YOUNG GEOTECHNICAL ENGINEERS CONFERENCE 2013

A Presentation of
Geotechnical Engineering Projects of
Young Sri Lankan Engineers

March 02, 2013
At ICTAD Auditorium

Organised by the
SRI LANKAN GEOTECHNICAL SOCIETY

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Message from the President - SLGS

Research is the key to the development and prosperity of a country. A country in the development path need to develop new knowledge and innovative techniques thorough well directed research to resolve numerous problems it encounters.

Sri Lankan engineers encounter many challenges in the field of Geotechnical Engineering. To overcome those challenges we need to be aware of the current developments in the field and should find ways of applying this new knowledge under our own conditions and make further innovations. Over the last twenty six years the Sri Lankan Geotechnical Society has provided a forum for disseminating new knowledge in the field of geotechnical engineering and promoting research.

Having clearly identified the need to promote research in the field of geotechnical engineering from an early stage in a carrier, the Project Day competition among Sri Lankan undergraduates doing projects in the field of geotechnical engineering was commenced in year 2001. This was held as an annual event thereafter.

Now, SLGS wants to extend it further to the practicing engineers.

"Young Geotechnical Engineers Conference" is intended specially for the young practicing engineers involved in the field of geotechnical engineering. They may formulate a project based on correct engineering fundamentals to critically analyse a problem encountered in practice and come up with an innovative solution. There will be many senior engineers and academics to guide them in this endeavour. With many infrastructure development projects in progress at present there will be enough opportunities to identify a research area.

Alternatively, they may present research done for their postgraduate degrees.

There may be many engineers involved in creative work but not making any attempt to publish them. If they start to write up what they have done they themselves would be able to identify the shortcomings and sharpen their work. Through the "Young Geotechnical Engineers Conference", SLGS hopes to encourage the Young Civil Engineers of the country to publish their work in the field of geotechnical engineering and enhance their skills in research as well as skills in writing and presentation. This will be held as an annual event.

Young engineers are expected to publish their findings in a concise four paged paper and make a 20 minute presentation. The best paper will receive a cash award and a certificate.

This year there are eight papers on a wide variety of topics and I thank all the authors for their interest and commitment.

I also wish to convey my sincere gratitude to the panel of evaluators; Emeritus Professor B. L. Tennekoon, Mr. K. S. Senanayake and Prof. T. A. Peiris.

Prof. Athula Kulathilaka
President - SLGS

Once it was well said by the highly esteemed Sri Lankan Scholar - Kumaratunga Munidasa

"Aluth aluth de nothanana jathilya lova nonagee- singa kema beri wunu thena lagee gaya maragee"

"A nation that does not attempt to innovate and create new knowledge will not prosper. It will mourn to a sorrowful death when the round of begging fails"

CONTENTS

BACK ANALYSIS OF A LARGE LANDSLIDE IN OVERCONSOLIDATED CLAYS

H.M.D. Harshani, B.S.S.Dareeju, P.Tommasi and Alesandro Pagaliaroli

Abstract : *The Orvieto hill, situated in Province of Terni, South-Western Umbria, Italy, has been affected by landslide phenomena that have been well documented since the middle age by historical sources. Monitoring system has been started using casagrande type piezometers and inclinometers in 1982 in the northern slope of the Orvieto hill that was involved in the huge 1900 Porta Cassia land slide. Following a geotechnical investigation and monitoring results, back analysis is carried out using conventional limit equilibrium methods and finite element method to establish the mobilized strength of the slope and determine the mechanism of failure. The back analysis shows that the mobilized strength along the slope lies between residual and peak strength values, and therefore the slide must be a first time slide with progressive failure after the excavation or slide must be utilizing a pre-existing failure surface. However, extending yield zone along the rupture surface from the results of FEM, illustrates that Porta Cassia landslide utilizes a pre-existing failure surface and further toe excavation which has been made during the Rome Florence railway construction in 1865 has a considerable influence on this reactivation.*

Keywords: *Overconsolidated clays, landslides, Limit Equilibrium Method(LEM), Finite Element Method(FEM), Back analysis, shear zone*

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1. Introduction

Most of the active slow slope movements in clayey soils exhibits continuous movements on completely developed shear zones resulting from past major landslides and this is a highly concentrated area for researchers to carry out their studies. The very basic and accepted definition for landslide is ''the movement of a mass of rock, debris or earth down a slope''(Cruden, 1996). Landslides occur when shear strength of soil is less than the shear stress required to maintain equilibrium and they can be triggered by gradual processes such as weathering, or by external factors including: rainfall or vibrations, and human intervention. For a reactivated landslide or a first time landslide in a region of previously unstable slope, back analysis is an effective method to evaluate the shear strength of the soil in the region.

