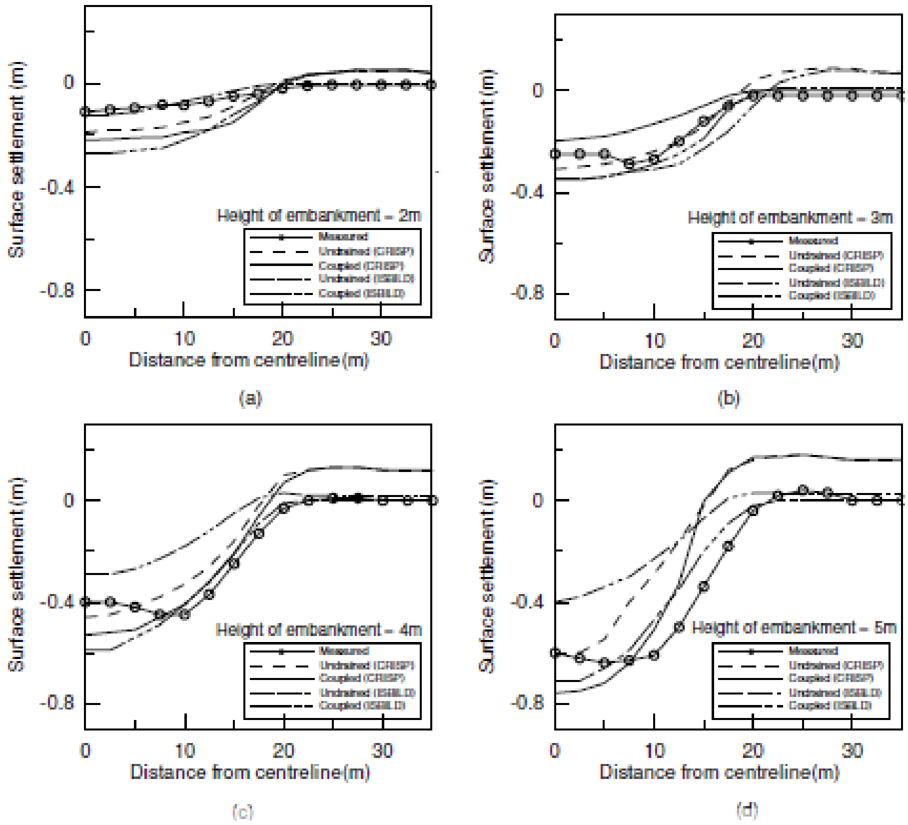


Figure 2.3: surface settlement for various fill heights

Source: Indraratne et al., 2005

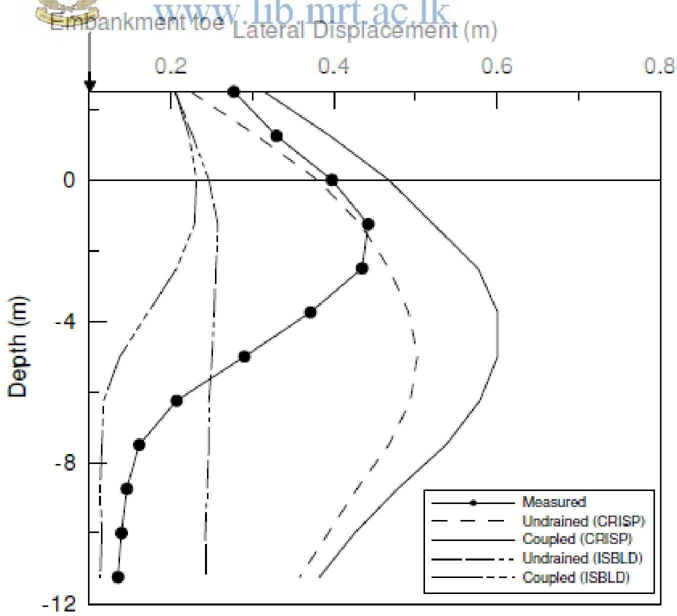


Figure 2.4: Variation of lateral displacement

Source: Indraratne et al., 2005

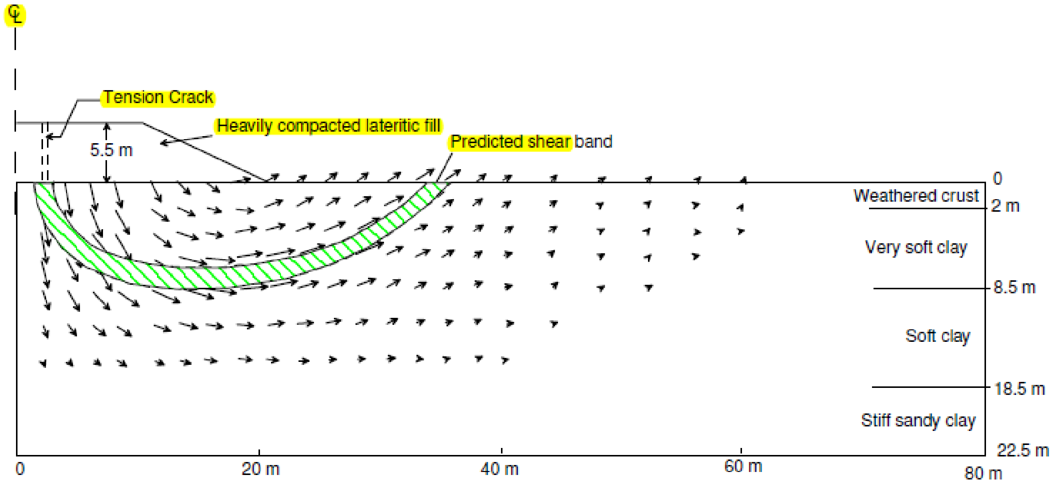


Figure 2.5: Maximum incremental development at failure

Source: Indraratne et al., 2005

Aziz (2010) conducted a research to evaluate the deformation and stability criteria of a reinforced embankment on soft clay. Full scale instrumented embankment was constructed at the research center of soft soil Engineering, University of Tun Hussein Onn, Malaysia. Performances of the embankment were analyzed by using finite element method (PAXIS 2D) and the Limit equilibrium method (SLOPE/W). Then predicted values were compared with the field values.

Table 2.4 shows the properties of the sub soil materials under the embankment and the Figure 2.6 shows the geometry model of the embankment.

Table 2.4 : Material properties under the trial embankment

Parameter	Very soft to soft silty CLAY	Dark greenish grey silty CLAY	Dark grey silty CLAY	Whitish grey and Firm silty CLAY	Fill
Material model	MC	MC	MC	MC	MC
Type of behavior	Un-drained	Un-drained	Un-drained	Un-drained	Un-drained

Dry soil unit weight (kN/m^3)	12.52	13.48	11.55	14.44	16.6
Sat. soil unit weight (kN/m^3)	15.78	16.52	14.61	17.27	18
Horizontal permeability (m/day)	14.29*10^{-4}	16.26 *10^{-4}	14.705*10^{-4}	10.109*10^{-4}	0.04
Vertical permeability (m/day)	7.145 *10^{-4}	8.130 * 10^{-4}	7.353*10^{-4}	5.054*10^{-4}	0.04
Young's modulus (kN/m^2)	1286	1724	1088.44	1465.33	2000
Poisson's ratio	0.35	0.35	0.35	0.35	0.30
Cohesion (kN/m^2)	15	17	37.3	20	10
Friction angle	2	1	4.9	10	23.54
Dilatancy angle	0	0	0	0	0

Source: Aziz, 2010

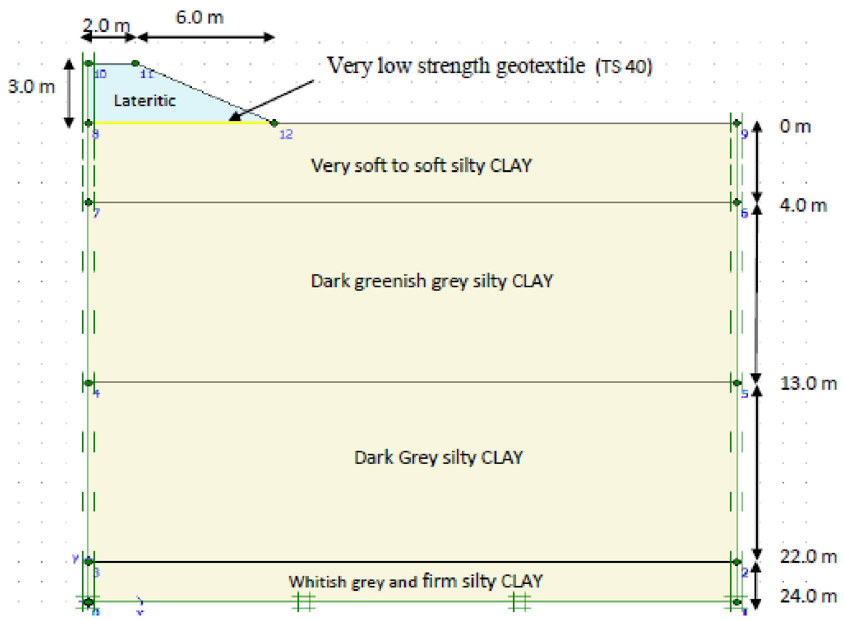


Figure 2.6 : Geometry model of the embankment using PLAXIS

Source: Aziz, 2010

Comparison between the predicted data (PLAXIS) and the observed data is shown in the Table 2.5.

Table 2.5: Comparison between predicted and observed data

Parameter	PLAXIS modeling		Instrumentation data	
Day	17 (During construction)	231 (After the construction)	17 (During construction)	231 (After the construction)
Excess pore water pressure (kN/m^2)	21.85	15.81	23.47	11.5
Settlement (m)	0.249	0.307	0.331	0.634
Lateral movement (m)	0.232	0.241	0.0054	0.0123

Source : Aziz, 2010

According to the above results, there is a significant difference between the predicted data and the observed data. Observed settlement values are higher than the predicted data and predicted lateral displacement values are much higher than the observed values. Aziz (2010) mentioned that this is probably due to the inaccuracies of the soil properties used for modeling. Aziz (2010) measured Factor of safety values by using PAXIS 2D and SLOPE/W programs to predict the stability of the embankment. Table 2.6 shows the predicted factor of safety values.

Table 2.6: Factor of safety values

Factor of Safety		
PLAXIS		SLOPE/W (Morgentsern and price method)
During construction	After construction	
1.798	1.872	1.893

Source: Aziz, 2010

Factor of safety in Finite element method is smaller compared to the factor of safety obtained from the SLOPE/W modeling. Aziz (2010) mentioned that PLAXIS computes the inter-slice forces more accurately by taking into account the local stress distribution in soil mass.

Maula & Zhang (2011) evaluated the stability of the homogenous soil slope by using FEM and the LEM. Height and the angle of the slope are equal to the 6m and 45^0. Shear strength parameters were varied from 5, 10, and 20 kPa while friction angle varied from 5,25and 45. Unit weight of the soil layer was kept 20 kN/m^3. Geometry of the slope is shown in Figure 2.7.

Factor of safety values obtained from the FEM and the LEM are given in Table 2.7. For most cases results given from the LEM and FEM are very similar. Maula et al., (2011) found that FOS values obtained from the two programs increases with increasing friction angle. However difference between the two programs is small. When the cohesive strength of soil is small, the differences in FOS between the two programs results are greater for higher friction angles. For larger shear strength values differences in FOS between the two programs results are lowest for higher friction angles.

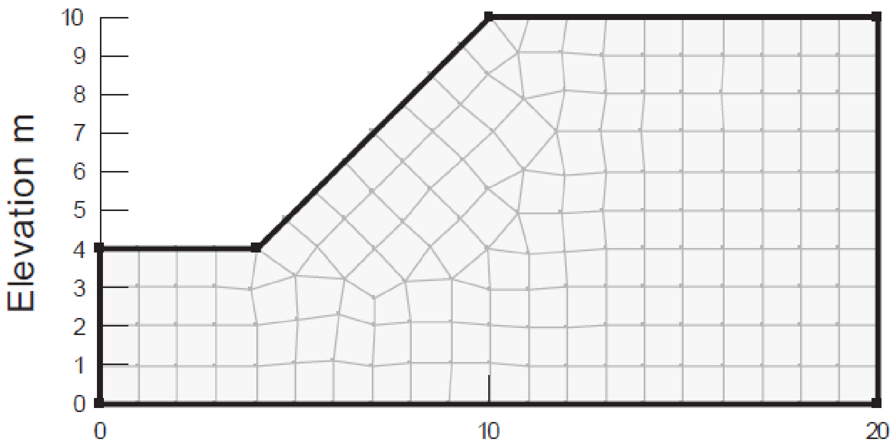


Figure 2.7: Geometry of the slope model

Source: Maula et al., 2011

Table 2.7: Factor of safety values using FEM and LEM

Case	C(kPa)	φ	FOS(Geo studio 2007) using LEM method	FOS (Plaxis 2D) using SRM method	FOS differences with Geo2007(Plasix) %
1	5	5	0.42	0.43	2
2	5	25	1.029	1.03	0.09
3	5	45	1.86	1.68	10
4	10	5	0.7	0.71	1.4
5	10	25	1.32	1.36	3
6	10	45	2.165	2.05	5.3
7	20	5	1.2	1.24	3.3
8	20	25	1.89	1.95	3.2
9	20	45	2.75	2.67	2.9

Source: Maula et al., 2011

Alkasawneh, Malkawi, Nusairat & Albataineh (2007) analyzed two embankments using FEM and LEM. Soil properties used for these two embankments are given in Table 2.8. Geometries of the three embankments and slip surfaces are shown in Figure2.8-2.9. Results obtained from the numerical analysis are given in Table 2.9.

SAS-MCT, UTEXAS3 and STABL5M programs represent the Limit Equilibrium analysis. Results obtained from the LEM are match well with the FEM. However Alkasawneh et al. (2007) stated that limit equilibrium methods are reliable and can be used with confidence to investigate the stability of slopes. Analysis using the finite element methods can be expensive and not justified unless other complex soil boundary conditions exist.

Table 2.8: soil properties used for the two examples

Example no.	Layer no.	γ (kN/m^3)	Cohesion (kPa)	ϕ (°)
1	1	17	15	20
2	1	18.2	80.0	0
	2	21.0	0.0	38
	3	18.2	100.0	0
	4	18.0	40.0	0
	5	16.9	95.0	0
	6	18.3	95.0	0

Source: Alkasawneh et al., 2007

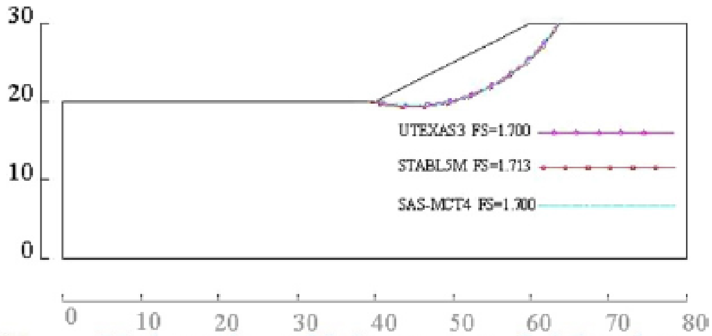


Figure 2.8: Geometry of slope and slip surfaces for example 1

Source: Alkasawneh et al., 2007

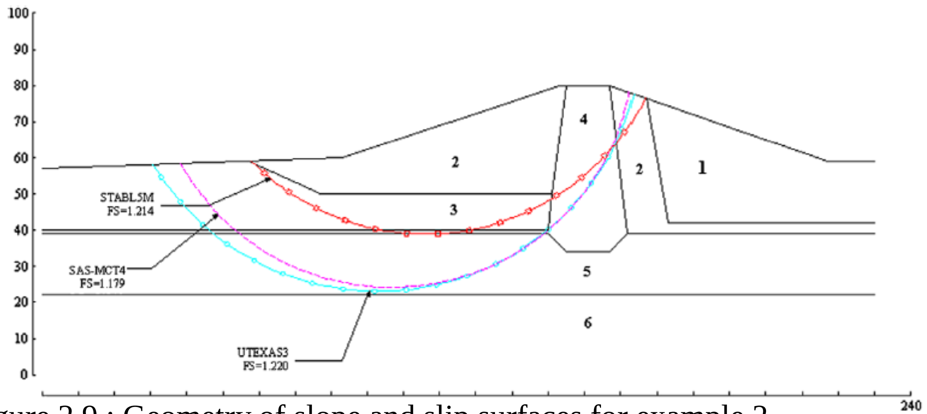


Figure 2.9 : Geometry of slope and slip surfaces for example 2

Source: Alkasawneh et al., 2007

Table 2.9: Factor of safety values obtained different slope stability

Example no.	SAS-MCT 4.0	UTEXAS3	STABL5M	PLAXIS 2D	PLAXIS 3D
1	1.70	1.70	1.713	1.70	1.80
2	1.179	1.22	1.214	1.14	1.22

Source: Alkasawneh et al., 2007

Rabie (2014) evaluated the stability of saturated and the unsaturated slope using FEM and LEM. Shear strength reduction technique was used in FEM and simplified Bishop method, simplified Janbu method and Fellenius method were used in LEM to estimate the factor of safety values. Results obtained are given in the Table 2.10.

Table 2.10: comparison between saturated and un-saturated slopes

Method	Factor of safety	
	Unsaturated	Saturated
Finite element method	2.547	1.953
Simplified Bishop method	1.094	0.722
Simplified Janbu method	1.093	0.728
Fellenius method	1.065	0.723

Source: Rabie, 2014

Rabie (2014) finally concluded that classical limit equilibrium methods are highly conservative compared to the finite element approach and for assessment of the factor of safety for slope using the FEM, no assumption needs to be made in advance about the shape or location of the failure surface, slice side forces and their directions.

Hammouri, Malkawi & Yamin (2008) studied about stability of the un-drained clay slopes using FEM and the LEM. Slope geometry of the model is shown in the Figure 2.10. Unit weight for both layers is 20 kN/m^3 and un-drained friction angle is zero. Trial embankment was analyzed for the different ratio of Cu1/γH (0.15, 0.2, 0.25, and 0.3) where H is the slope height then Cu2 value was calculated from the different ratios of Cu2/Cu1 from 0.5 to3. Figure 2.11 shows the results obtained for the different values of Cu2/Cu1 and the Cu1/γH. According to the results which were obtained from the FEM, FOS values are increase with the increase of the Cu2/Cu1 ratio. But in LEM, FOS value has increased up to the Cu2/Cu1 = 1.5 and afterward it becomes constant. Obtained factor of safety values are given in Table 2.11 and the difference between the FOS values are small.

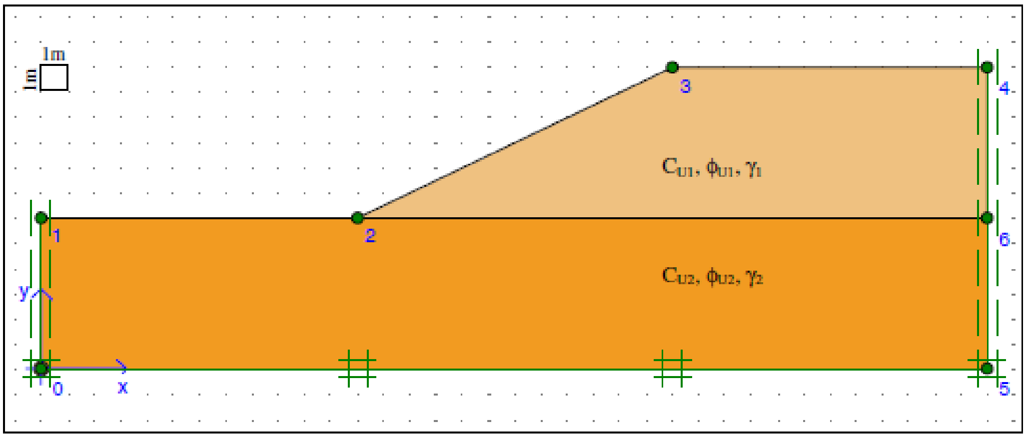


Figure 2.10: slope geometry of the un-drained clay slope

Source: Hammouri et al.,2008.