Skempton suggests that if the sliding mass undergoes a large shear displacement, e.g., several feet, then the shear strength of the material present along the slip surface will exhibit the residual value(Skempton,1985). Skempton (1964) also concludes "if failure has already occurred, any subsequent movement on the existing slip surface will be controlled by the residual strength.

For slope stability analysis, the Limit Equilibrium Methods are the most popular among engineers and researchers because these are well established. In LEM, the factor of safety (FS) is calculated using one or more of the equations of static equilibrium at the sliding mass. Advance numerical methods improve the assessment of slope stability analysis even though LE methods provide a sufficient feedback for geotechnical engineering purpose. The principle difference between above two methods is in the analysis approach since various equilibrium conditions are applicable in LEM whereas constitutive laws are also used in advance numerical methods.

This paper refers to the historical town of Orvieto; located on the top of pyroclastic formation underlining with overconsolidated clay. Orvieto hill was experienced with several landslides from ancient time (Tommasi et.al, 1996). One of the most disastrous landslide occurred on November 1900 in the Northern flank of the hill. Monitoring system has been in placed with casagrande type piezometers and inclinometers in 1982 in the northern slope of the Orvieto hill that was involved in the huge 1900 Porta Cassia land slide.

2. Methodology

2.1 Geological and Geotechnical Analysis

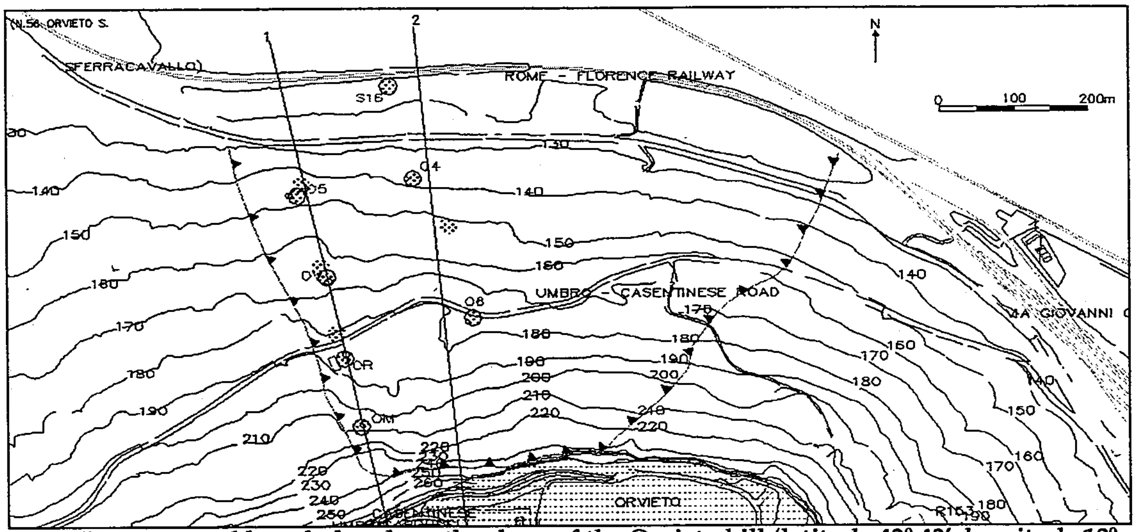

Figure 1: Instrumented bore holes along the slope of the Orvieto hill (latitude 42° 43′; longitude 12° 7′)

Orvieto is a major historical town which is located about 100km north of the Rome, has an artistic heritage. The scarp of the Porta Cassia land slide which was about 400m long and 600m wide in its central part, is presented in Figure 1. The displaced soil mass which has extended over the whole slope interrupted the main road to Orvieto (Umbro Casentinese National Road) at an intermediate level and Rome and Florence railway at the toe. (Tommasi et al. 1996). Further that paper indicated some authors showed that this (Porta Cassia) landslide was triggered by the excavation work carried out by railway department at toe of the slope during construction of the Rome Florence railway. However, Tommasi et al. (1996) has suggested that 1900 Porta Cassia landslide may be a reactivated movement considering comprehensive geotechnical data.

For this study two sections along the land slide area, shown in Figure 1, are created to check the stability. Section "1" which locates at the corner of the scarp, is going through O5, OV, OR and OM bore holes. Section "2" which locates along the middle of the landslide scarp goes through bore holes O4, O6 and OM.

Depth to the failure surface and rate of movement has been recorded by means of probe inclinometers inserted in aluminium casings. Figure 2 illustrates the inclinometer data; is used for identification of failure surfaces and their rate of movements along section 1.

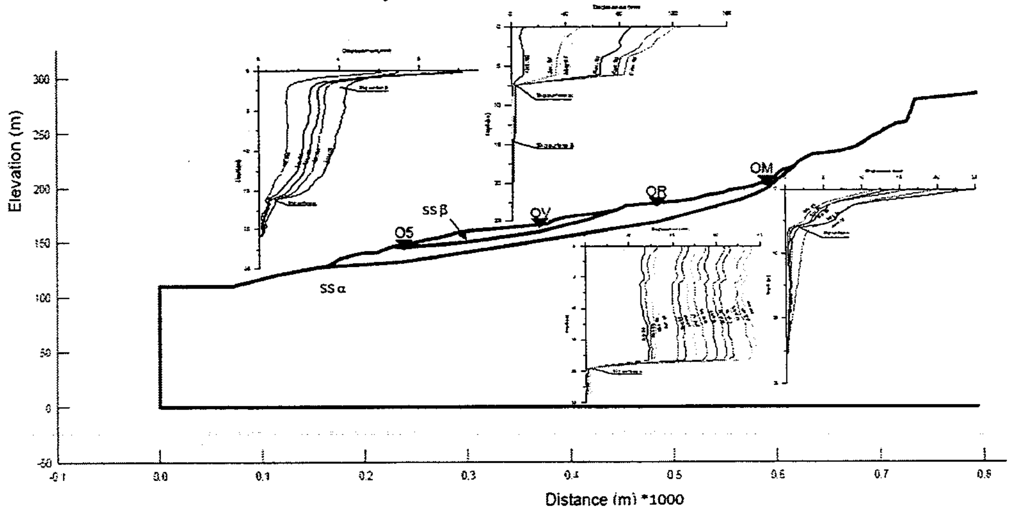

Figure 2: Kinematic characteristics along the slope for section "1"

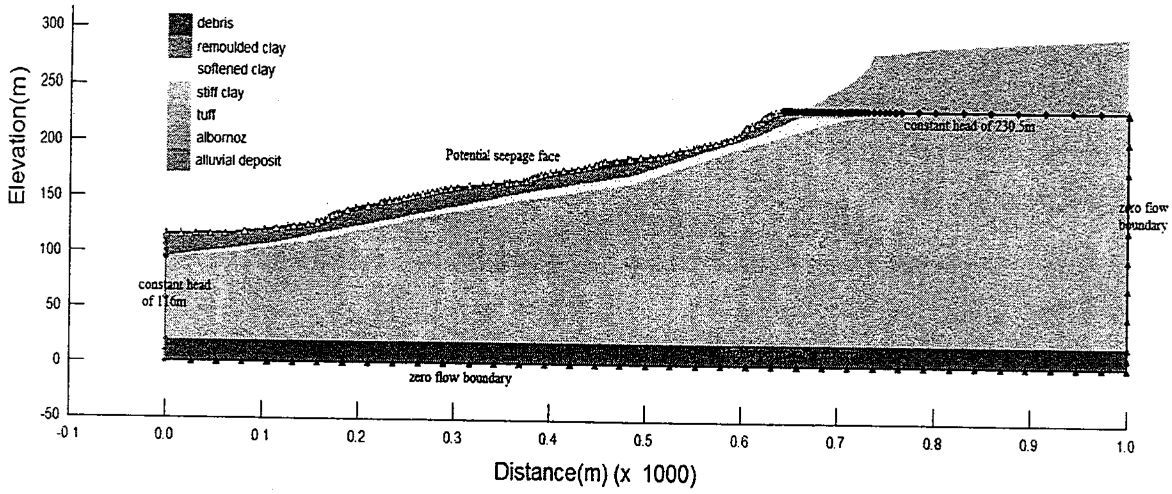

Figure 3: Geotechnical section of the slide with boundary conditions for seepage model