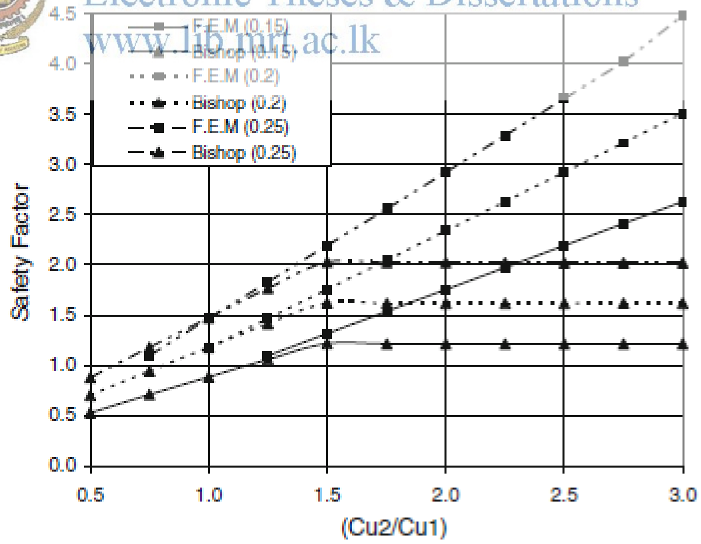


Figure 2.11: Comparison of the FEM and LEM for different values of Cu2/Cu1 and Cu1/γH = 0.15, 0.2, 0.25

Source: Hammouri et al., 2008

Table 2.11: comparison of the FOS values obtained from the LEM and FEM (a) Cu2/Cu1 =1 and (b) Cu2/Cu1 = 1.5

	Safety factor		LEM Method	Safety factor difference %
	FEM	LEM		
(a)	1.464	1.472	Bishop	1
(b)	2.193	2.026	Bishop	8

Source: Hammouri et al., 2008

Cheng, Lansivaara & Wei (2006) analyzed a homogeneous soil slope by using LEM and the Strength Reduction Method. Various models were evaluated by varying the shear strength parameters (c'& Φ'). Table 2.12 gives the obtained FOS values and Figure 2.12 gives the position of the failure surface.

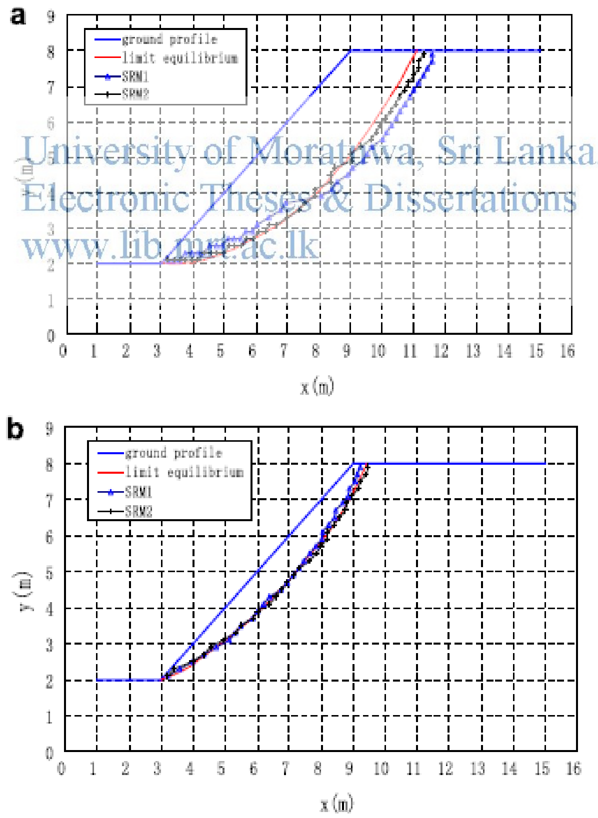


Figure 2.12: Slip surface comparison with increasing friction angle c'=2kPa; (a) Φ= 5^0 (b) Φ= 45^0

Source: Cheng et al., 2006

Table 2.12: Factor of safety values by LEM and SRM; SRM1- dilation angle=0 and SRM2- dilation angle= friction angle.

Case	c' (kPa)	ϕ' (°)	FOS (LEM)	FOS (SRM1, non-associated)	FOS (SRM2, associated)	FOS difference with LEM (SRM1, %)	FOS difference with LEM (SRM2, %)	FOS difference between SRM1 and SRM2
1	2	5	0.25	0.25	0.26	0	4.0	4.0
2	2	15	0.50	0.51	0.52	2	4.0	2.0
3	2	25	0.74	0.77	0.78	4.0	5.4	1.3
4	2	35	1.01	1.07	1.07	5.9	5.9	0
5	2	45	1.35	1.42	1.44	5.2	6.7	1.4
6	5	5	0.41	0.43	0.43	4.9	4.9	0
7	5	15	0.70	0.73	0.73	4.3	4.3	0
8	5	25	0.98	1.03	1.03	5.1	5.1	0
9	5	35	1.28	1.34	1.35	4.7	5.5	0.7
10	5	45	1.65	1.68	1.74	1.8	5.5	3.6
11	10	5	0.65	0.69	0.69	6.2	6.2	0
12	10	15	0.98	1.04	1.04	6.1	6.1	0
13	10	25	1.30	1.36	1.37	4.6	5.4	0.7
14	10	35	1.63	1.69	1.71	3.7	4.9	1.2
15	10	45	2.04	2.05	2.15	0.5	5.4	4.9
16	20	5	1.06	1.20	1.20	13.2	13.2	0
17	20	15	1.48	1.59	1.59	7.4	7.4	0
18	20	25	1.85	1.95	1.96	5.4	5.9	0.5
19	20	35	2.24	2.28	2.35	1.8	4.9	3.1
20	20	45	2.69	2.67	2.83	0.7	5.2	6.0
21	5	0	0.20		0.23		15	
22	10	0	0.40		0.45		12.5	
23	20	0	0.80		0.91		13.8	

Source: Cheng et al., 2006

According to the Table 2.12 and Figure 2.12 it can be seen that obtained FOS values and the critical failure surfaces are much similar under different combination of the soil parameters except when Φ = 0. When Φ=0, reasonable difference can be seen between the FOS values and the failure surfaces. For the most of the cases where Φ>0 FOS values obtained from the SRM differ by less than 7.4% with respect to the LEM results.

Chang & Huan (2005) studied about the reliability of the FEM under different conditions: Homogeneous slope, Slope under seismic actions, Slope with simple pore water pressure distribution, Slope with varied free surface and reservoir loading, Earth embankment, Slope with inhomogeneous soil layers. Results obtained are summarized in Table 2.13. According to the following Table 2.13, it can be seen that FOS values obtained from the FEM are lower than the Bishop's simplified method. Chang et al (2005) stated that "Although the differences between them are small, the proposed procedure can provide engineers a more solidly based concept of slope stability analysis"

Table 2.13: comparison of the safety factor for different conditions

Conditions		FEM	BSM	Relative difference (%)
1. Homogeneous		1.360	1.377	1.235
2. Seismic action with a horizontal acceleration of 0.25g		0.835	0.838	0.358
3. Pore pressure parameter r_u=0.25		1.079	1.106	2.441
4. Drawdown	L/H=0.5	1.333	1.340	0.522
	L/H=0.6	1.284	1.289	0.388
	L/H=0.7	1.252	1.257	0.398
	L/H=0.8	1.238	1.244	0.482
	L/H=0.9	1.240	1.254	1.116
	L/H=1.0	1.257	1.277	1.566
5. Earth embankment		1.805	1.814	0.496
6. Inhomogeneous		1.228	1.259	2.542
7. Varied dilation angle (ψ)		1.353(0)	1.370	1.241
		1.361(1/2ϕ)		0.657
		1.367(ϕ)		0.219

Source : Chang et.al,2005

Cala & Flisiak (2003) compared the applicability of the LEM and the FEM under different conditions.

- Simple, homogeneous slope

Embankments were simulated with slope angles ranging from 18.43° to 63.43°. The height of the embankment was changed from 15 m to 35 m. Angle of internal friction (φ) ranging from 10° to 30° and cohesion (c) from 25 kPa to 75 kPa. According to the results given in the Figure 2.13 it can be seen that FOS values are similar.

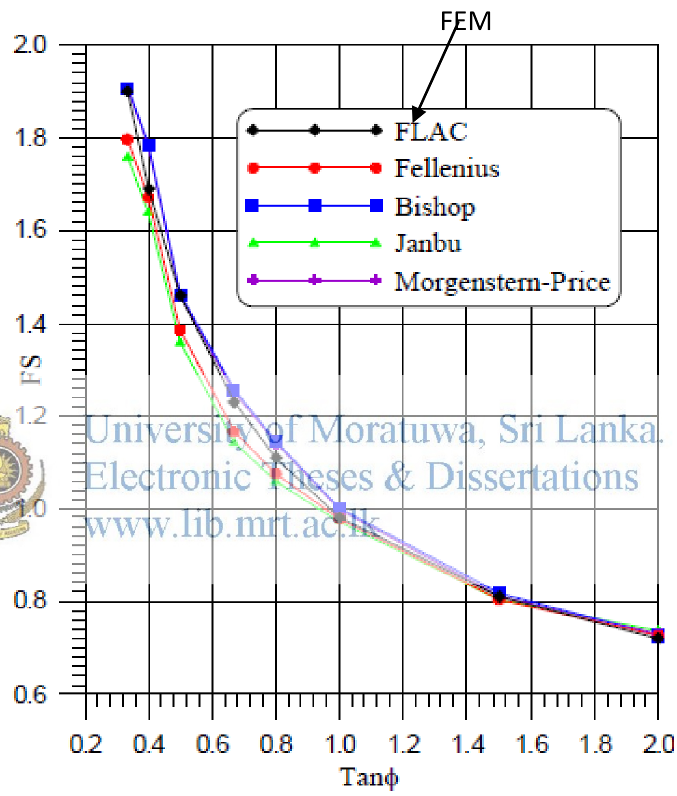


Figure 2.13: FS for embankment of height 25 m, friction angle φ = 20^{o}, cohesion c = 30 kPa for several sloping angles with FEM and LEM.

Source: Cala et.al, 2003

- Slope consisting of two different geological units

The soil below the embankment was given friction angle φ = 10° and cohesion c = 0. The stability of the embankment of height 25 m, friction angle φ = 20° and cohesion c = 30 kPa for several sloping angles was analyzed.

FOS values calculated for the different slope angles using FEM method and the LEM are given in Figure 2.14. FOS values obtained for the slope angle 18.43^0-41^0 are approximately similar but for slope angle 41° to 64.43° FOS calculated with FEM are 20% lower than FOS obtained from LEM. Cala et.al (2003) mentioned that, slip surfaces obtained from FEM are localized deeper than slip surfaces from LEM and it could be the reason for the above differences.

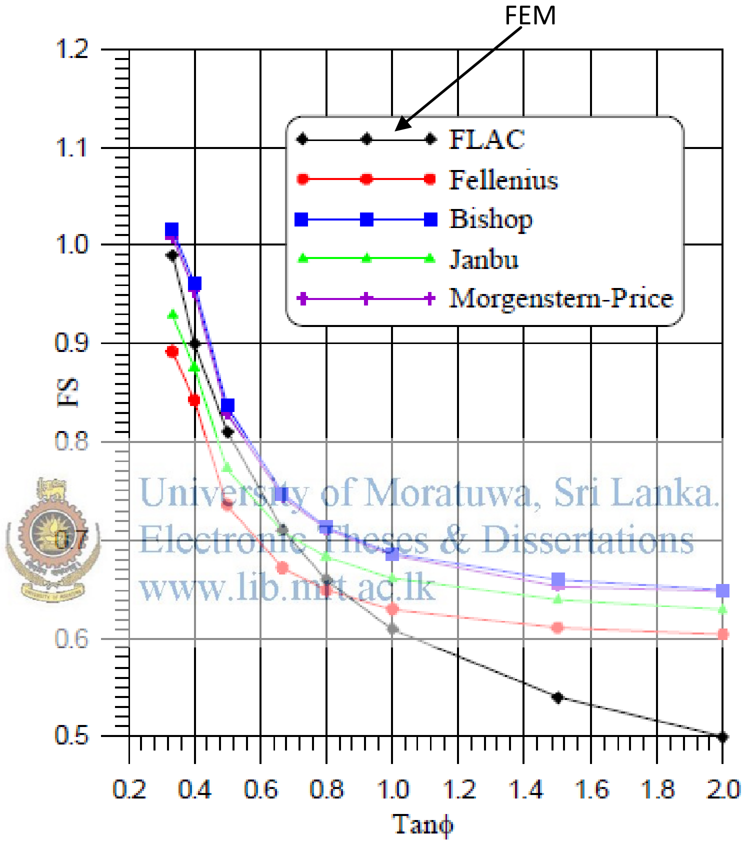


Figure 2.14: FOS for embankment (height 25 m) consisting of two geological units for several sloping angles with FEM and LEM.

Source: Cala et.al, 2003

- Slope with weak stratum

Geometry of the slope is given in Figure 2.15. Embankment is 25 m high and has a slope angle equal 45°. The soil was given friction angle =30° and cohesion c = 75 kPa. Friction angle and cohesion of the weak is equal to the 10° and 25 kPa. Unite weight of the both soils is 20 kN/m^3. The thickness "g" of horizontal weak layer was changed from 1.0 m to 10.0 m and its distance "h" from top of the slope changed from 0 to 50 m.

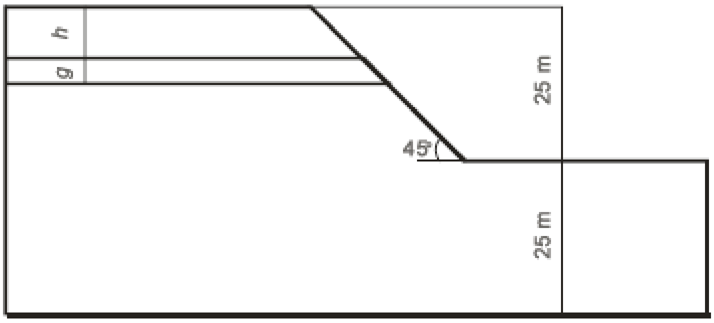


Figure 2.15: Slope with a weak layer

Source: Cala et.al, 2003

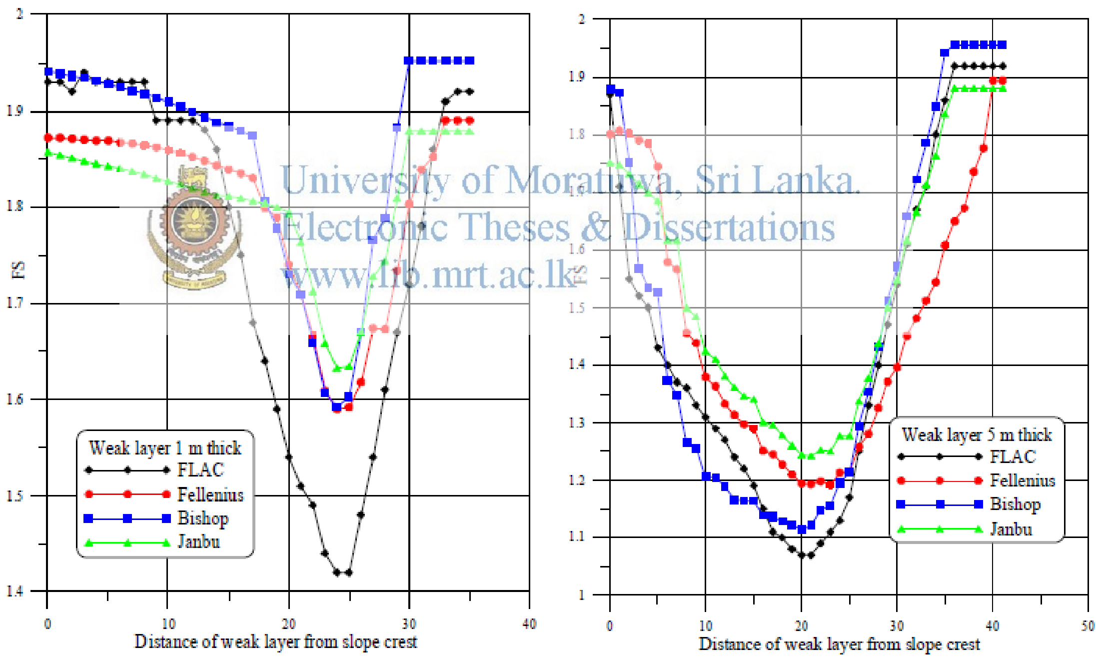


Figure 2.16: FOS values for a 1.0 m and 5.0m thick weak layer.

Source: Cala et.al, 2003

According to the results given in the Figure 2.16 it can be seen that increase of the weak layer thickness reduces the differences between FS values from LEM and

FEM. Finally Cala et.al (2003) concluded that in the case of complex geometry and geology slopes FEM is much more “sensitive” than LEM.

Chai, Igaya, Hino & Carter (2013) predicted the behavior of the test embankment using FEM and test results were compared with the field observations. This study involved with two types of predictions: Class A- conducted before and Class B – conducted after the embankment was constructed. Modified clam- clay model was used to represent the behavior of soft clay layer. Predictions were made in terms of settlements, lateral displacements and excess pore water pressure of soil layer. According to the obtained results, Class A predictions are in a poor simulation with the field performance. Chai et al.(2013) stated that this may be due to the over estimation of the yield stresses.

Huang, Fityus, Bishop, Smith & Sheng (2006) numerically analyzed the consolidation behavior of an instrumented embankment on a soft soil foundation by using a coupled, nonlinear, finite element analysis. Predictions were compared with the field data. Predicted values for the settlement match well with the observed data however numerical model over estimates the horizontal displacement and the pore water pressure.

Apimeteetamrong, sunitsakul & swatparinich (2007) stated that “with some adjustments on the engineering properties of the soft Bangkok clay, finite element analyses provide good estimated settlement of highway embankment” (p.7).

Gunduz (2010) constructed a test embankment at the Lilla Mellösa and Skå-Edeby and the deformation characteristics of the embankment were evaluated by using PLAXIS program. Finally they found that predicted data are much similar to the field measurements for both drained and un-drained conditions.

Fredlund & Scoular (1999) stated that “use of the finite element method yields more detailed information on stress state in the soil than is available from conventional limit equilibrium method”.

Premalal, Jayasinghe, Indrachapa & Thilakasiri (2012) evaluated the stability of three failed embankments using Matsuo chart. According to the Matsuo chart two embankments showed this instability condition and other failed section didn’t show this instability condition. Premalal et al. (2012) stated that it may be a sudden failure due to stockpiling of the fill material on the embankment or any other reason.

All of these previous studies talk about the applicability of the LEM and the FEM in predicting the stability of the embankment slopes and the reliability of the FEM in evaluating the deformation characteristics of the embankment. However limited researches have been conducted so far on the applicability of the Matsuo chart. But Matsuo chart is used by most of Asian countries to evaluate the stability of the embankment slopes. So applicability of the Matsuo chart need to be further investigated. This study is mainly focused on this area.

CHAPTER 3 EVALUATION OF THE STABILITY OF SLOPES

Construction of an embankment over soft soil is challenging due to its low shear strength and the high compressibility. It would often lead to sliding failure or settlement of sub surface. Some other problems with heaving, side flows and subsidence would also occur due to the fall of ground water levels. So Assessment of the stability of the embankment during construction and after the construction is essential.

3.1 Embankment failure

Embankments can be failed either by bearing capacity failure or excessive settlement. Due to the weight of the fill material stresses with in the foundation soil tends to increase and it will lead to increase the pore water pressure with in soil mass. However in soft soil dissipation of pore water pressure is slow due to low permeability. Because of that saturated soil undergo constant-volume deformations and it will increase the shear strain and the shear stress with in the soil below the embankment and beyond the toe of the embankment.

The aspect ratio of a slide or failure generally is used to classify the slope failure type. As presented in Figure 3.1, a rotational slide produces a failure surface with an aspect ratio in the range of 0.15 <D/L<0.33. D - Depth of the sliding surface perpendicular to the slope face, L - length of the sliding surface (Abramson et al, as cited in Griffiths et al, 2010).

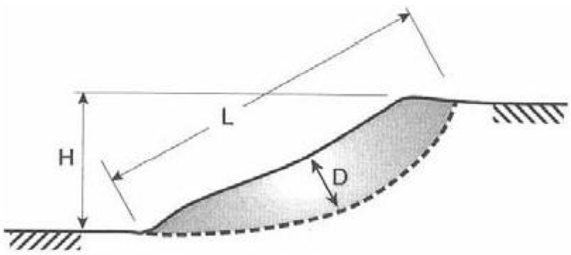


Figure 3.1: Aspect ratio of failure mass

Source: Griffiths et al, 2010

When the aspect ratio *D*/*L* < 15% or failure surface depth ≤ 10 ft., the slide is characterized as shallow. Hansen (as cited in Helwany & Titi, 2007) characterized shallow surface slips as those with *D*/*L* ranging from 3 to 6%.

3.1.1 Modes of the failure of the embankment on soft soil

A. Bearing capacity failure

Bearing capacity refers to the value of stress which a foundation soil can support safely without any sudden catastrophic settlement of the foundation soil due to shear failure (Bhattacharyya, 2009, p.22). Capacity of soil to support the loads applied to the ground is called as the bearing capacity failure. Risk of bearing capacity failure can be minimized by increasing the force resisting failure and strength of the sub soil through consolidation (Aziz, 2010).

B. Rotational slip failure

It includes a circular or noncircular path. Circular slips are formed in homogeneous, isotropic soil and noncircular slips are formed with nonhomogeneous soil (Craig, 2005). Failure surface is passing through the foundation soil and the embankment ("Guidelines on soft soils- Stage construction method", 2005).