Inclinometer OR and O5 show a rigid movement of the landslide body on a well defined slip surface respectively at a depth of 20 meters and 16 meters below ground surface. On the other hand, inclinometers OM and OV show a mass creep profile

Visual analysis of borehole and physical-mechanical properties as natural water content, undrained strength, elastic wave velocity and $CaCO_3$ content are utilized to recognize the upper limits and lower limits of each available soil types and then construct layers stratigraphy. Subsoil consists of a layer of clayey debris and remoulded clay for upper cover, a middle softened clay layer and a stiff clay bedrock. (Figure:3)

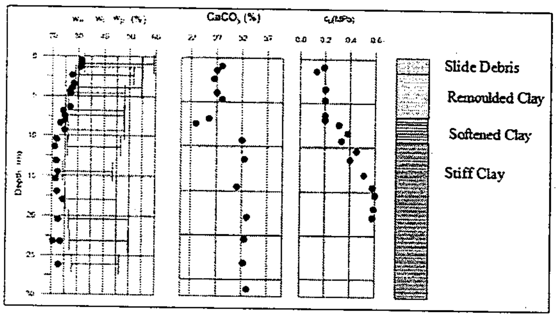

Figure 4: Log detail of borehole OM

2.2Seepage Analysis

Conceptual model of the hydraulic conditions in the clayey slope is developed using commercial finite element code SEEP/W (GEO-Slope 2007) and pore water pressure for this study is predicted under the steady state condition. Upper head boundary of the slope has been assumed to be one meter above the bottom of Alboronz layer and zero constant flow has been introduced to the right side boundary of the section considering its symmetric condition(Figure: 4)

2.3Stability Analysis

Conventional limit equilibrium method by method of slices was used to calculate the mobilized shear strength along the failure surface. Back analysis of the active landslide is possible considering that the factor of safety of an active landslide can be assumed to be unity according to the limit equilibrium theory. The profile before 1900 porta cassia land slide is constructed with the aid of contour lines which have drawn soon after the construction of the Roma-Florence railway at 1865. The slip surface is traced considering the inclinometer data, excavation during the railway construction and present morphology along the slope.

Table 01: Shear strength parameters of each soil type

		Remoulded clay	Softened clay	Stiff clay
Peak	C(kPa)	17.1	24.6	97.2
	φ (deg)	21.16	23.32	27.56
Residual	C(kPa)	0	0	0
	φ (deg)	11.6	11.6	11.6

Shear strength parameters obtained through the drained direct shear tests conducted on samples obtained from different layers are presented in Table 01. These values are adjusted till the factor of safety is unity (FS≈1.0) to determine the back calculated mobilized shear strength along the slip surface.

In order to model strain softening behaviour which cannot be modelled in LEM, a complete FEM analysis is done. Since SIGMA/W does not have a well defined strain softening model (SIGMA/W, 2007), the method proposed by Lo and Lee, 1973 (Figure: 5) is referred. Elastic Modulus (E) values, found by drained oedometer test, are used (Table 02)

Table 02: Elastic Modulus values from Oedometer test

Soil Type	Elastic Modulus (E)
Remoulded clay	3825.92kPa
Softened clay	6050.21 kPa
Stiff Clay	10446.8 kPa

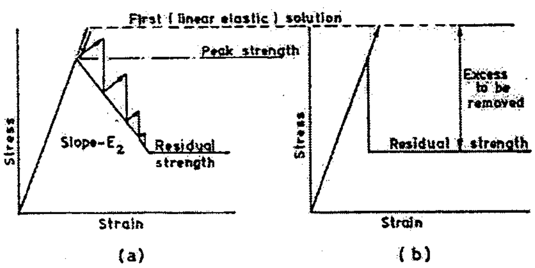

Figure 5: Stress-strain relationships for strain-softening material (Lo and Lee, 1973)

3. Results and Discussion

It is well clear that at depth, clay is stiff and apparently in-tact but however, becomes soften in upward direction as shown in Figure 4. This behaviour directly influence the permeability of soil and in here, it continuously decreases with the depth.