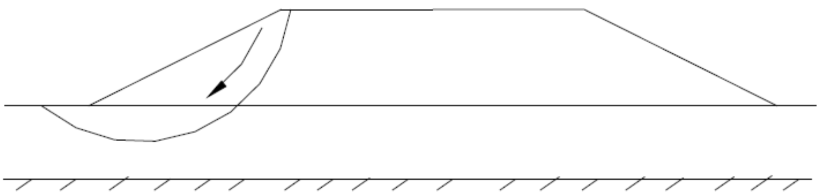


Figure 3.2 : Rotational slip failure

Source: "Guidelines on soft soils- Stage construction method", 2005

C. Lateral spreading

Lateral spreading is possible when the foundation soil subject to the large lateral pressure from the embankment because of that excessive horizontal sliding of embankments and foundation soils can be expected (Aziz, 2010). As shown in the following figure, soil wedge A B’ C slides along B’C due to the active pressure P1’ acting on the face A’B’. Failure occur when P1' >P2' (“Guidelines on soft soils- Stage construction method”, 2005).

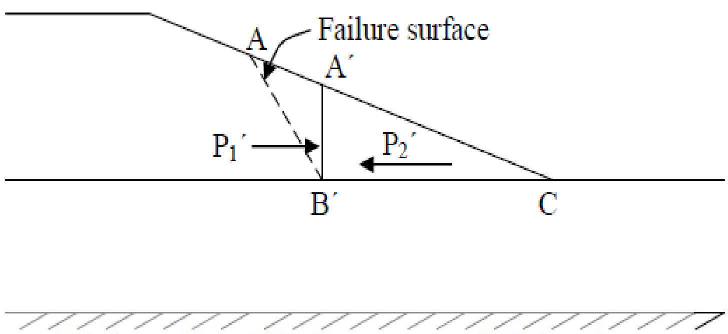


Figure 3.3: Lateral spreading

Source: “Guidelines on soft soils- Stage construction method”, 2005

D. Translational and compound slips

Occur where the form of the failure surface is influenced by the presence of an adjacent stratum of significantly different strength, most of the failure surface being likely to pass through the stratum of lower shear strength. The form of the surface would also be influenced by the presence of discontinuities such as fissures and pre-existing slips. Translational slips tend to occur where the adjacent stratum is at a relatively shallow depth below the surface of the slope, the failure surface tending to be plane and roughly parallel to the slope. Compound slips usually occurs where the adjacent stratum is at greater depth, the failure surface consisting of curved and plane sections (Craig, 2005, p.347).

3.1.2 Settlement of Embankment and Foundation Soil

Due to the high compressibility nature of the soft ground embankment can be failed by excessive settlement. Settlements take place both in the embankment and in the foundation on which embankment is constructed. Settlement of the embankment mainly can be divided in to two as immediate settlement and the consolidation settlement. Consolidation settlement consists with both primary consolidation and secondary consolidation.

Immediate settlement of the soil occurs immediately after the application of loads due to the deformation of the soil under un-drained condition. This associated with the shearing of materials at constant volume without any change in the water content of the sub soil.

In cohesive soils, immediate settlement occurs due to the distortion and the compression of air filled voids. It is anticipated that very little immediate settlement would occur in saturated cohesive soils. However, for unsaturated and/or highly over-consolidated (OCR $\geq$ 4) cohesive soils immediate settlement can be a predominant portion of the total settlement ("SCDOT geotechnical design manual", 2010). The immediate or distortion settlement although not actually elastic is usually estimated by using elastic theory (Holtz & Kovacs, 1981).

Consolidation is the gradual reduction in volume of a fully saturated soil of low permeability due to dissipation of excess pore water pressure. Compression of the soil due to dissipation of excess pore water pressure under the application of load is called as primary consolidation settlement. The determination of the amount of settlement is dependent on whether the soil is normally consolidated, over consolidated or a combination of both. Secondary compression is occurred after the complete of dissipation of excess pore water pressure. Secondary consolidation is a process of gradual readjustment of the clay particles into a more stable configuration. (Craig, 2005; Das, 2010).

Mesri, Stark, Ajlouni & Chen (1997) mentioned that Peat soil exhibits significant secondary compression due to the following reasons,

- High void ratio and the high natural water content
- Have a highest value of C_α / C_c ratio
- Primary consolidation occurs quickly due to initial high values of permeability as a result high value of compression index (C_c), so secondary compression start earlier than other soils.

3.1.2.1 Tolerable settlement

Hsi and Martin (2005) suggested the following tolerable limits of residual and differential settlement from their experience of the construction of the Yelgum to Chinderah freeway in Australia.

a) Residual settlement: Maximum allowable settlement of between 100-160 mm over 40 years.

b) Differential settlement: Maximum differential settlement in lateral direction 1% and the longitudinal direction 0.3% for 40 year period.

Long and O'Riordan (as cited in Bhattacharyya, 2009) proposed maximum allowable residual settlement of 350mm and differential settlement should not exceed 50mm after the operation of 25 years design life.

Das (2005) mentioned that when the embankment is constructed with non-compressible materials and each layer is well compacted, the settlement with in the embankment in the long term can be kept within 0.2% of the height of the embankment.

Hardman (as cited in Bhattacharyya, 2009) found a relationship between the settlement of the sub soil and the embankment height as shown in the Figure 3.4. Four embankment heights were considered for this analysis and noted that lower the embankment height produce large settlement.

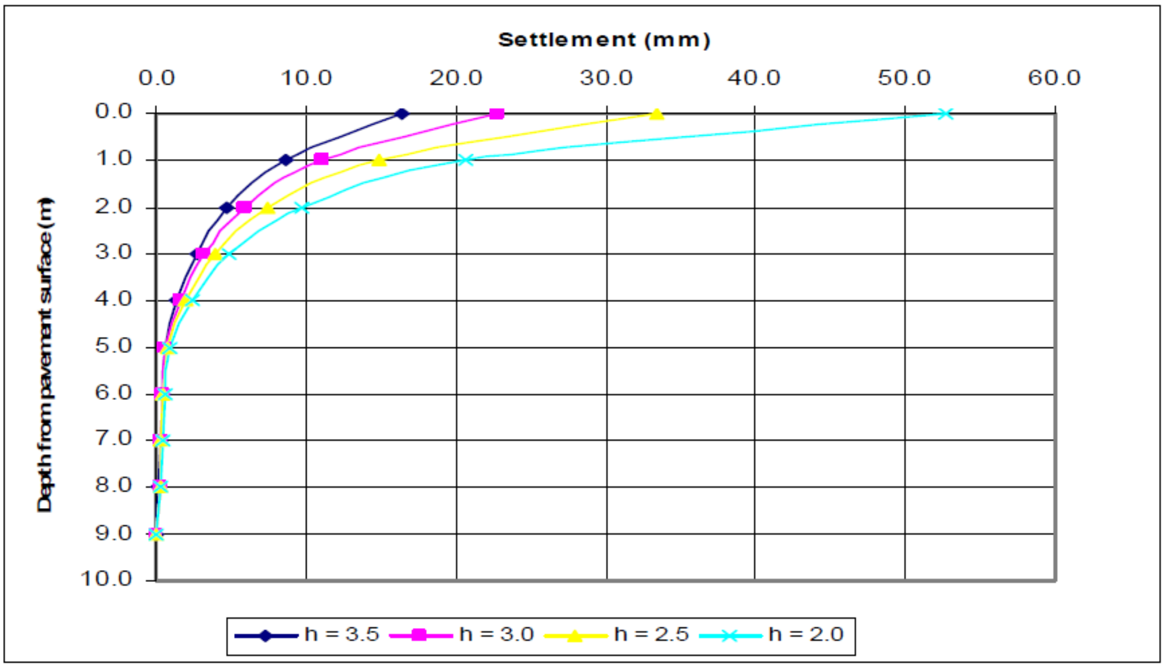


Figure 3.4: settlement of sub soil foundation due to construction traffic for various embankment heights
Source: Bhattacharyya, 2009

3.2 Factor of safety (FOS)

Factor of safety is defined as the ratio between the available shear strength of soil and the shear stress developed along a critical failure surface. Selecting a factor of safety value for a particular slope is depends on the including the level and accuracy of soil data, the experience of the design engineer and the contractor, level of construction monitoring, and consequence of slope failure (risk level). Abramson et al. (as cited in Helwany & Titi, 2007) indicated that for a typical slope design the required factor of safety ranges between 1.25 and 1.50.

Chin (2005) mentioned that the FOS on shear strength from total stress or un-drained strength analyses used in temporary stage is usually taken as between 1.2 to 1.3. Factor of safety of 1.4 and 1.5 are normally adopted in effective stress analyses of embankment for permanent stage.

Bjerrum (as cited in Das, 2010) found the variation of the Factor of Safety values with time with embankment on soft clay as shown in Figure 3.5.

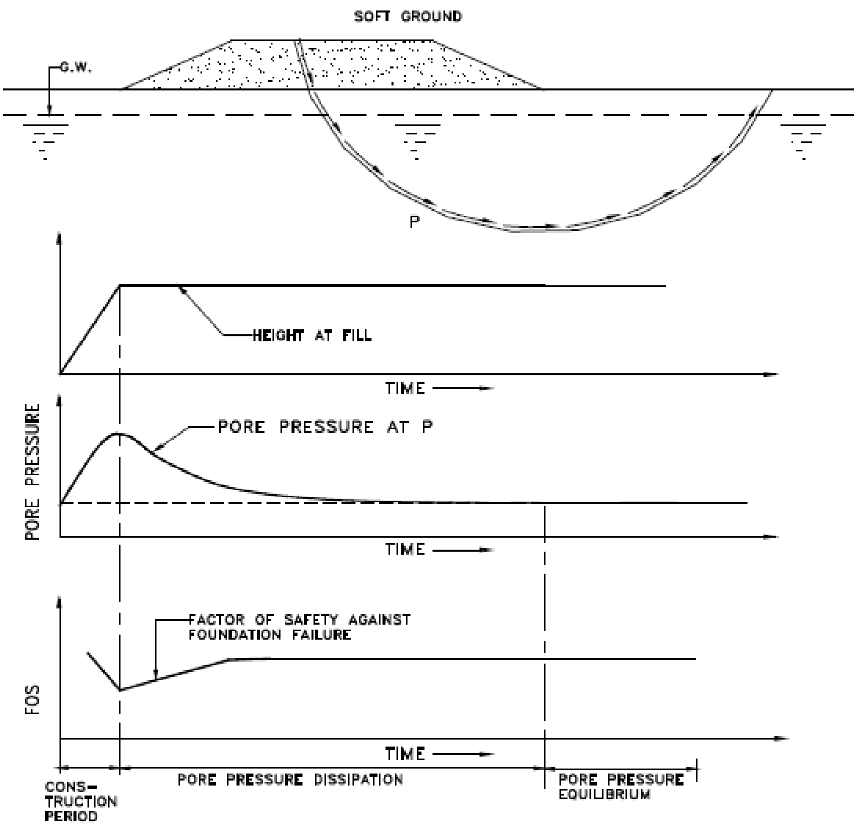


Figure 3.5: Factor of safety variation with time for embankment on soft clay

Source: Bhattacharyya, 2009

According to the above figure it can be seen that FOS values are decrease during the construction period but after that it increases with time and become constant. This is due to the soft soil gain strength with time due to consolidation.

3.3 Slope Stability Analysis

Slope stability analyses are conducted to assess the safe and economic design of manmade or natural slopes. Main objectives of slope stability analysis are finding endangered areas, investigation of potential failure mechanisms, determination of slope sensitivity to different triggering mechanisms, design an optimal slope with regards to safety, reliability and economics.

Geological information and site characteristics are required to design a successful slope. Here proper knowledge about the properties of soil, slope geometry and ground water condition is essential. Stability of the natural slopes or manmade slopes can be estimated by using Analytical methods or field monitoring data. Choice of correct analysis method depends on both site condition and the potential mode of failure.

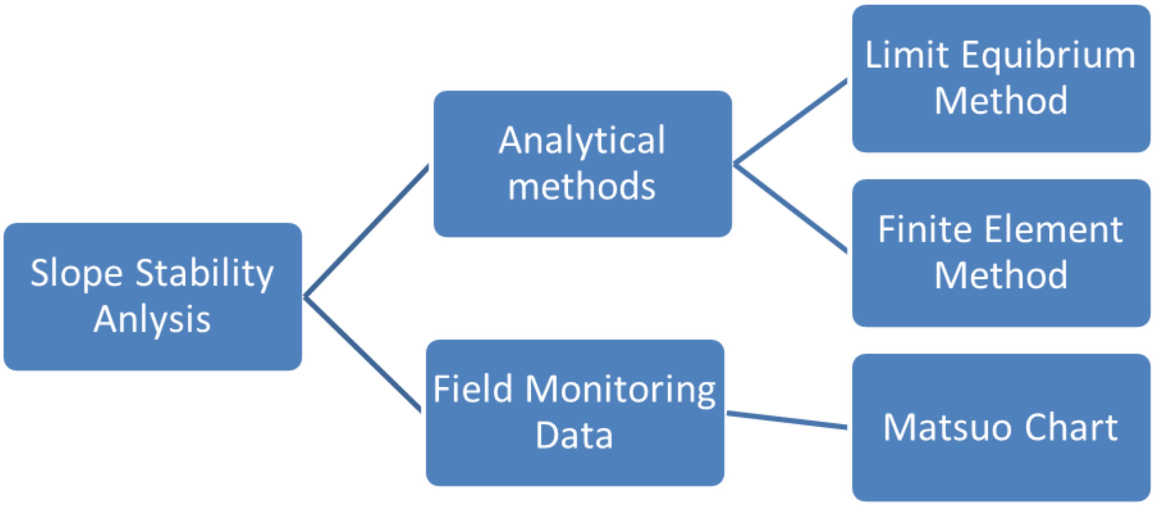


Figure 3.6: Methods of stability analysis

3.3.1 Limit equilibrium method

In limit equilibrium method appropriate different failure mechanisms are assumed and determine the factor of safety for each one of them. Minimum value of the factor of safety obtained is taken as the factor of safety for the slope. Corresponding failure surface is the critical failure surface.

The idea of discretizing a potential sliding mass in vertical slices was introduced early in the 20th century. In 1916, Petterson presented the stability analysis of the Stigberg Quay in Gothenberg, Sweden. Here circular type sip surface was taken and sliding mass was divided into the slices. After that Fellenius introduced the Ordinary or Swedish method of slices. In the mid1950s Jumbu developed a method for generic slip surfaces and Bishop developed a method for circular slip surfaces. In the 60s and 70s most methods were invented ,some making the limit equilibrium method a more powerful and refined tool of analysis of slope stability (Spencer, Morgenstern &

Price, Sarma methods) and other making it more suitable for hand calculations (Krahn,2004).

The introduction of powerful desktop personal computers in the early 1980s made it economically viable to develop commercial software products based on these techniques. Today modern, user friendly limit equilibrium soft wares such as Slope/W are available and then it is easy to apply the limit equilibrium theories into practice.

Due to this modern limit equilibrium software, now it is possible to deal with complex stratigraphy, highly irregular pore-water pressure conditions, and various linear and nonlinear shear strength models, almost any kind of slip surface shape, distributed or concentrated loads, and structural reinforcement (Rahman, 2012).

Slope/w has a variety of options for specifying trial slip surfaces. The position of the critical slip surface is affected by the soil strength properties. The position of the critical slip surface for a purely frictional soil (c = 0) is radically different than for a soil assigned untrained strength ($\varphi = 0$). This complicates the situation, because it means that in order to find the position of the critical slip surface, it is necessary to accurately define the soil properties in terms of effective strength parameters. Slope/w makes no distinction between effective strength and total strength parameters.

3.3.1.1 Methods of limit equilibrium stability analysis

Modern computer programs are consisting with the No of different analysis methods (Bhattacharyya, 2009; Rahman, 2012; “Geo studio tutorials”; 2004; chin, 2005).

1. Ordinary method of slices

This method called as the Swedish Method of Slices or the Fellenius Method. Here assume a circular slip surface and neglect all inter-slice forces and only satisfies moment equilibrium. This method is over conservative, because safety obtained by this method is found to be usually over than the over bound of solutions that satisfy statics. Ordinary method of slices is highly inaccurate for effective stress analyses of flat slopes with high pore pressures.

The method is perfectly accurate for $\Phi = 0$ analyses, and quite accurate for any type of total stress analysis using circular slip surfaces. However this is one of the simplest procedures based on the method of slices and hand calculations are possible.

2. Bishop Simplified method

This method satisfies the moment equilibrium and assumes a circular failure surface. In this method inter-slice shear forces are ignored but inter slice normal forces are considered. Wright, Kulhawy, Duncan (1974) compared Bishop Simplified method with linear and nonlinear finite element analyses and for all the case studied the difference was found to range between 0-8%.

3. Janbu simplified method

Similar to the Bishop Simplified method in here also inter-slice shear forces are ignored and inter slice normal forces are considered. But account the effect of the inter slices forces using a correction factor. This correction factor is related to cohesion, angle of internal friction and the shape of the failure surface. This satisfies only horizontal force equilibrium and considers a noncircular slip surface. Here also hand calculations are possible.

4. Spencer method

This method considers both normal and shear inter slices forces, and satisfies both force and moment equilibrium. The unique condition in Spencer's method is that the ratio of shear to normal inter-slice forces is a constant. Slope /W calculate two factor of safety, one with respect to the moment equilibrium and other one with respect to the horizontal force equilibrium. Spencer's method was found by the US Bureau of Reclamation to be 'useful and cost effective' for design work.

5. Morgentsern and price method

This method can be used to circular or noncircular failure surfaces. This method is similar to the Spencer's method, except that the inclination of the inter-slice resultant force is assumed to vary according to a "portion" of an arbitrary function. On the other hand Morgentsern and price method allows us to specify different types of inter-slice force functions. This method is not suitable for hand calculations.

6. General Limit Equilibrium method

This is a combination of all other methods and can be used to satisfy either force or moment equilibrium, or if required, just the force equilibrium conditions. This encompasses most of the assumptions used by various methods and may be used to analyze circular and noncircular failure surfaces.

7. Sarma method

Failure surface is noncircular and this is a modification of the Morgentsern and price method. This includes small no of iteration and need small computing time. The static factor of safety is obtained by reducing the soil shear strength parameters. This method is not suitable for hand calculations.

Krahn (2004) summarized the above mentioned methods as shown in the Table 3.1

Table 3.1 : Limit equilibrium methods

Method	**Moment equilibrium**	**Force equilibrium**	**Inter-slice Normal**	**Inter-slice Shear**
Ordinary method	yes	No	No	No
Bishop Simplified method	yes	No	yes	No
Janbu simplified method	No	yes	yes	No

Spencer method	yes	yes	yes	yes
Morgentsern and price method	yes	yes	yes	yes
Corps of engineers 1	No	yes	yes	yes
Corps of engineers 2	No	yes	yes	yes
Janbu generalized method	yes	yes	yes	yes
Sarma method	yes	yes	yes	yes

Source: Krahn, 2004

3.3.2 Finite element method

Application of the numerical methods to solve the geotechnical problems has expanded due to availability of the modern, high capacity computers. These numerical methods rely on the solution of the partial differential equations which describe the behavior of the soil for the particular boundary conditions considered. However in here exact solution cannot be achieved and only approximate solution can be found.

The finite element analysis of geotechnical problems relies on the discretization of a continuum into a number of elements which are connected at nodal points (Russell, 1992). On the other hand finite element analysis is a numerical technique for finding approximate solutions of partial differential equations as well as of integral equations. The solution approach is based either on eliminating the differential equation completely or rendering partial differential equations into an approximate system of ordinary differential equations (Aziz, 2010). Degree of approximation is a function of the complexity of the analysis and the material model, the amount of computational effort used and the experience of the analyst (Russell, 1992).

Finite element method originated to solve the complex elastic and structural analysis problems in civil and aeronautical engineering. First concept for the Finite Element Method was introduced by Alexander Hrennikoff (1941) and Richard Courant (1942). Hrennikoff discretized the domain by using a lattice analogy and Courant divided the domain into triangular sub regions. However the term Finite Element was first used by Clough in 1960 in the context of plane stress analysis and has been in common usage since that time. During the 1960s, the finite element software code NASTRAN was developed in conjunction with the space exploration program of the United States and it was the first major finite element software code (Hutton, 2004; Aziz, 2010).

3.3.2.1 PLAXIS 2D

PLAXIS is a special purpose two-dimensional finite element computer program used to perform deformation and stability analyses for various types of geo technical applications. It was introduced by technical university of Delft in 1987 as an initiative of the Dutch department of public works and water management. Real situations can be modeled either by using plain strain or axisymmetric model. PLAXIS uses a convenient graphical user interface that enables users to quickly generate a geometry model finite element mesh (Aziz, 2010; "Reference manual", 2011). Brinkgreve and Broere (as cited in Aziz, 2010) mentioned that PLAXIS is a finite element program for geo technical applications in which soil modes are used to simulate the soil behavior.

Plaxis is mainly a two-dimensional program for statically computing but there are also additional versions of the program which can calculate dynamical models.

Generally advanced constitute models are required to simulate the nonlinear, time dependent and anisotropic behavior of soil and rock. And also special procedures are required to deal with hydrostatic pore – pressure in the soil.

3.3.2.1.1 Material models

Material models are used to represent the soil behavior qualitatively, and model parameters are used to quantify the soil behavior. Material model is a set of mathematical equations that describes the relationship between stress and strain. They are often expressed in a form in which infinitesimal increment of stress are related to infinitesimal increment of strain (“Material model manual v8”, 2011).

1. Mohr-Coulomb model

Mohr-Coulomb model is a simple, robust, nonlinear model and it represents a first order approximation of soil or rock behavior. In PLAXIS Mohr-Coulomb model uses an elastic perfectly plastic constitutive model for three dimensional state of stress. Stiffness behavior before reaching the local shear strength is poorly modeled in the Mohr-Coulomb in PLAXIS, where it assumes the stiffness behavior to be linear elastic below the failure surface. However strength behavior is modeled better in Mohr-Coulomb model (Ehsan, 2013; “Material model manual v8”, 2011).

Ehsan (2013) mentioned that the use of effective strength parameters in un-drained analysis of Mohr-Coulomb model may result in an over estimation of the shear strength of the material in un-drained conditions. Pickles (as cited in Ehsan, 2013) found the difference between effective stress paths for Mohr-Coulomb model and real soil as shown in Figure 3.7.

Mohr-Coulomb model basically requires five parameters,

- Young’s modulus
- Poisson’s ratio
- Friction angle
- Cohesion
- Dilatancy angle

And also Mohr-Coulomb model consists with some advanced parameters. These advanced features comprise of the increase of stiffness and cohesive strength with depth and the use of the tension cut-off. Tension cut-off can be used for the situations where soil has failed due to tension instead of the shear.

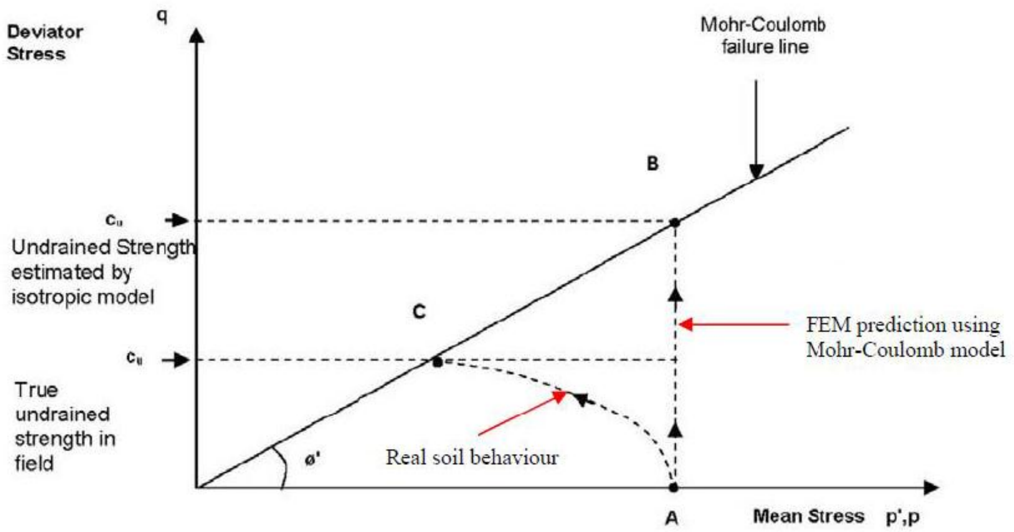


Figure 3.7 : Effective stress paths followed in real soil and MC model

Source: Ehsan ,2013

2. Soft Soil model (SS)

Soft soil model is a cam clay type model and use to model the consolidation behavior of soft soil. Use of soft soil model should be limited to the situations that are dominated primarily by compression.

Features of the Soft Soil model,

- Stress dependent stiffness.
- Distinction between primary loading and unloading – reloading.
- Memory for pre-consolidation stress.
- Failure behavior according to the Mohr-Coulomb criteria.

Soft Soil model consists with the five basic parameters such as modified compression index, modified swelling index, cohesion, friction angle and Dilatancy angle. Instead of that it consists with three advanced parameters such as Poisson’s ratio for unloading/reloading, coefficient of lateral stress in normal consolidation and K_0^{NC} parameter.

3. Soft Soil creep model (SSC)

Soft Soil creep model can be used to model the time dependent and logarithmic compression behavior of normally consolidated soft soil. It's an extension of the original soft soil model. Soft soil creep model has been developed to evaluate the settlement problems of foundations, embankments etc. for unloading problems, as normally encountered in tunneling and other excavation problems, the soft soil creep model hardly replace the simple Mohr-Coulomb model. Neher et al. (as cited in Gunduz, 2010) mentioned that the low value of OCR does better match the Creep model then the Soft Soil model.

For over consolidated soils SSC model has no advantages compared to SS model. However for normally consolidated soft clays creep becomes significant and SSC model exceeds the SS model (Neher, Wehnert & Bonnier, 2001)

Basic features of the soft soil creep model (Gustafsson & Tian, 2011)

- Stress-dependent stiffness (logarithmic compression behavior)
- Distinction between primary loading and unloading-reloading
- Time-dependent compression
- Memory of pre-consolidation pressure
- Soil strength following the Mohr-Coulomb failure criteria
- Yield surface adapt from the Modified Cam Clay model
- Associated flow rule for plastic strains

In here also strength parameters are similar to the Mohr-Coulomb model. In addition to that it consists with three stiffness parameters and three advanced parameters.

Stiffness parameters,

- Modified swelling index (K^*)
- Modified compression index (λ^*)

- Modified creep index (μ^*)

Advanced parameters,

- Poisson's ratio for unloading/reloading (v_{ur})
- $\sigma_{xx}' / \sigma_{yy}'$ Stress ratio in a state of normal consolidation (K_0^{NC})
- K_0^{NC} -Related parameter (M)

Gunduz (2010) compared the soft soil model and the soft soil creep model with field data as shown in Figure 3.8 and Figure 3.9. In here two test embankments were constructed in Ska- Edeby area, one with un-drained condition and one with drained condition. He concluded that soft soil creep model matches much better the field data than soft soil model and the differences between those two models become less for deeper layers.

Neher et al.(2001) found the relationship between the soft soil model, soft soil creep model and field data as shown in Figure 2.10.

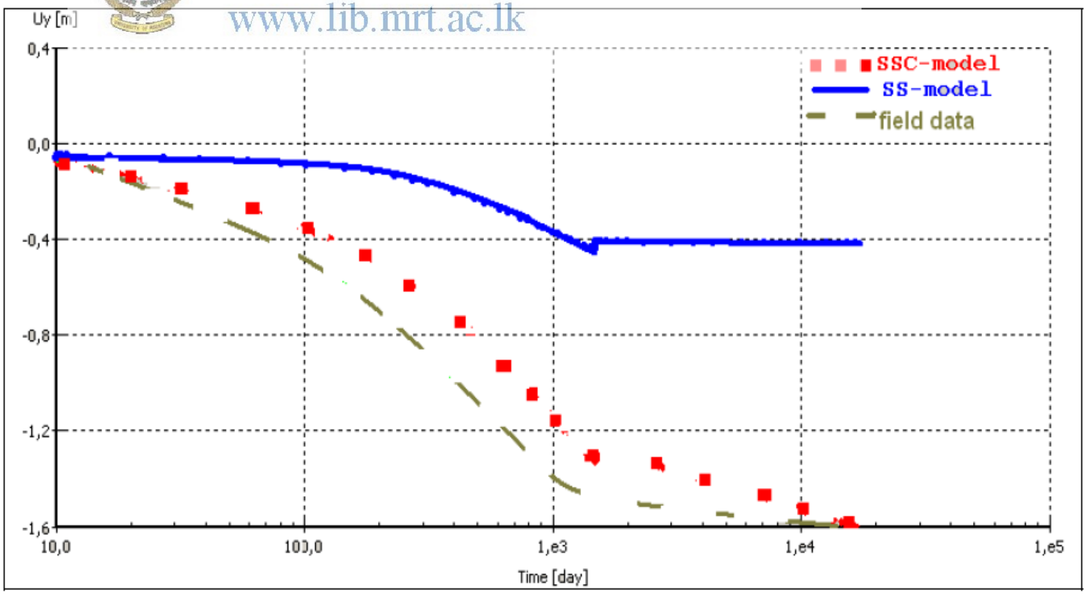


Figure 3.8 : comparison between soft soil model and soft soil creep model below the drained embankment

Source: Gunduz, 2010

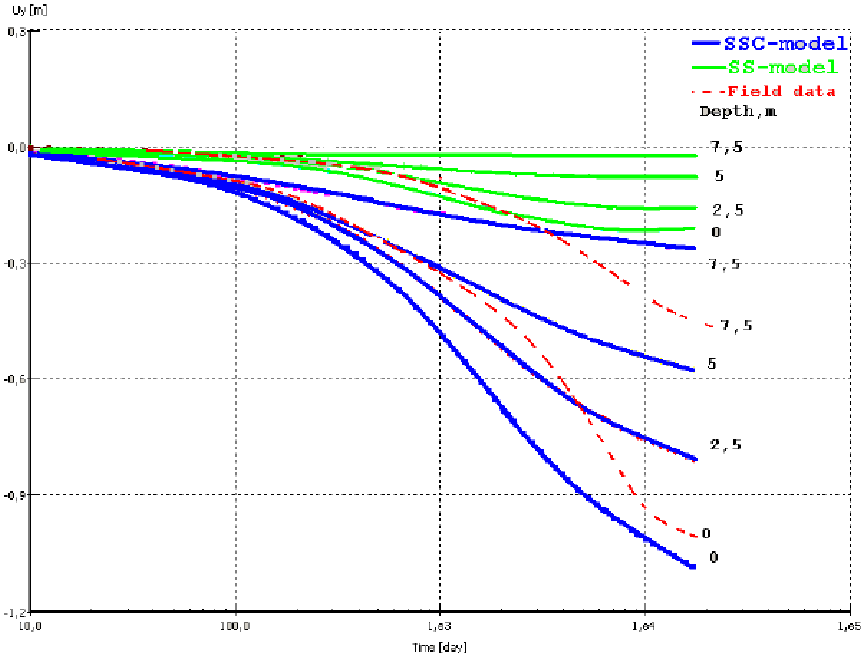


Figure 3.9: comparison between soft soil model and soft soil creep model below the un-drained embankment

Source: Gunduz, 2010

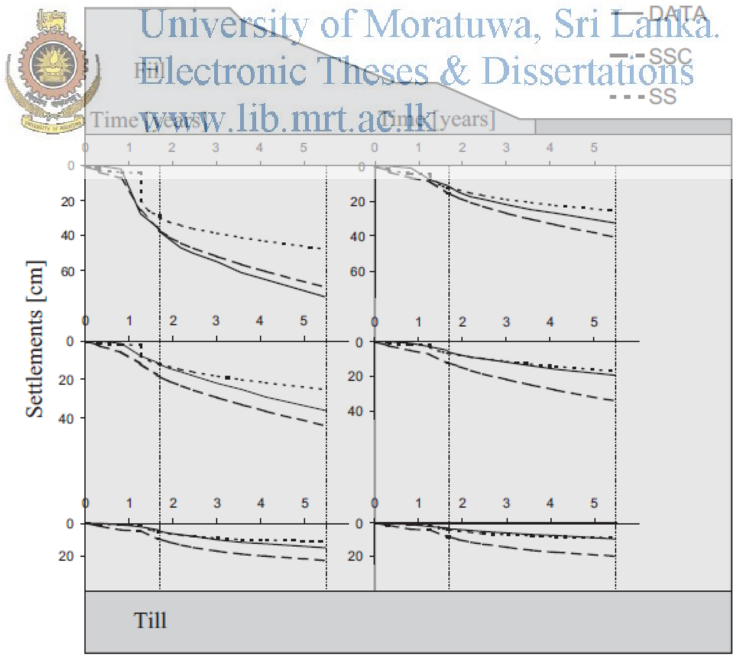


Figure 3.10: comparison between soft soil model and soft soil creep model at different depths under the embankment.

Source: Neher et al., 2001

3.3.2.1.2 Phi – c reduction

Phi-c reduction is an option available in PLAXIS to compute the Factor of Safety values. Here strength parameters (tan ϕ and c) of the soil are successively reduced until failure of the structure occurs. In Phi – c reduction method, total multiplier (ΣMsf) is used to express the value of the soil strength parameters at a given stage in the analysis.

$$\Sigma Msf = \frac{tan\varphi_{input}}{tan\varphi_{reduced}} = \frac{c_{input}}{c_{reduced}}$$

Input parameters refer to the properties entered in material sets and reduced parameters refer to the reduced values used in the analysis. At the beginning, to set all material strength to their unreduced values ΣMsf should be set to one. The strength parameters are successively reduced automatically until failure of the structure occurs. At this point FOS is given by;

$$FOS = \frac{available\ strength}{strength\ at\ failure} = values\ of\ \Sigma Msf\ at\ failure$$

In the Factor of safety calculation, the total displacements do not have a physical meaning, but the incremental displacements in the final step give an indication of the likely failure mechanism. Hadjigeorgiou, Kyriakou & Papanastasiou (2006) found that the factor of value does not depend on the modulus of elasticity and Poisson's ratio. However variation of modulus of elasticity and Poisson's ratio largely affect for the computed deformations.

3.3.2.1.3 Updated mesh analysis

Updated mesh analysis includes second order deformations on the other hand changes of the geometry are also taken into account. Updated mess analysis is necessary to analysis of reinforced soil structures, large offshore footing collapse problems and the study of problems where soils are soft and large deformations occur. In updating mesh analysis stiffness matrix is always updated at the beginning of the load step.

Updated mesh procedures used in PLAXIS involve considerably more than simply updating nodal coordinates as the calculation proceeds. These calculation procedures are in fact based on an approach known as an Updated Lagrangian formulation. Four basic types of calculations (Plastic, Plastic drained, Consolidation and safety) can optionally be performed as an Updated mesh analysis, taking into account the effects of large deformations. Water pressures also can be recalculated according to the updated position of the stress points. This option is termed as the updated water pressure it includes the effect of the soil settling below a constant phreatic level ("PAXIS 2D reference manual", 2011).

3.3.2.2 Comparison of numerical and limit equilibrium analysis methods

Lorig & Varona (as cited in Hadjigeorgiou et al., 2006) compared the limit equilibrium method and numerical method as shown in Table 3.2.

Table 3.2 : Comparison of numerical and limit equilibrium analysis methods

Analysis result	**Numerical Solution**	**Limit Equilibrium**
Equilibrium	Satisfied everywhere	Satisfied only for specific objects, such as slices
Stresses	Computed everywhere using field equations	Computed approximately on certain surfaces
Deformations	Part of the solution	Not considered
Failure	Yield condition satisfied everywhere; slide surfaces develop "automatically" as conditions dictate	Failure allowed only on certain pre-defined surfaces; no check on yield condition elsewhere
Kinematics	The "mechanisms" that develop satisfy kinematic constraints	A single kinematic condition is specified according to the particular geologic conditions

Source: Hadjigeorgiou et al., 2006

3.3.3 Slope stability analysis using field monitoring data

Use of field monitoring data to evaluate the stability of the embankment is easy and more practicable method than use of analytical methods. On the other hand embankment over soft ground is usually constructed by the smaller safety factor compared to the other structures. So evaluate the stability of such embankment can be done quickly and easily by using practically possible measurements.

Generally during the loading of the soft ground both of the consolidation settlement and the shear deformation are occurred. Failure will occur when the progress of the shear deformation is faster than that of consolidation settlement. It is important to find out the relationship between the displacement and the failure, for the prediction of the behavior of the embankment with in short time period (chin, 2005).

Matsuo & Kawamura (1977) proposed a diagram after observing the deformation of many embankments and plotted the progress of displacement during construction of each embankment. It is an approximation method. However it will give much contribution to the present engineering problem of the soft ground due to following reasons,

1. Deformation of a ground is closely related to the fall and failure of function of an embankment constructed on soft ground.
2. It is very useful for the observational method in earth work if the prediction of the failure is possible from observed deformations.

The basic parameters used in the Matsuo chart are vertical settlement at the center of the embankment (d) and the lateral displacement at the toe of the embankment (δ) as shown in Figure 3.11. Vertical settlement represents the consolidation settlement and the lateral displacement is used to represent the shear deformation.

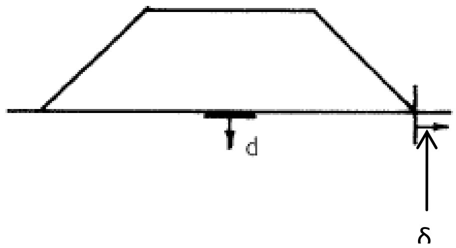


Figure 3.11: Basic parameters used in the Matsuo chart

Matsuo & Kawamura (1975) reported that although the section and unit weight of each embankment, soil properties and the thickness of each soft layer and other surroundings are different from each other, but many embankments under such different conditions failed near the one curve which can be regarded as the “Failure criterion line” as shown in Figure 3.12.

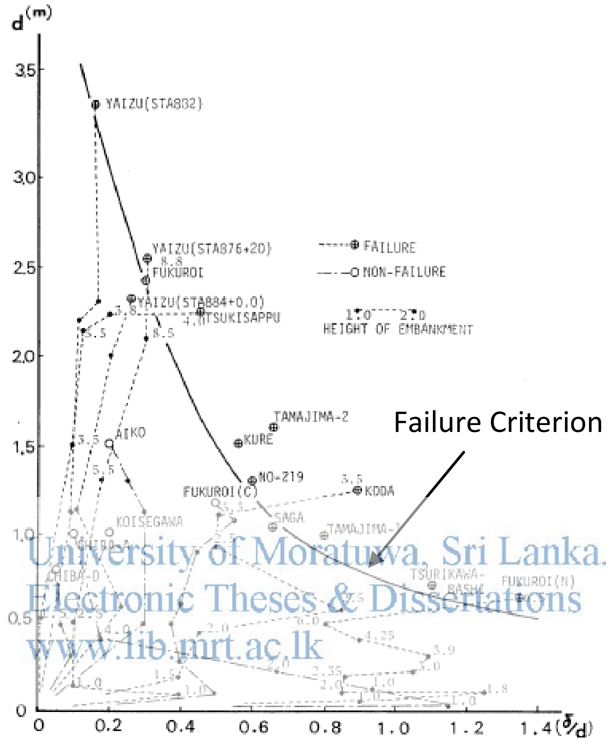
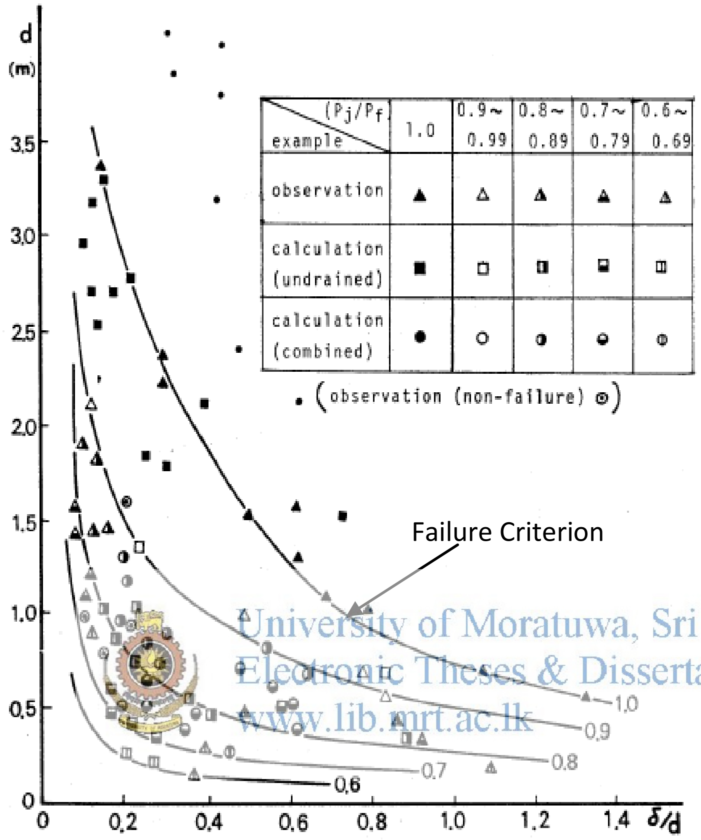


Figure 3.12: (δ /d – d) diagram for prediction of failure

Source: Matsuo et.al, 1977

Figure 3.12 is very useful for the engineering problem, but it is not sufficient to practically control the construction. According to the Figure 3.12 failure can be realized after it has occurred. On the other hand this diagram doesn’t give any idea about the sign of failure. Therefore if there is a diagram to measure the degree of safety of an embankment under construction, the safety level of the embankment can be easily recognized and can take precautions to ensure the stability.

To full fill the above requirement Matsuo & Kawamura (1977) modified the previous diagram as shown in Figure 3.13. In the modified diagram the distance between each contour line becomes larger with increase of (Applied load /Load at failure). This means that the rate of deformation is accelerated as an embankment approaches to the failure. And also stability of the ground tends to increase, when the displacement curve move towards the smaller contour lines.



Pj - Applied Load

Pf - Load at failure

d – Settlement at the midpoint of the embankment

δ – displacement at the midpoint of the embankment

Figure 3.13: modified (δ /d – d) diagram

Source: Matsuo et.al, 1977

CHAPTER 4 METHODOLOGY

This chapter includes the methodology implemented to achieve the objectives of the study. General procedure of the research can be summarized in to a chart as shown in Figure 4.1.

Literature review

↓

Numerical modeling of test embankments published in the literature

↓

Comparison of results with the field observation data to calibrate the model

↓

Numerical modeling actual embankments which are belong to Colombo – Katunayaka highway using FEM and analyze the stability of embankment slopes using LEM.

↓

Comparison of results with the results obtained from the Matsuo chart

↓

Investigate the effect of embankment height, width and sub soil parameters on the prediction of stability by using FEM, LEM and Matsuo chart

Figure 4.1 : Flow chart of methodology

4.1 Analysis of the behavior of embankments published in the literature

In order to verify the accuracy of numerical modeling, Deformation characteristics and Factor of safety values of the following embankments were analyzed by using PLAXIS 8.2 and the predicted results have been compared with the field observation data.

- Aiko embankment (Shoji & Matsumoto ,1976)
- Muar embankment (Indraratna et.al, 1992)

4.1.1 Aiko embankment

Shoji & Matsumoto (1976) constructed the Aiko test embankment in the Aiko district in the middle of the northern Kanagawa Prefecture. Sub surface profile of the embankment is shown in the Figure 4.2.

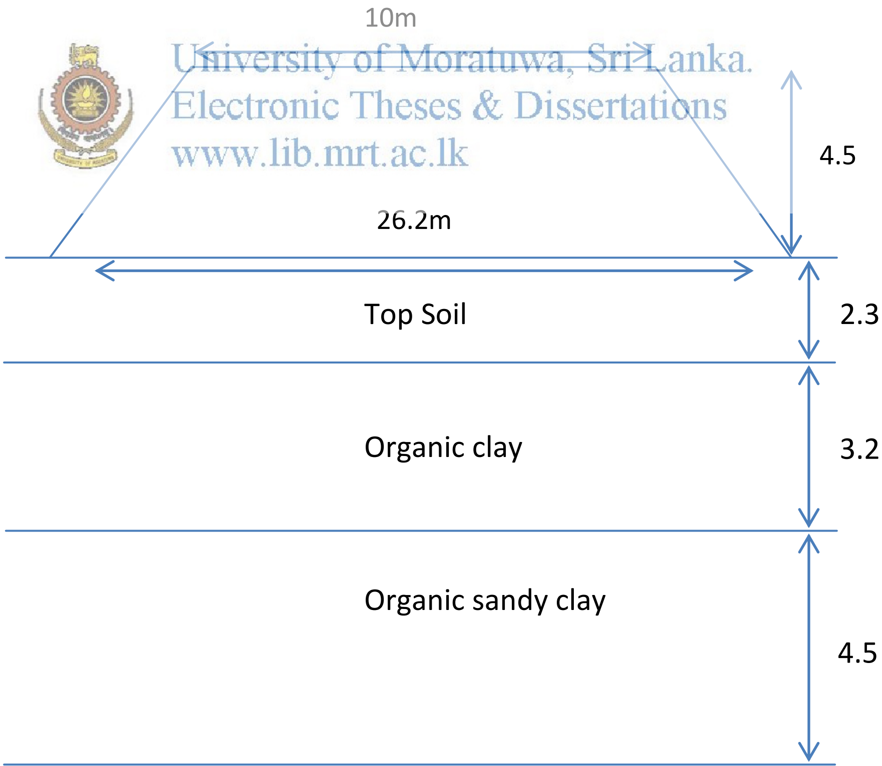


Figure 4.2: Sub surface profile of the Aiko test embankment

Source: Shoji et al., 1976

Embankment was filled as one step within 18 days followed by 100 days consolidation period. Then embankment was analyzed by using Mohr Coulomb Model, soft soil model and soft soil creep model. To model the behavior of soft clay plain strain model and 15 nodes elements were used and standard fixity option was used to define the boundary conditions. Finally Factor of safety values were obtained by using Phi – c reduction technique. Parameters used for the soft soil model and soft soil creep model are shown in Table 4.1 and Table 4.2.

Most of the soil parameters were given by the author (Shoji & Matsumoto, 1976). Approximate values were assumed for unknown parameters.

4.1.2 Muar test embankment

Indraratna et.al (1992) built a full scale test embankment (Muar embankment) on soft Malaysian marine clay to evaluate the performance of the embankment. Muar test embankment was reanalyzed by using finite element method and limit equilibrium method in order to compare the predicted values with field observation data. Subsurface profile of the embankment and the geometry of the embankment are shown in the Figure 4.3.

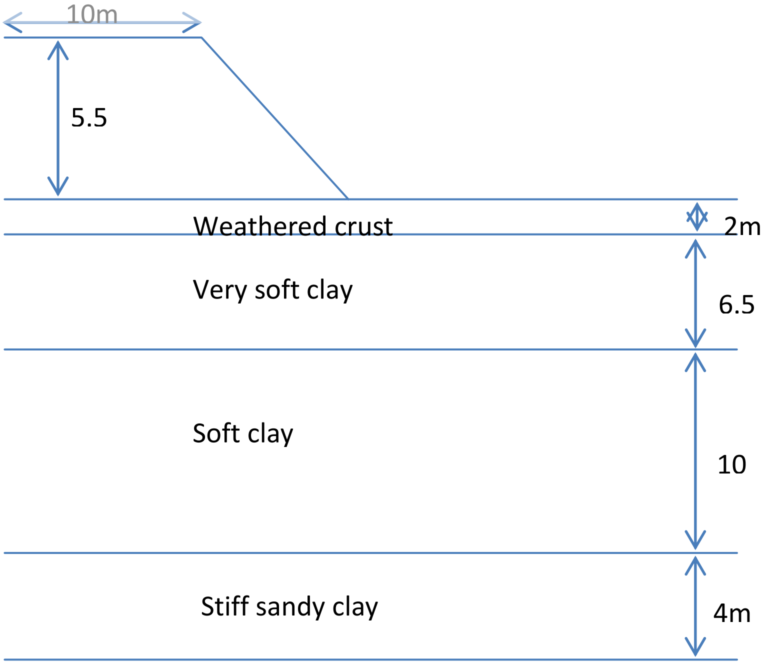


Figure 4.3 : Sub surface profile of the Muar test embankment

Table 4.1: soil properties for soft soil model (Aiko embankment)

Layer	Soil type	model	condition	γ_{unsat} (kN/m^3)	γ_{sat} (kN/m^3)	Φ	C_u (kN/m^2)	Cc	Cs	e_{int}	kx (m/day)	ky (m/day)	E (kN/m^2)	v
I	Top soil	SS	undrained	11	12	0.1	20	0.104	0.010	0.5	$1.8*10^{-3}$	$1.8*10^{-3}$	600	0.333
II	Organic Clay	SS	undrained	11	11	0.1	15	0.421	0.042	0.5	1.5	0.4	200	0.333
III	Organic Sandy clay	SS	undrained	11	13.5	0.1	25	0.104	0.010	0.5	0.02	0.02	850	0.333
	fill	M/C	drained	16	20	30	5	-	-	-	1	1	3000	0.3

Table 4.2 : soil properties for soft soil creep model (Aiko embankment)

Layer	Soil type	model	condition	γ_{unsat} (kN/m^3)	γ_{sat} (kN/m^3)	Φ	C_u (kN/m^2)	Cα	Cc	Cs	e_{int}	kx (m/day)	ky (m/day)	E (kN/m^2)	v
I	Top soil	SSC	undrained	11	12	0.1	20	0.010	0.104	0.0104	0.5	$1.8*10^{-3}$	$1.8*10^{-3}$	600	0.333
II	Organic Clay	SSC	undrained	11	11	0.1	15	0.008	0.421	0.0421	0.5	1.5	0.4	200	0.333
III	Organic Sandy clay	SSC	undrained	11	13.5	0.1	25	0.010	0.104	0.0104	0.5	0.02	0.02	850	0.333
	fill	M/C	drained	16	20	30	5	-	-	-	-	1	1	3000	0.3

4.1.2.1 Finite element analysis

Finite element analysis was conducted by using PAXIS 8.2. Filling was done in three layers and filing sequence is shown in the Table 4.3. Plain strain model and 15 nodes elements were used to define the problem and standard fixity option was used to define the boundary conditions. Embankment was analyzed by using Soft soil creep model and updated mesh analysis. Parameters used for the Soft soil creep model are shown in the Table 4.4. Variation of the following parameters with time was plotted by using PAXIS 8.2.

- Vertical settlement at the center of the embankment
- Lateral deformation near the inclinometer no 3
- Pore water pressure distribution at the location p2

These predicted values were compared with the field values. Locations of the field monitoring instruments are shown in the Figure 2.2. Finally factor of safety values were obtained by using Phi – c reduction technique.

Table 4.3 : filling sequence of the Muar test embankment

Layer no	Fill height (m)	Time period (days)
I	2	35
II	2	35
III	1.5	26

4.1.2.2 Limit equilibrium analysis

Geometry of the embankment is similar to the geometries used in the PLAXIS modeling. Embankment was analyzed by using SLOPE/W software and adopted method for the analysis is Morgentsern and Price method. Finally factor of safety value was determined by using grid and radius method.

Table 4.4 : Soil properties for Muar test embankment

Layer	Soil type	Model	γ_{unsat} (kN/m^3)	γ_{sat} (kN/m^3)	Φ	Cu (kN/m^2)	Kx (m/day)	ky (m/day)	E (kN/m^2)	v	e_0	Cc	Cr	Cα
1	Weathered crust	SSC	16.5	17	0.1	15.4	1.296*10^{-4}	6.912*10^{-5}	-	0.3	0.5	0.75	0.075	0.05
2	Very soft clay	SSC	15.5	16	0.1	13.4	1.296*10^{-4}	6.912*10^{-5}	-	0.4	0.5	0.75	0.075	0.05
3	Soft clay	SSC	15.5	16	0.1	19.5	0.095*10^{-3}	5.184*10^{-5}	-	0.4	0.5	0.45	0.045	0.03
4	Stiff sandy clay	SSC	16	17	0.1	25.9	0.095*10^{-3}	5.184*10^{-5}	-	0.3	0.5	0.15	0.015	0.006
fill	Lateritic soil	MC	18	20	26	19	1	1	5100	0.3	-	-	-	-

4.2 Analysis of actual embankments

Three embankments (K7+870, K6+530 and K6 +850), which are belong to the Colombo- Katunayaka expressway were analyzed by using finite element method, limit equilibrium method and Matsuo chart.

4.2.1 Finite element modeling

Three embankments were analyzed by using PLAXIS 8.2. 2D plain strain model and 15 nodes elements were used to define the problem and standard fixity option was used to define the boundary conditions. Geometries of the embankments are shown in Figure 4.4 – 4.6. Soil parameters used for the PLAXIS model are shown in Table 4.8 – 4.10.

At the end of the analysis following parameters were obtained,

- Settlement at the center of the embankment
- Lateral displacement at the toe of the embankment
- Factor of safety value at the final Phase

Values obtained for the settlement at the center of the embankment and the lateral displacement at the toe of the embankment were used to read the Matsuo chart.

4.2.2 Limit equilibrium modeling

SLOPE/W 2004 was used to conduct the limit equilibrium analysis. Adopted method for the analysis is Morgentsern and Price method. Factor of safety values were determined by using grid and radius method. Geometries of the embankments are similar to the geometries used in the PLAXIS modeling.

Table 4.5: filling sequence of the K7+870

Layer (Fill)	Height (m)	Construction period	Consolidation period
1	2	16	39
2	1.5	22	173
3	1.5	7	481

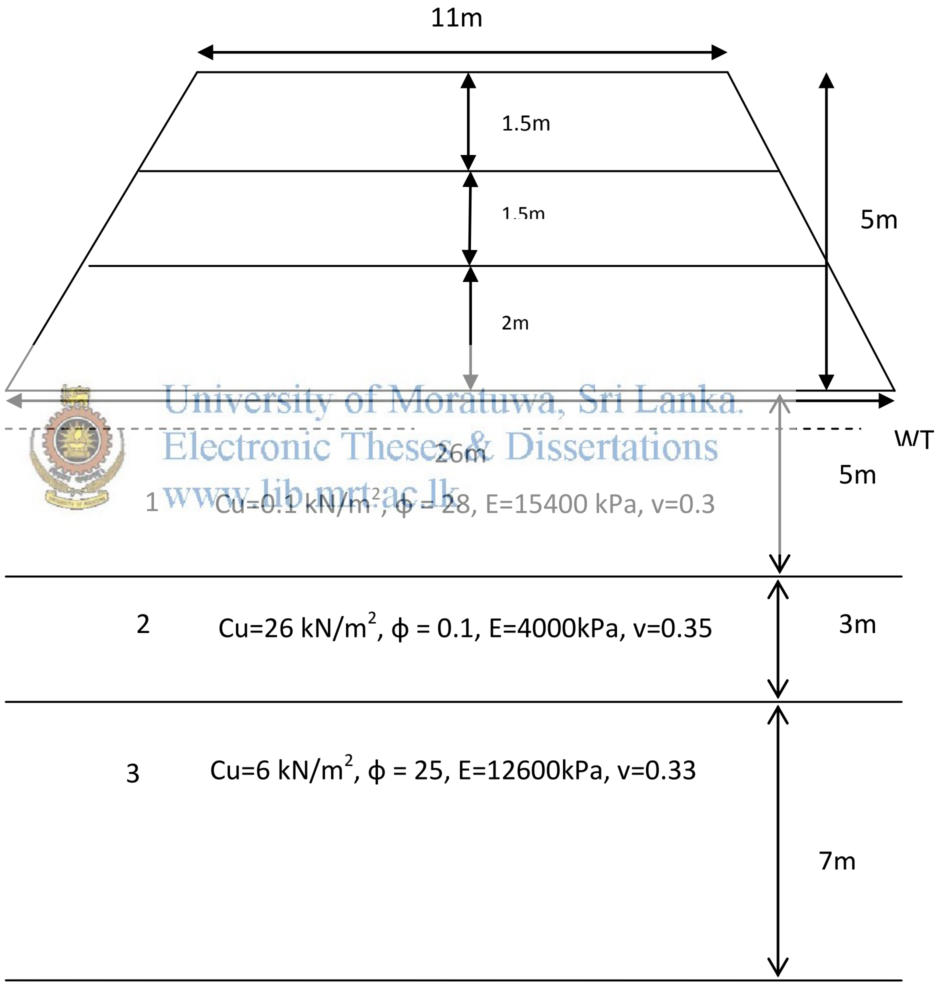


Figure 4.4 : Details of the K7+870 embankment

Table 4.6 : filling sequence of the K6 +850

Layer (Fill)	Height (m)	Construction period	Consolidation period
1	0.5	7	8
2	0.5	7	293
3	1	3	18
4	3	23	67

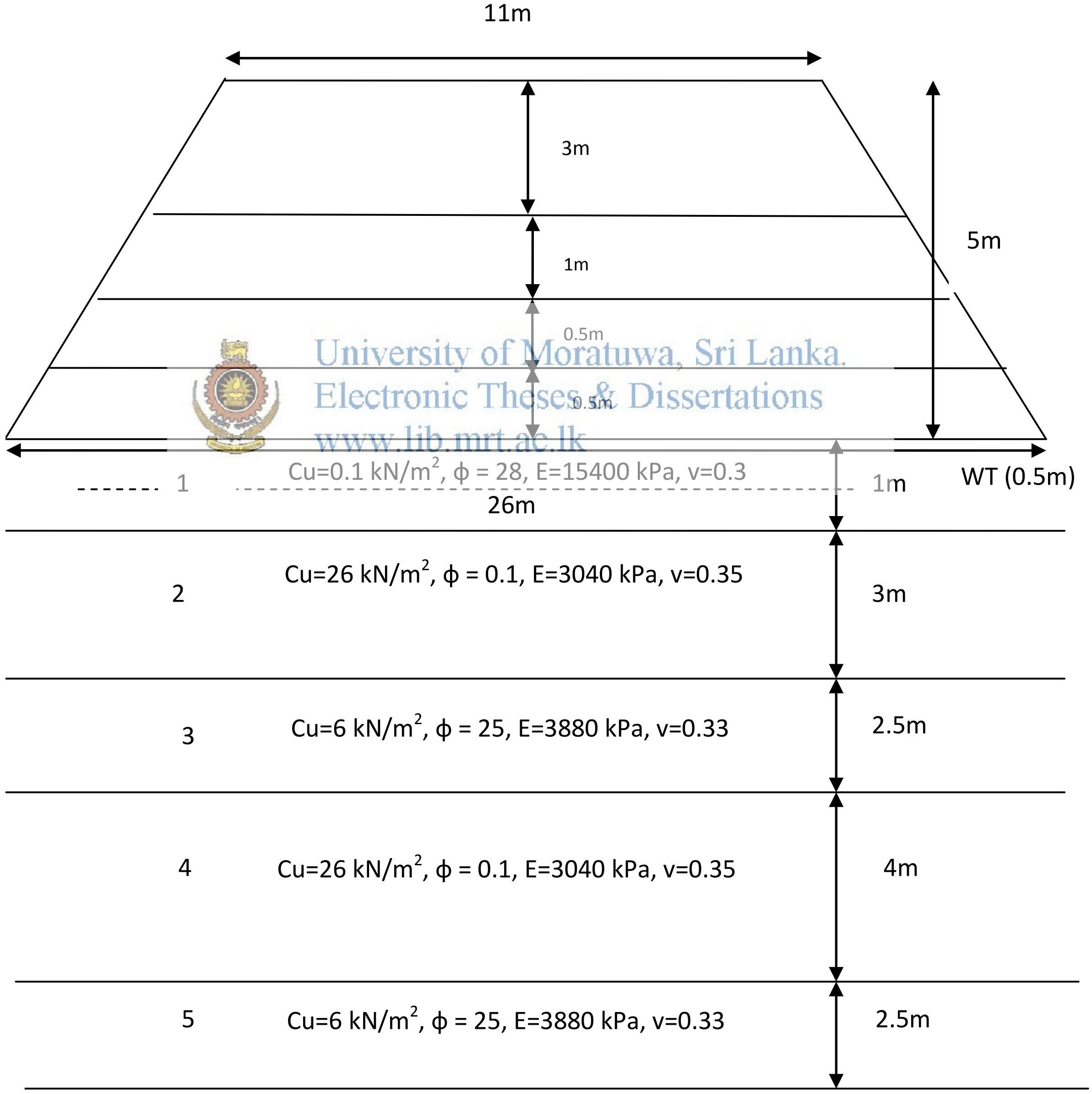


Figure 4.5 : Details of the K6+850 embankment

Table 4.7: filling sequence of the K6+530

Layer	Height (m)	Construction period	Consolidation period
1	1.5	8	30
2	0.5	4	8
3	2.5	10	35
4	1.1	2	65

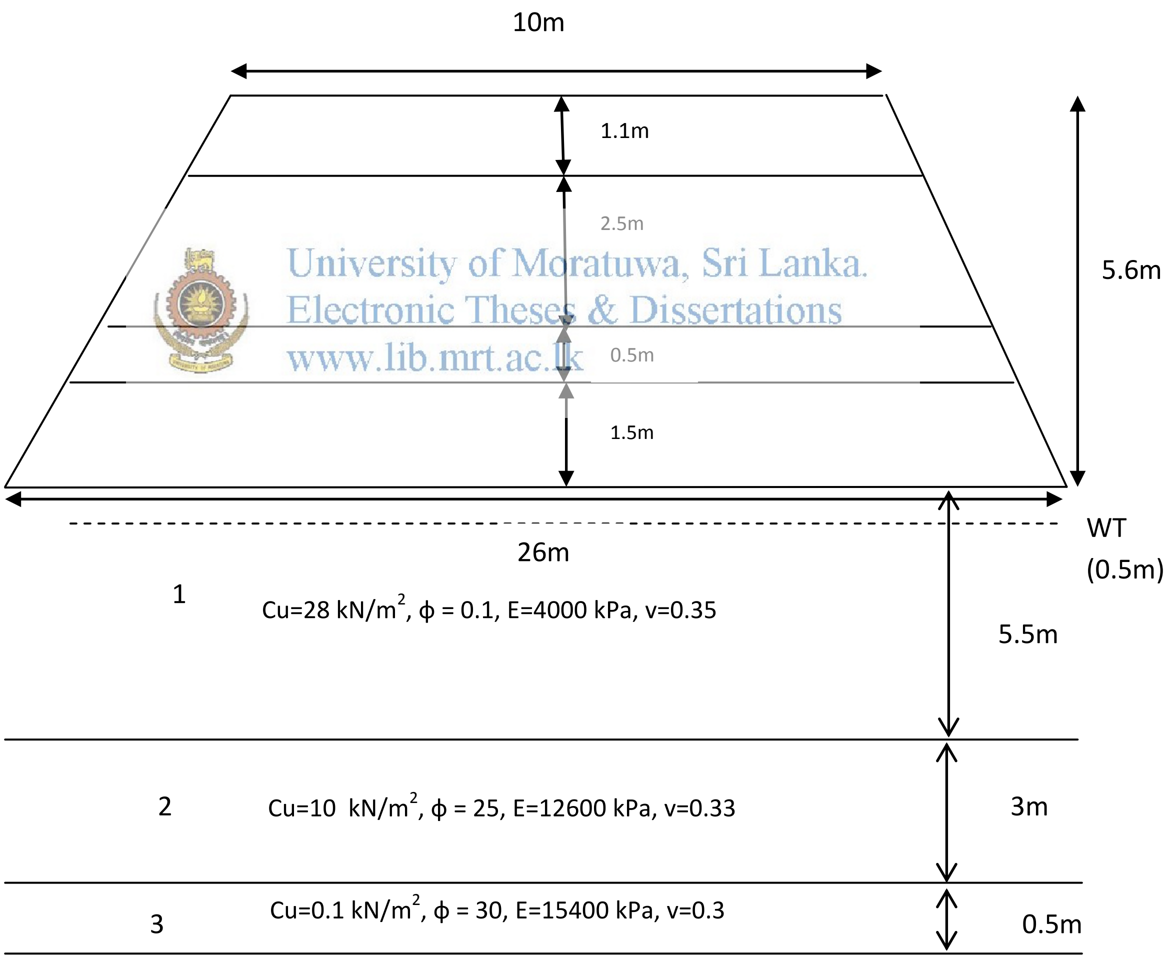


Figure 4.6 : Details of the K6+530 embankment

Table 4.8 : Soil properties of the K7+870

Layer	γ_{unsat} (kN/m^3)	γ_{sat} (kN/m^3)	Φ_u /Φ'	C_u / C' (kPa)	Ky (m/day)	Kx (m/day)	E (kPa)	V	C_C	C_S	e_{int}	Model	Condition
Layer 1	17	18	28	0.1	86	86	15400	0.3	-	-	-	MC	Drained
Layer2	16	16.5	0.1	26	$1.8*10^{-4}$	$9*10^{-5}$	4000	0.35	0.73	0.073	0.5	SS	Undrained
Layer3	16	17	25	6	$9*10^{-3}$	$9*10^{-3}$	12600	0.33	-	-	-	MC	Drained
Fill	16	20	30	5	1	1	3000	0.3	-	-	-	MC	Drained

Table 4.9 : Soil properties of the K6+530

Layer	γ_{unsat} (kN/m^3)	γ_{sat} (kN/m^3)	Φu / Φ'	C_u / C' (kPa)	Ky (m/day)	Kx (m/day)	E (kPa)	V	C_C	Cs	e	Model	Condition
Layer 1	16	16.5	0.1	28	$1.8*10^{-4}$	$9*10^{-5}$	4000	0.35	0.88	0.088	0.5	SS	Undrained
Layer2	16	17	25	10	$9*10^{-3}$	$9*10^{-3}$	12600	0.33	-	-	-	MC	Drained
Layer3	17	18	30	0.1	86	86	15400	0.3	-	-	-	MC	Drained

Table 4.10: Soil properties of the K6+850

Layer	γ_{unsat} (kN/m^3)	γ_{sat} (kN/m^3)	Φu / Φ'	C_u / C' (kPa)	Ky (m/day)	Kx (m/day)	E (kPa)	V	Cc	Cs	e_{int}	Model	Condition
Layer 1	17	18	28	0.1	86	86	15400	0.3	-	-	-	MC	Drained
Layer2	16	16.5	0.1	26	$1.8*10^{-4}$	$9*10^{-5}$	3040	0.35	0.88	0.088	0.5	SS	Undrained
Layer3	15	16	25	6	0.86	0.86	3880	0.33				MC	Drained
Layer 4	16	16.5	0.1	26	$1.8*10^{-4}$	$9*10^{-5}$	3040	0.35	0.88	0.088	0.5	SS	Undrained
Layer 5	15	16	25	6	0.86	0.86	3880	0.33				MC	Drained

4.2.3 Analysis of the embankments using Matsuo Chart

The factor of safety values were determined by using the Matsuo Chart. Factor of safety values are based on two sets of vertical and lateral deformation values,

1. Based on field monitoring data
2. Based on vertical settlement and lateral displacement obtained from PLAXIS.

4.3 Effect of Embankment Height, Width and Sub Soil Parameters on the Stability of the embankment

A hypothetical embankment was selected as illustrated in Figure 4.7 and effect of the embankment height (H), embankment width (B) and the sub soil parameters (Cu) were analyzed. Embankment consisted with a 10m thick peaty clay layer and water table located on the ground surface. Embankment height, embankment width and the sub soil parameters were varied according to the Table 4.11. Each embankment was modeled by using finite element method and the limit equilibrium method.

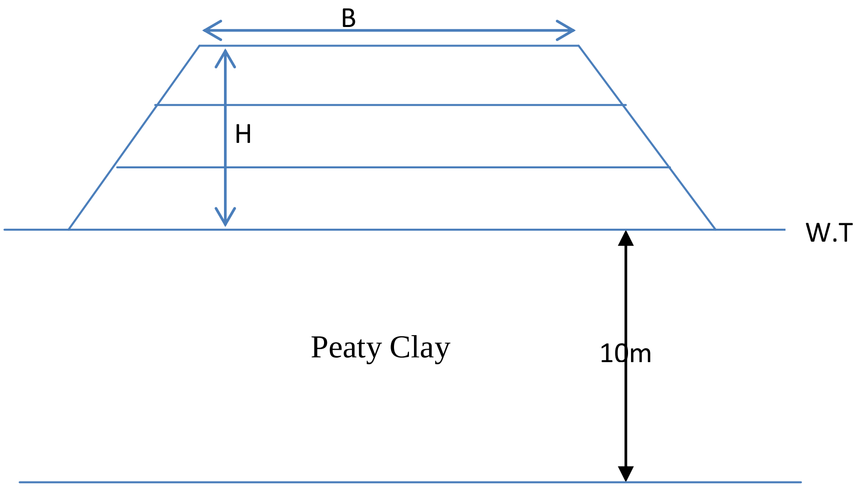


Figure 4.7 : Geometry of the hypothetical embankment

Table 4.11 : variation of the embankment height (H), width (B) and the un-drained shear strength

B (m)	H (m)	Cu (kN/m^2)
3	3	5,15,25
	4	5,15,25
	5	5,15,25
	6	5,15,25
5	3	5,15,25
	4	5,15,25
	5	5,15,25
	6	5,15,25
8	3	5,15,25
	4	5,15,25
	5	5,15,25
	6	5,15,25

4.3.1 Finite element modeling

Embankments were analyzed using 2D plain strain model and determined the factor of safety values by using Phi – c reduction technique. Filling consisted with three phases and rate of 0.5m/week was maintained. Each layer subjected to 200 days of consolidation period. Soil parameters of the hypothetical embankment are shown in Table 4.12.

In addition to the FOS values, vertical settlement at the center of the embankment and the lateral displacement at the toe of the embankment were also obtained from the finite element analysis.

4.3.2 Limit equilibrium modeling

Geometry of the embankment and the soil properties of the sub surface material are similar to the PLAXIS model. Factor of safety values were obtained by using Grid and radius method. Morgentsern and price method has been used to analyze the stability of the embankment.

4.3.3 Analysis of the embankments using Matsuo Chart

Factor of safety values were determined by using the vertical settlement and lateral displacement values obtained from the PLAXIS.

Table 4.12 : Soil parameters of the hypothetical embankment

	Peaty Clay	Fill
Model	SSC	MC
condition	Un – Drained	Drained
γ_{unsat} (kN/m^3)	11	18
γ_{sat} (kN/m^3)	11	20
Φ_u/ Φ’	0.1	30
C_u / C’(kN/m^2)	5,15,25	5
kx (m/day)	$2*10^{-3}$	1
ky (m/day)	$1*10^{-3}$	1
E (kN/m^2)	350	20000
v	0.35	0.3
e_0	1.5	-
Cc	0.75	-
Cr	0.075	-
Cα	0.05	-

CHAPTER 5 RESULTS AND DISCUSSION

5.1 Results of the Aiko test embankment

Results of lateral deformation at the toe of the embankment (δ) and the vertical settlement at the center of the embankment (d) for phase 1 (after 18 days) and phase 2 (after 100 days) are given in Table 5.1 and Table 5.2. Results obtained from the manual calculation are shown in Table 5.3 and manual calculation is attached to the Appendix A. Refer the Figure 4.2 for the details of the embankment. Table 4.1 and Table 4.2 give the parameters used for the analysis.

Table 5.1 : Lateral and vertical deformation values for the phase 1

M/C		SS		SSC	
d (m)	δ (m)	d (m)	δ (m)	d (m)	δ (m)
1.274	0.355	1.026	0.138	0.059	0.037
Updated mesh analysis					
0.929	0.198	0.929	0.083	0.028	0.02

Table 5.2 : Lateral and vertical deformation values for the phase 2

M/C		SS		SSC	
d (m)	δ (m)	d (m)	δ (m)	d (m)	δ (m)
1.723	0.413	2.616	1.308	1.026	0.622
Updated mesh analysis					
1.158	0.208	1.549	0.356	0.6	0.29

Table 5.3 : Calculated values for the Aiko embankment

	Vertical settlement /when all layers are in Normally consolidated state (NC)	Vertical settlement /when 3rd layer is in over consolidated state (OC)
Primary consolidation settlement	1.35m	1.175m
Secondary consolidation settlement	0.174m	0.173m
Total settlement	1.524m	1.348m

According to the results given in the Table 5.1 and Table 5.2 Soft soil creep model gives the very low vertical and lateral deformation values compared to the values obtained from the MC and SS methods. Calculated results are much similar to the SS model. Aiko embankment consists with approximately 10m thick subsoil layer and it will take at least one year to complete primary consolidation. During 100 days' time period most probably it couldn't complete primary consolidation and because of that SSC model gave these low values for the deformations.

Sub surface beneath the Aiko embankment mainly consist of organic clay soils and first two layers are in normally consolidated state and third layer is in over consolidated state. And also secondary consolidation is predominant with in the sub soil. Mohr coulomb method is insufficient to model the consolidation behavior and Soft soil model doesn't consider about the creep effect. So soft soil creep model is the best option. However soft soil creep model works well with the normally consolidated soil or soils with lower over consolidation ratio. Neher et.al (2001) mentioned that for over consolidated soils SSC model has no advantages compared to SS model.

On the other hand updated mesh analysis gives lower deformation values. Generally soft soils incorporate with larger deformations so we have to consider about the influence of the geometry changes of the mesh on the equilibrium condition. So values given by the updated mesh analysis are more reliable than the conventional finite element analysis. Because updates mesh analysis includes second order deformations and it consider about the changes of geometry.

Factor of safety values obtained from the Matsuo chart and the Phi – c reduction technique are given in the Table 5.4.

Table 5.4: Factor of safety values for Aiko embankment

FOS						
	Phase 1 (After 18 days)			Phase2 (After 100 days)		
Model	M/C	SS	SSC	M/C	SS	SSC
Phi – c reduction	1.259	1.259	1.256	1.258	1.264	1.254
Matsuo	1.11	1.31	1.7	1.08	1.26	1.05
Updated mesh						
Phi – c reduction	1.259	1.260	1.258	1.259	1.263	1.256
Matsuo	1.23	1.38	1.75	1.18	1.11	1.25

According to the field monitoring data (After 100 days)

Lateral deformation at the toe of the embankment (δ) – 0.2m

Vertical settlement at the center of the embankment (d) – 1.5m

FOS value (Matsuo chart) – 1.25

According to the results given in the Table 5.4, factor of safety values obtained from the Phi – c reduction technique are almost similar. In some cases factor of safety values obtained from the Phi – c reduction technique are similar to the Matsuo chart. They are shown as shaded boxes in Table 5.4. However it is difficult to find that kind of similarity in the other results.

There's a reasonable agreement between the factor of safety values which are given by the SS model and the SSC model. Because of that, both of the SS model and SSC model can be used to find out the factor of safety values for embankments which are constructed on soft clay. But when the creep condition is predominant we have to use the SSC model to obtain real and accurate results.

5.2 Results of the Muar Test Embankment

5.2.1 Finite element analysis

Lateral displacement near the inclinometer 3 was interpreted by using the finite element software PLAXIS as shown in Figure 5.1. Field data were reproduced by using original work done by Indraratne et al. (1992). Variation of the fill thickness with time is given in the Table 5.5. Details of the embankment and parameters used for the analysis are given in the Figure 4.3 and Table 4.4.

Table 5.5 : Variation of the fill thickness with time

Time (days)	Fill thickness (m)
35	2
70	2
96	1.5

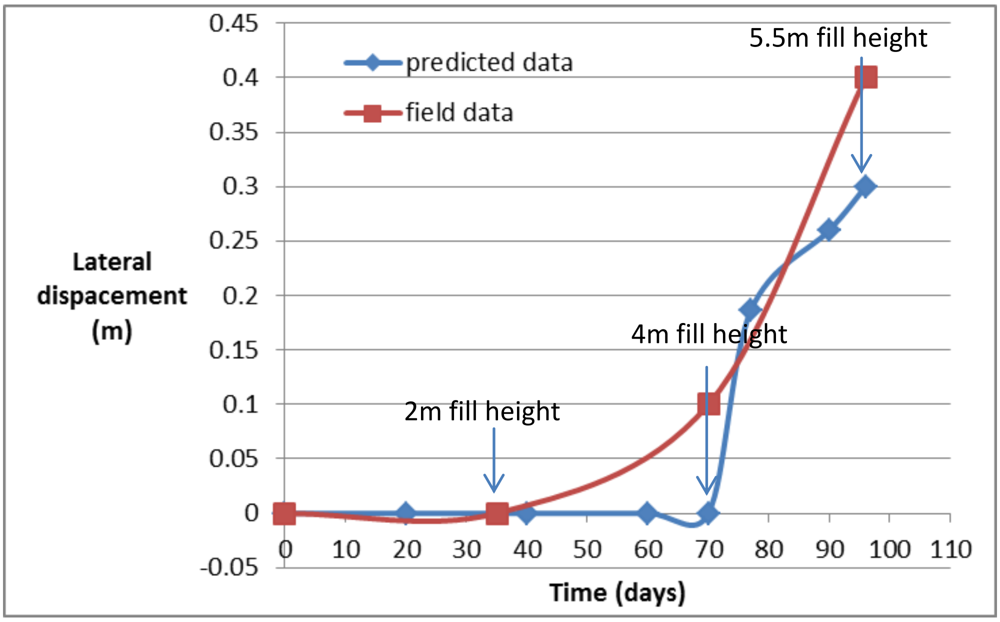


Figure 5.1 : Variation of lateral displacement with time - Muar embankment

Variation of the vertical settlement at the center of the embankment with time was plotted and shown in the Figure 5.2. Indraratne et al. (2005) found the variation of the surface settlement for the embankment height 5m as given in Figure 5.3.

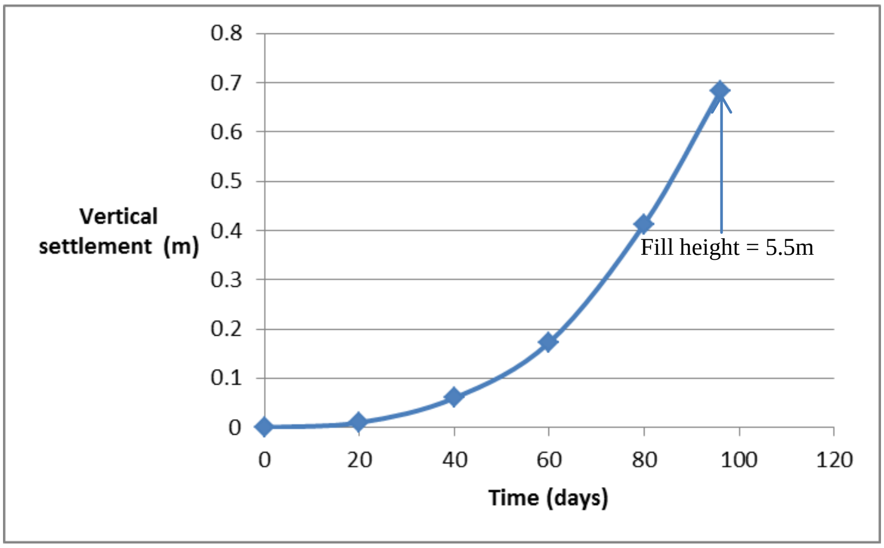


Figure 5.2: Variation of vertical settlement with time - Muar embankment

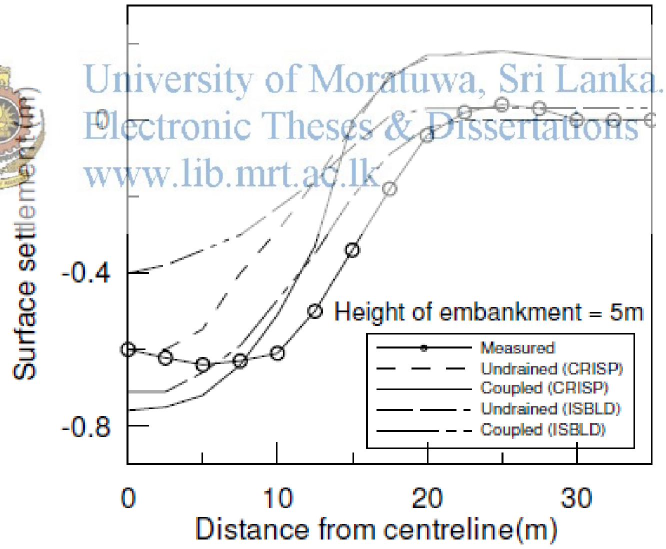


Figure 5.3: Surface settlement profiles for 5m fill height

Source: Indraratne et al., 2005

According to the Figure 5.2 predicted value of the vertical settlement at the center of the embankment is equal to 0.68m. Measured value for the vertical settement at the center of the embankment (Distance from center line = 0) is 0.6m as shown in Figure 5.3.

Figure 5.4 shows the variation of the excess pore water pressure with time and Field data which were reproduced by using original work done by Indraratne et al. (1992).

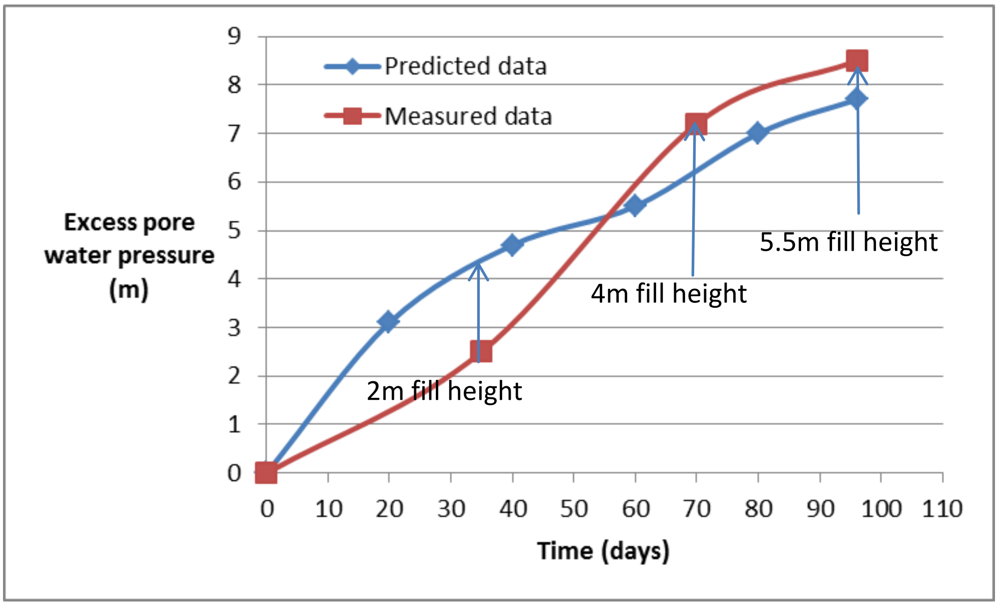


Figure 5.4 : variation of excess pore water pressure with time – Muar embankment

Above figures can be summarized in to the following table,

Table 5.6: Comparison between the predicted values and observed values - Muar test embankment

Parameter	Predicted value	Observed value
Lateral displacement for 5.5m fill height (m)	0.3	0.4
Vertical settlement at the center of the embankment (m)	0.68	0.6
Pore water pressure after 5.5m fill height (m)	7.7	8.5

According to the above table, predicted values for the lateral displacement, vertical settlement and excess pore water pressure are agreed well with the observed values. So, it can be stated that the finite element method can be used to predict the deformation characteristics of the embankment.

Predicted Failure surfaces obtained from the numerical analysis for phase 2 and phase3 are shown in the Figure 5.5 and Figure 5.6. Figure 5.7 shows the actual failure surface. Actual failure surface is almost similar to the predicted failure surface.

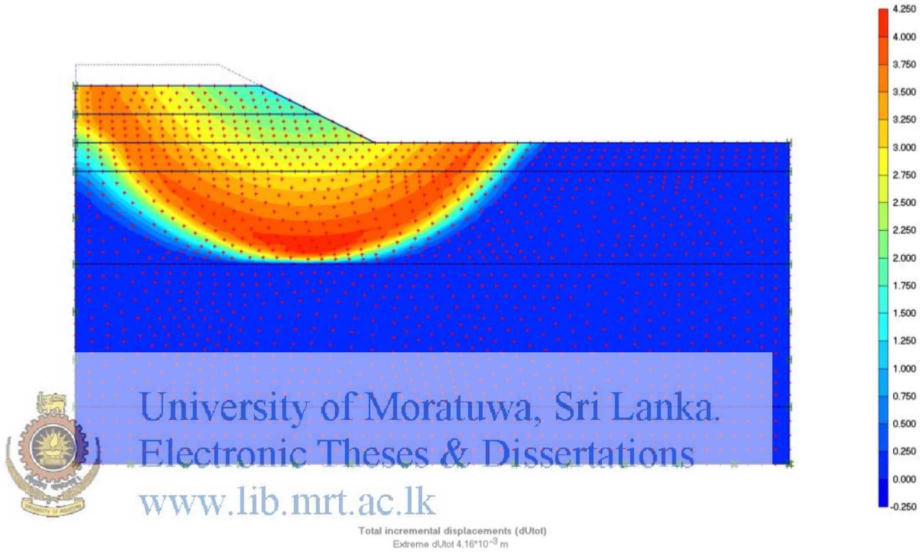


Figure 5.5: Predicted failure surface using FEM for Muar test embankment – Phase 2

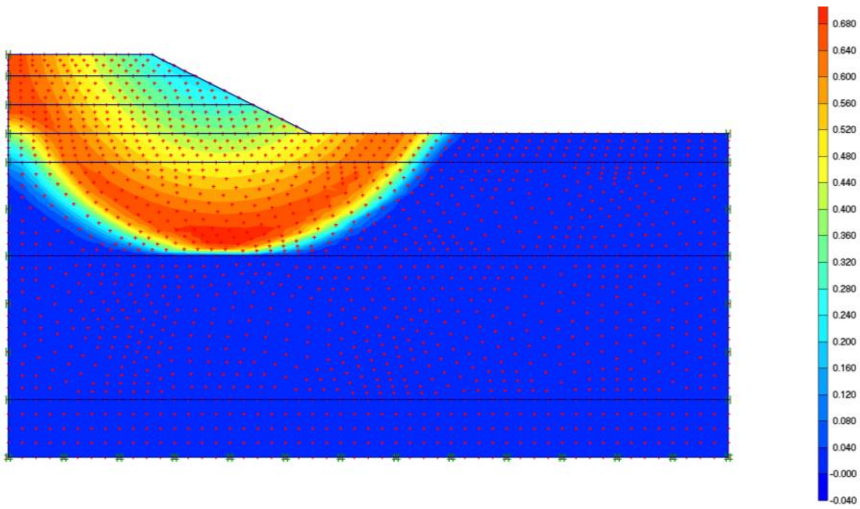


Figure 5.6: Predicted failure surface using FEM for Muar test embankment – Phase 3

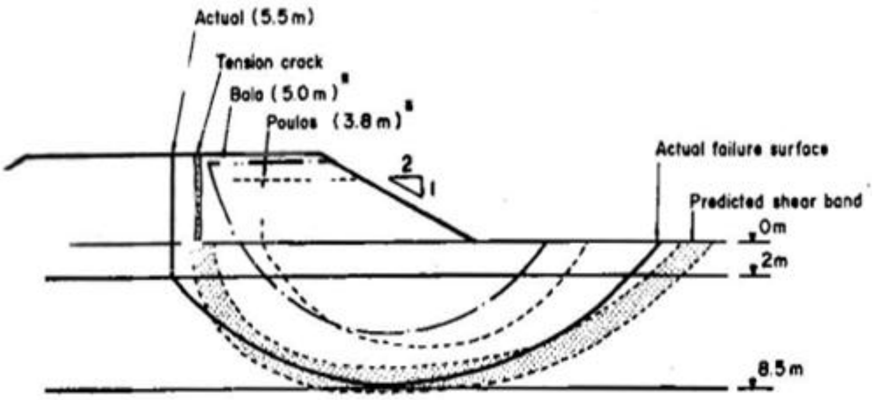


Figure 5.7: Actual failure surface of the Muar test embankment

Source: Indraratne et al. (1992)

Factor of safety values obtained from the phi-c reduction technique for phase 2 and phase 3 are equal to 1.098 and zero. So it can be said that actually failure has started during the 2nd phase of filing. It can be concluded that FEM can reasonably predict the stability of the embankment.

5.2.2 Limit equilibrium analysis

Figure 5.8 and Figure 5.9 show the predicted failure surfaces of the Muar test embankment obtained from the Limit equilibrium analysis.

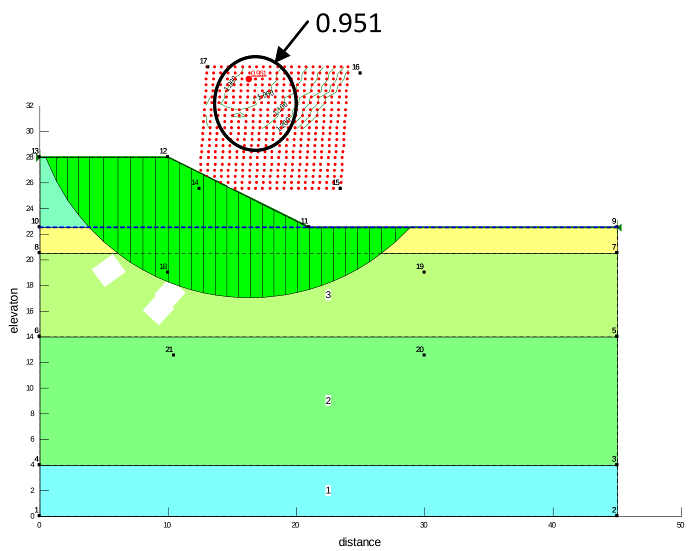


Figure 5.8 : Predicted failure surface using LEM for Muar test embankment – phase 3

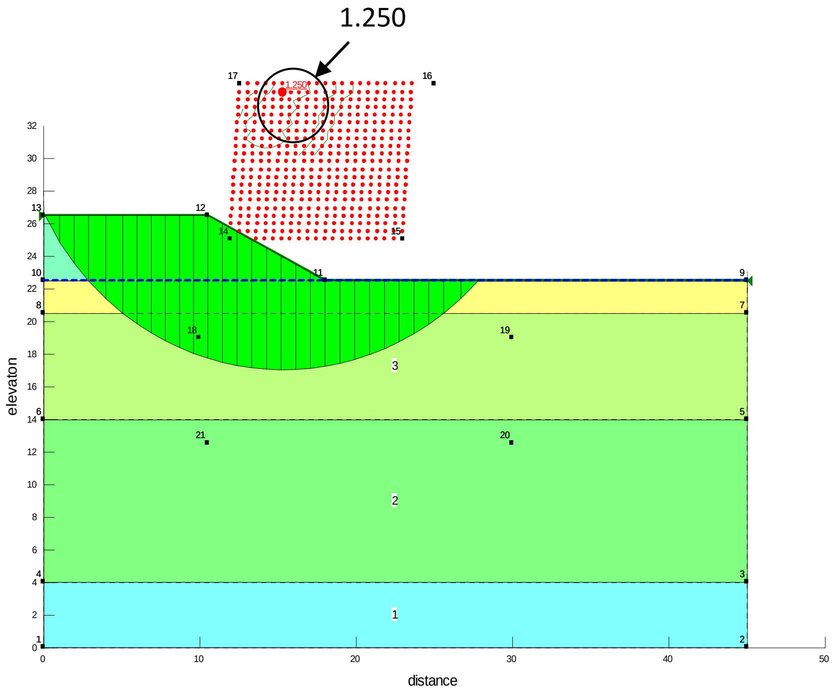


Figure 5.9 : Predicted failure surface using LEM for Muar test embankment – phase 2

Predicted failure surface is slightly deviate from the actual one. Calculated FOS values are equal to 0.951 and 1.250 for phase 3 and phase 2 respectively. However calculated factor of safety values reflects the instability condition. It can be seen that the SLOPE/W software can be used to predict the stability of the embankment accurately.

5.3 Results of the CKE embankments

Predicted and observed values for the vertical settlement at the center of the embankment and lateral displacement at the toe of the embankment are shown in the Table 5.7. Details of the models are given in the Figure 4.4 – 4.6 and parameters used for the analysis are shown in Table 4.8 – 4.10.

Table 5.7 : Predicted and observed deformation values for CKE

Embankment No	Predicted values (m)		Observed values (m)	
	Vertical settlement	Lateral displacement	Vertical settlement	Lateral displacement
K7+870	0.489	0.022	0.968	0.819
K6+850	0.763	0.059	0.666	0.089
K6+530	0.790	0.091	0.116	0.034

Predicted failure surfaces by using Finite element method and the Limit equilibrium method are attached to the Appendix B. Table 5.8 summarized obtained FOS values from the FEM, LEM and the Matsuo chart as follows,

Table 5.8 : Predicted FOS values for CKE

Embankment No	FOS			
	SLOPE /W (Morgenstren & Price)	PLAXIS (Phi - c reduction)	Matsuo chart	
			Based on field data	Based on “d” and “δ” obtained from PLAXIS
K6+530	1.746	1.711	> 1.7	1.43
K6+850	1.739	1.411	1.43	1.53
K7+870	1.612	1.487	1	> 1.7

Factor of Safety values obtained for the embankment K6+530 and K6 +850 under FEM, LEM and Matsuo chart are almost similar. Among these three embankments K7+870 is a failed embankment. Result obtained from the Matsuo chart (based on the field data) shows an instability condition. Since factor of safety value obtained from the Matsuo chart is equal to 1, it reaches to the failure criterion line. However LEM and the FEM give larger factor of safety values and they don't reflect this instability condition. This may be due to the inaccuracies of the soil properties used for modeling or due to a construction issue such as lack of compaction etc. Soil properties used were obtained by using cone penetration test results and empirical formulas. Then the obtained values are approximated values and can deviate from the actual values. So to find out the accurate soil properties back analysis was carried out to the K7+870 embankment. Here shear strength parameters of the different soil layers were reduced until get the lower factor of safety value. In layer 1 friction angle was reduced from 28^0 to 15^0 and in layer 2, un-drained cohesion was reduced from 26 kPa to 20 kPa. In layer 3 friction angle and the un-drained cohesion values were reduced from 25^0 to 15^0 and 6kPa to 5kPa. New deformation values and the soil parameters are given in Table 5.9 and Table 5.10. Table 5.11 summarized obtained FOS values from the FEM, LEM and the Matsuo chart.

Table 5.9: Deformation values for new K7+870

Embankment No	Predicted values (m)		Observed values(m)	
	Vertical settlement	Lateral displacement	Vertical settlement	Lateral displacement
K7+870	0.644	0.106	0.968	0.819

Table 5.10: FOS values for new K7+870

Embankment No	FOS		
	SLOPE/W (Morgenstern and Price)	PLAXIS (Phi - c reduction)	Matsuo chart (Based on field data)
K7+870	1.007	1.041	1

Table 5.11: Soil properties of the new K7+870

Layer	γ_{unsat} (kN/m^3)	γ_{sat} (kN/m^3)	Φ_u /Φ′	C_u / C′ (kPa)	Ky (m/day)	Kx (m/day)	E (kPa)	V	C_C	C_S	e_{int}	Model	Condition
Layer 1	17	18	15	0.1	86	86	15400	0.3	-	-	-	MC	Drained
Layer2	16	16.5	0.1	20	$1.8*10^{-4}$	$9*10^{-5}$	4000	0.35	0.73	0.073	0.5	SS	Un-Drained
Layer3	16	17	15	5	$9*10^{-3}$	$9*10^{-3}$	12600	0.33	-	-	-	MC	Drained
Fill	16	20	30	5	1	1	3000	0.3	-	-	-	MC	Drained

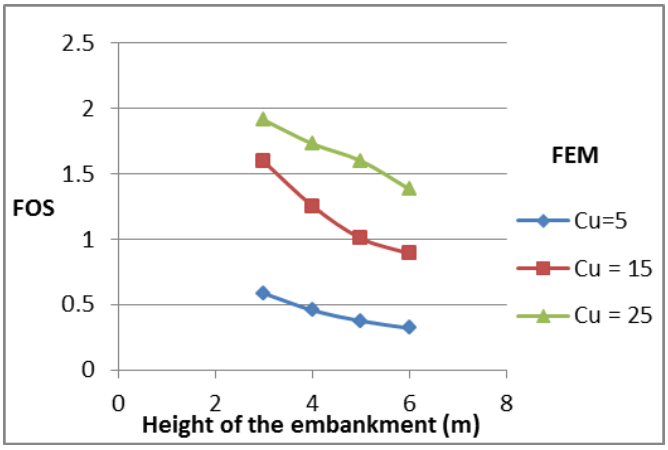
5.4 Effect of Embankment Height, Width and Sub Soil Parameters on the Stability using field observation data

Hypothetical embankment was selected as shown in Figure 4.7 and effect of the embankment width, height and the sub soil parameters on the stability was analyzed by using FEM, LEM and Matsuo chart. Soil parameters used for the analysis are given in the Table 4.12. Predicted failure surfaces by using FEM and LEM for some trial embankments have been attached to the Appendix C.

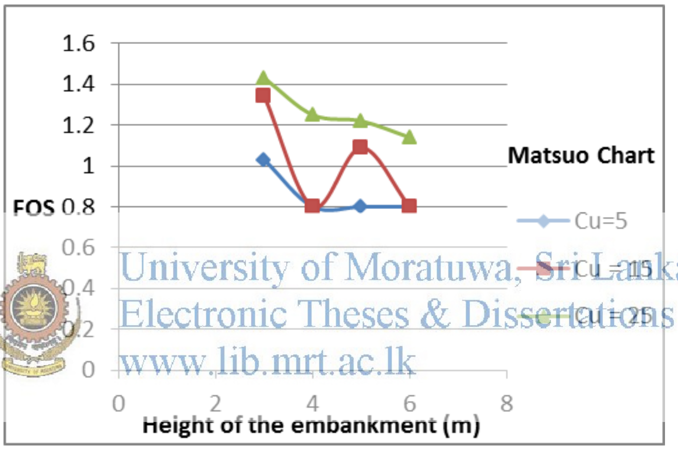
5.4.1 Variation of the FOS values with the sub soil parameters

Figure 5.10 shows the variation of the FOS values with the un-drained shear strength for different embankment heights. Here width of the embankment is equal to 3m and Figure 5.10 (a), (b) and (c) separately show the results obtained from the FEM, Matsuo Chart and LEM.

All these three figures show similar pattern of variation. According to the results, it can be said that factor of safety values increase with the increasing of the un-drained strength of the soil. It is a well-known factor that the stability of an embankment on soft soil depends upon the un-drained strength of the sub soil. Generally stability of an embankment is directly proportional to the shear strength of the underlain sub soil. Hence higher the strength tends to increase the stability of the embankment. On the other hand soil gains strength due to consolidation and with time strength of the soil increase.

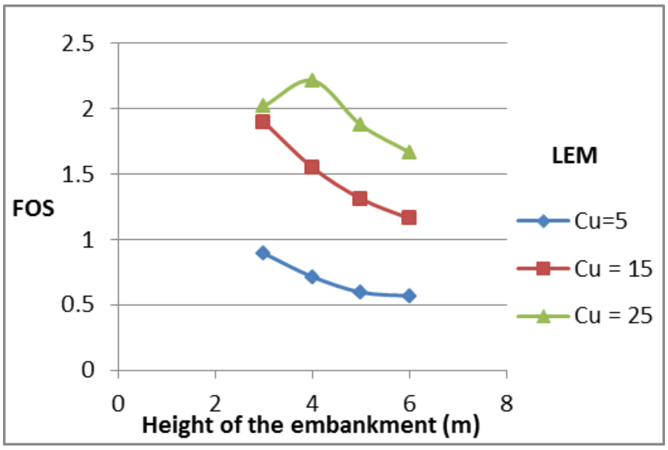


(a)



Based on the “d” and “δ” obtained from the FEM analysis.

(b)

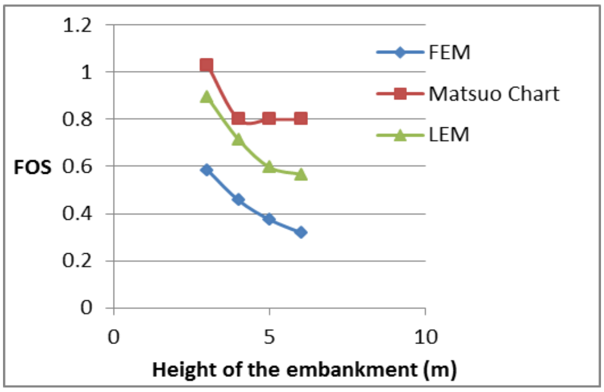


(c)

Figure 5.10 : Variation of the FOS with un-drained shear strength (a) FEM; (b) Matsuo Chart; (c) LEM.

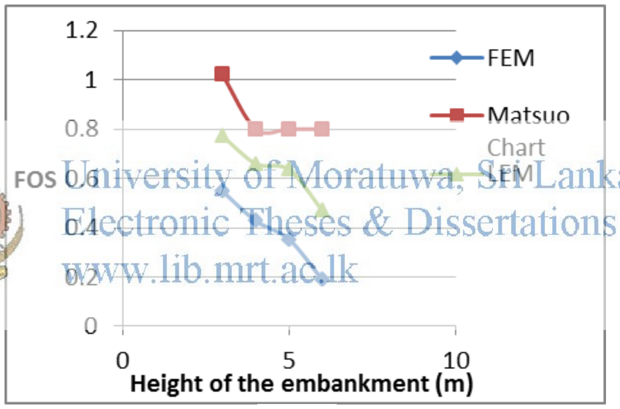
5.4.2 Variation of the FOS values with the embankment height

Variation of the FOS values with embankment height for various shear strength parameters are given in Figure 5.11 - 5.13 and Appendix D.



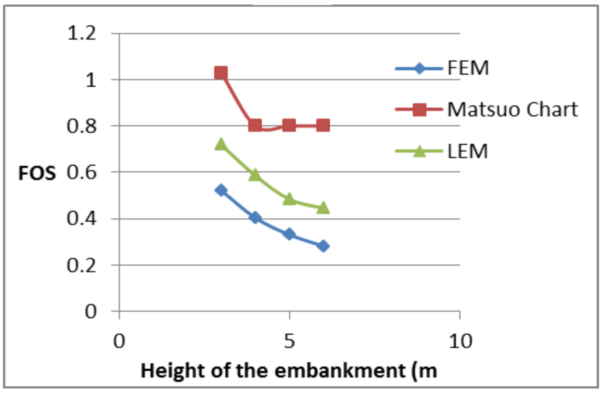
B = 3m

(a)



B = 5m

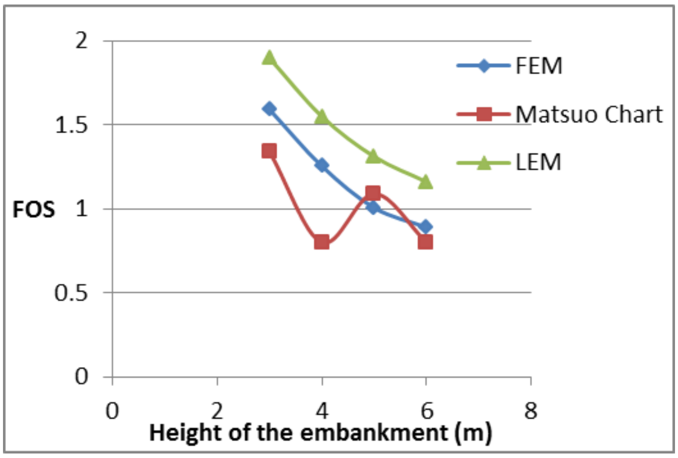
(b)



B = 8m

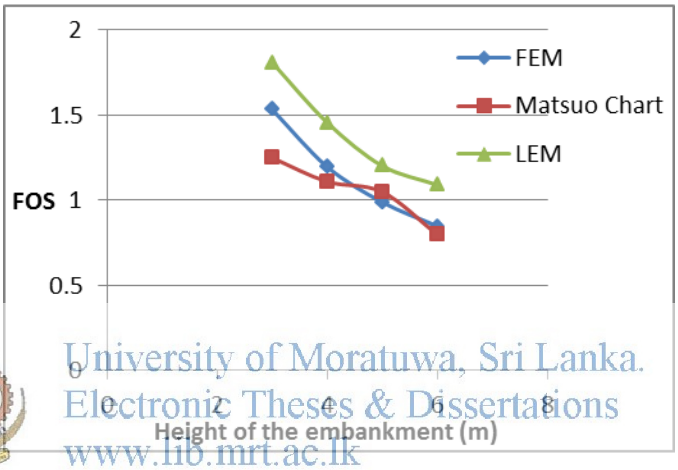
(c)

Figure 5.11 : Variation of the FOS with the embankment height / Cu =5 (a) B=3m; (b) B=5m; (c) B=8m



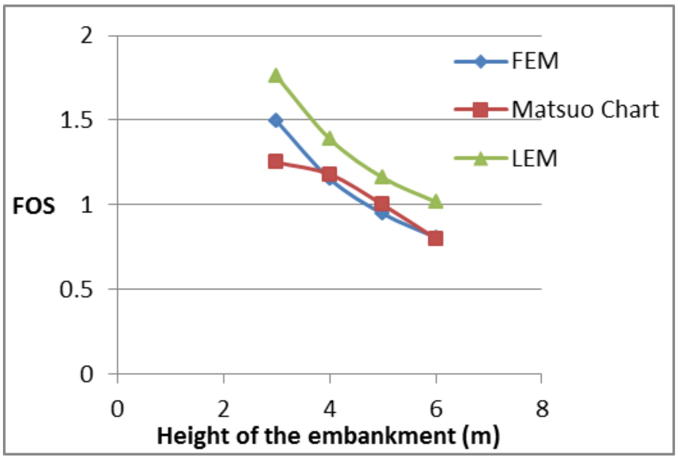
B=3m

(a)



B=5m

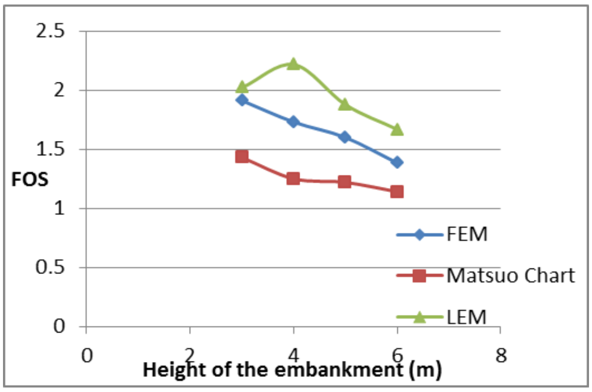
(b)



B=8m

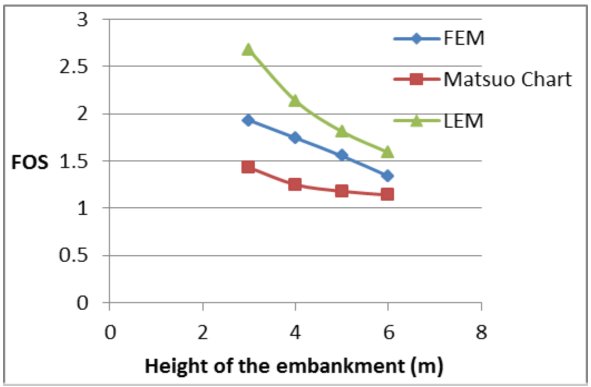
(c)

Figure 5.12 : Variation of the FOS with the embankment height / Cu =15 (a) B=3m; (b) B=5m; (c) B=8m



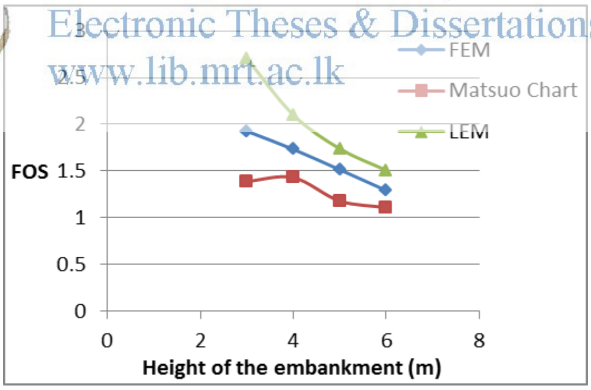
B=3m

(a)



B=5m

(b)



B=8m

(c

Figure 5.13: Variation of the FOS with the embankment height / Cu =25 (a) B=3m; (b) B=5m; (c) B=8m

According to the Figure 5.11-5.13 it can be seen that the factor of safety values decrease with the increase of the embankment height. Increase of the embankment height means it will put more load or stress on the soft soil. Then sometimes underlain soil cannot bear this extra load and when it exceeds the available shear strength of the underlain soil embankment can be failed.

FEM, LEM and the Matsuo chart show similar pattern of variation as shown in the Figure 5.11-5.13. However LEM gives higher factor of safety value compare with the other methods. For very weak soils as shown in Figure 5.11, it is not reliable to use Matsuo chart (based on "d" and "δ" obtained from the finite element analysis) to predict the stability of the embankment. Large values of "d'' and "δ/d'' represent a point beyond the failure criterion line and it is difficult to find an exact value for the FOS. And also for lower un-drained shear strength values Matsuo chart gives higher factor of safety values than the LEM and FEM for higher shear strength values it gives lower factor of safety values than the LEM and FEM.

In addition according to the above figures it can be seen that the FOS values obtained from the Matsuo chart (based on "d" and "δ" obtained from the finite element analysis) and the FOS values obtained using Phi-c reduction method gives similar results for large C_u values.

5.4.3 Variation of the FOS values with the embankment width

Variation of the FOS values with embankment width for various shear strength parameters are given in Figure 5.14 - 5.16 and Appendix E.

According to the Figure 5.14 - 5.16, it can be said that there's no significant variation of the FOS values with the embankment width. That means increase of the embankment width will not significantly affect to the embankment stability. LEM method gives the higher FOS value compare with the other two methods. The difference between the FOS values obtained from the FEM and LEM is higher for lower shear strength soils. For lower un-drained shear strength values Matsuo chart gives higher factor of safety values than the LEM and FEM as shown in Figure 5.14, but for higher shear strength values it gives lower factor of safety values than the LEM and FEM.

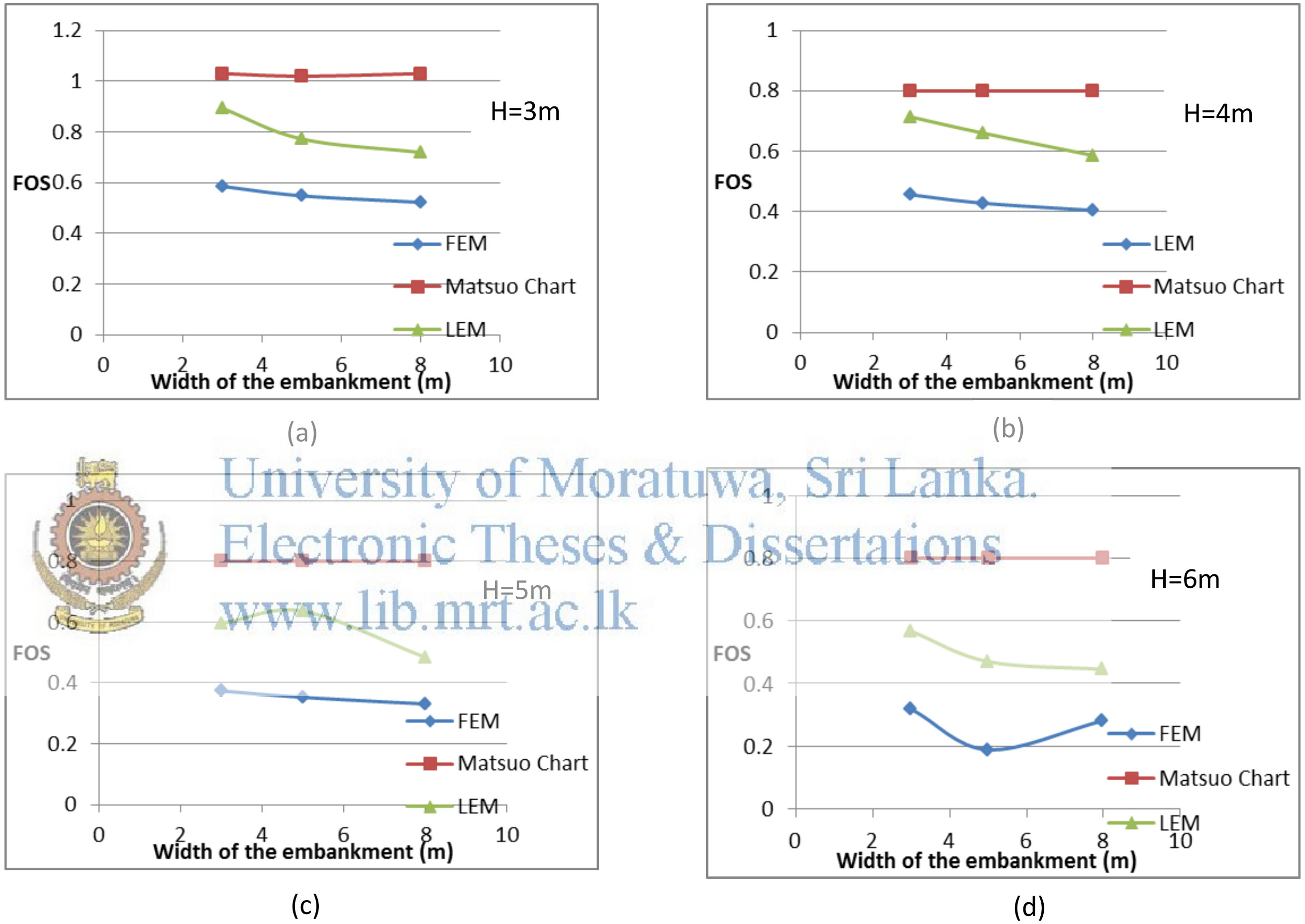


Figure 5.14: Variation of the FOS values with embankment width / Cu =5 (a) H=3m; (b) H=4m; (c) H=5m; (d) H=6m

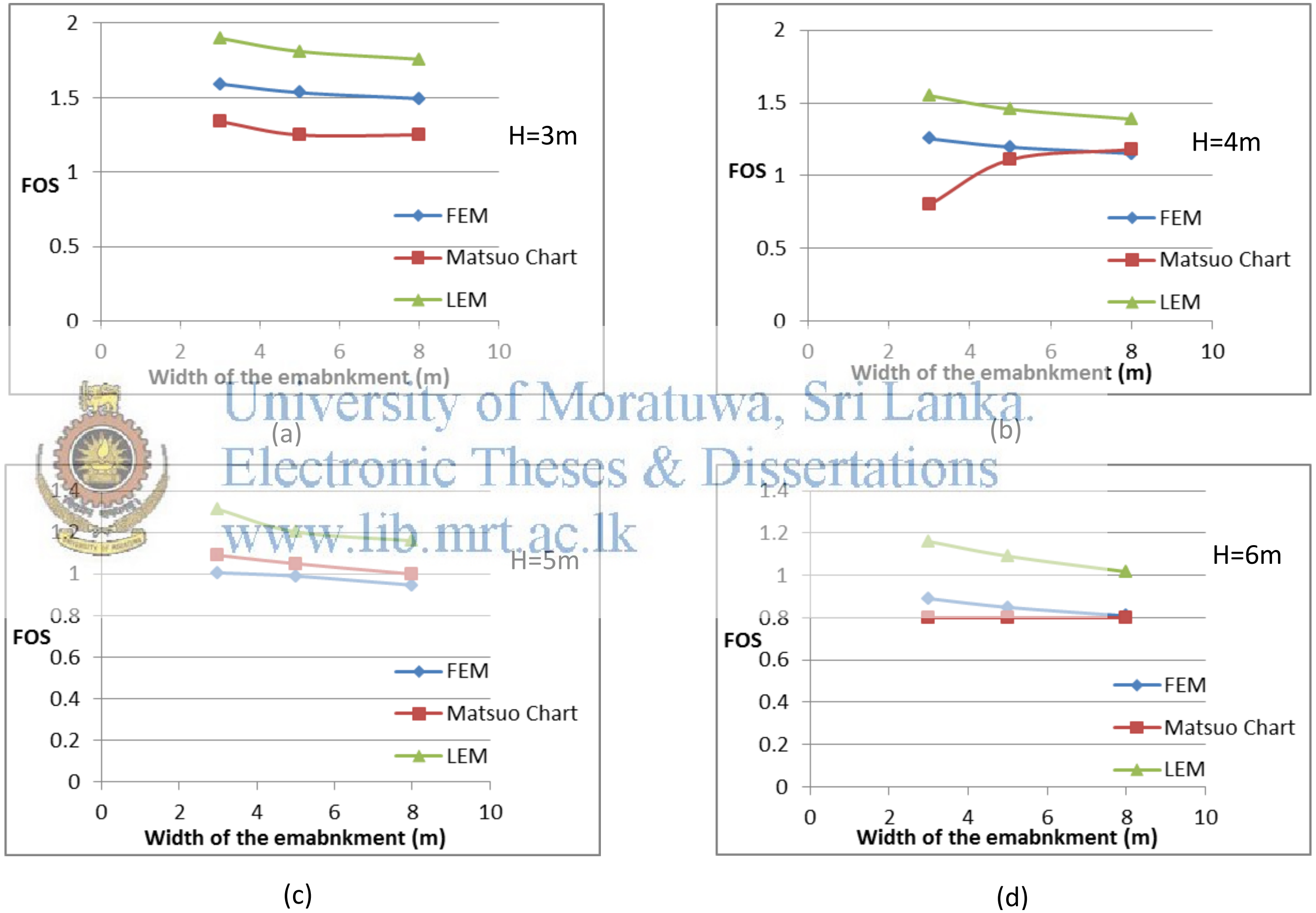


Figure 5.15 : Variation of the FOS values with embankment width / Cu =15 (a) H=3m; (b) H=4m; (c) H=5m; (d) H=6m

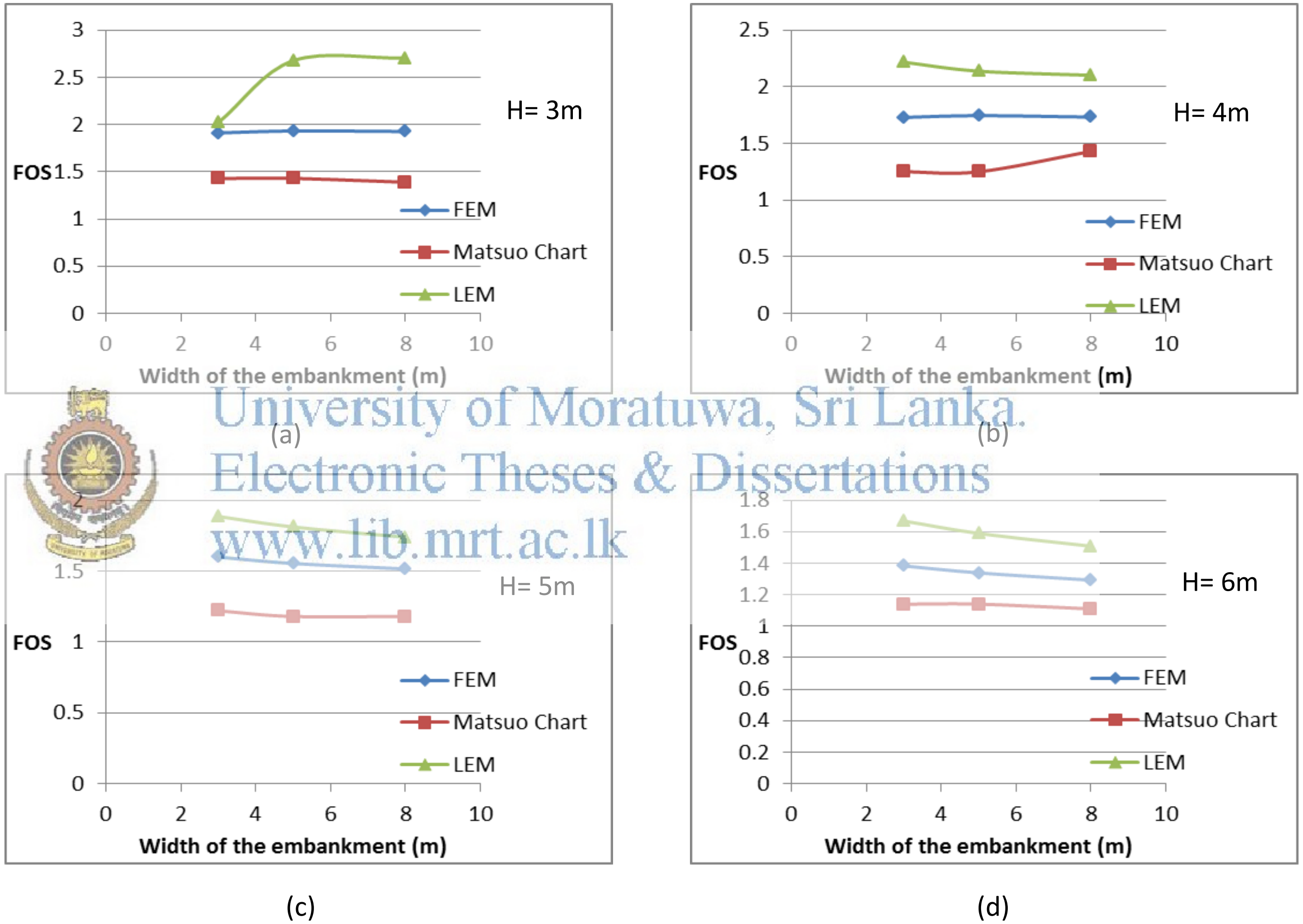


Figure 5.16 : Variation of the FOS values with embankment width / Cu =25 (a) H=3m; (b) H=4m; (c) H=5m; (d) H=6m

CHAPTER 6 CONCLUSION AND RECOMMENDATION

According to the results obtained from the Aiko test embankment predicted deformation values (lateral deformation at the toe of the embankment and the vertical settlement at the center of the embankment) are much more similar to the observed and calculated values. In Muar test embankment also predicted values for the lateral displacement, vertical settlement and the excess pore water pressure are agree well with the observed data. All predicted values were obtained using Finite Element Method. So it can be concluded that Finite Element Method can be used to predict the deformation characteristics of embankment very accurately.

In Aiko embankment results given by the SS model are tally with the observed and calculated data. However when soil exhibits secondary consolidation, It can be observed from the results that SSC model predicts the deformation of the embankment reasonably.

Factor of safety values given by FEM for the Aiko embankment are similar to the factor of safety values which were obtained from the LEM. Muar test embankment and the CKE embankments also show this kind of similarity. According to the results of the Muar test embankment predicted failure surface is similar to the observed failure surface. So, both of the FEM and the LEM have ability to predict the stability of embankment very precisely. In CKE embankments factor of safety values obtained from the LEM and the FEM are much similar to the factor of safety values given by the Matsuo chart. So it can be concluded that Matsuo chart also can be used to evaluate the stability of embankment.

Hypothetical embankment was used to analyze the effect of embankment width, height and the sub soil parameters on the stability by using FEM, LEM and Matsuo chart. According to the obtained results it can be seen that factor of safety values increase with the increasing of the un-drained shear strength of the soil. On the other way stability of an embankment is directly proportional to the un-drained shear strength of the underlain sub soil.

For every un-drained shear strength values factor of safety values are decrease with the increase of the embankment height. But it can be seen that there no significant variation of the factor of safety values with the embankment width. So it can be concluded that stability of the embankment inversely proportional to the height of the embankment, however increase of the embankment width will not significantly affect to the embankment stability.

Factor of safety values given by the Matsuo chat are higher for lower shear strength soils than the FEM and LEM. But for higher shear strength values it gives lower FOS values. On the other hand for very weak soils Matsuo chart cannot be used to find an exact factor of safety value. It only represents the instability condition (FOS<1). However there's no significant difference between the FOS values which were obtained from the FEM, LEM and the Matsuo chart. So finally it can be concluded that Matsuo chart can be applied for various embankment conditions to predict the stability.

Recommendations to future works

1. Stability of an embankment not only depends on the embankment width, height and sub soil parameters but also it depends on the filing material and ground water conditions. So we can extend this study to check the effect of the above parameters on the stability of the embankment.

2. Further this study can be extended for various sub soil conditions (soil with different un-drained shear strength, friction angle, young's modulus and Poisson's ratio).

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Appendix – A

Manual Calculation of the Aiko Embankment

Appendix – B

Failure Surfaces of the CKE Project

Appendix – C
Failure Surfaces of the Parametric Study

Appendix – D

Variation of the FOS values With the Embankment Height

Appendix – E
Variation of the FOS values With the Embankment Width