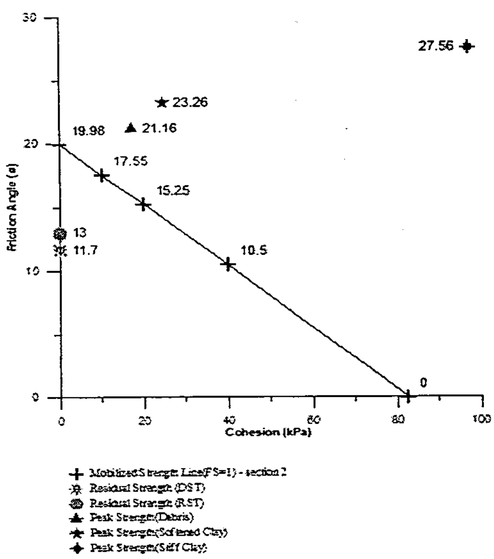

Fig:6 Comparison of mobilized shear strength with laboratory residual and peak values.

Calculated mobilized shear strength values from LEM lies between the laboratory measured residual and peak strength values (Figure 6). These results can be used to explain that the slide represents partial reactivation of pre existing slip surfaces or new slip surface with progressive failure. Furthermore, it is shown that FS changes due to railway excavation at the toe only by 2.2%. Therefore, conventional LEMs can not address the failure mechanism of 1900 Porta Cassia landslide and FEM is the fundamental to identify the actual failure mechanism.

By extending the results to FE analysis, it is clear that yield zone along the failure surface shows characteristics of a reactivated landslide and at the same time it is extending from toe of the excavation up to pre-failure surface. Also the landslide has occurred 35 years after the excavation for the railway construction. As this excavation depth (5m) is small comparable to London clay, it is not common for a first time failure to occur after 35 years. Therefore the slide must have utilized a pre-existing shear surface.

Therefore, it can be concluded that 1900 Porta Cassia landslide is not a first time landslide and it is pre-existing landslide which was reactivated by railway cutting and progressive failure.

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Comparison of the Ground Vibration due to Vertical and Horizontal Blasting Geometry: A Case Study

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Abstract: *This study was carried out to compare the ground vibration caused by vertical and horizontal bench blast(s) at a granitic gneiss rock quarry located in Colombo, Sri Lanka. For he above purpose, particle velocities of 38 and 35 horizontal and vertical bench blasts, respectively were measured in three perpendicular directions with the use of Instantel Blastmate II seismograph. In the blasts, Ammonium Nitrate (ANFO) (blasting agent) primed by a Gelatin Dynamite primer were electrically initiated. Scaled distance parameters (Maximum charge weight per delay and distance between blasting points to monitoring location) were also recorded. The extensively used equation for the propagation of wave proposed by Devine (1962) and Devine and Duvall (1963) was used for the prediction of peak particle velocities. Points were plotted with Peak Particle Velocity (PPV) in Y-axis against Scaled Distance ($D/Q^{0.5}$) in X- axis. Regression analysis was performed to define the line of best fit. At the end of statistical analysis, empirical relationships with good correlation were established for prediction of peak particle velocity for both types of blast. The established relationships, and result obtained are presented.*

Key words: *Horizontal bench blast, vertical bench blast Peak Particle Velocity, Scaled Distance*

1.0 Introduction

Blasting is the principal method of rock breaking in quarrying industries. Widely applied blasting geometry is the vertical (vertical bench blast). However, there is a trend to apply horizontal geometry (horizontal bench blast) in present days due to its economical advantages by grater production volume with wide range of particle sizes over the vertical bench blast per same drilling and blasting cost. However, it is also essential to evaluate the environmental impacts for both types of blasts to decide the most suitable blasting geometry. Ground vibration is the most critical impact, since it induces structural damages to nearby structures. Therefore, prediction of ground vibration for both types of blasts through the statistically reliable formula is vital.

2.0 Literature Review

According to Devine in 1962 and Devine and Duvall in 1963 (cited in Carlos et al. [1]), scaled distance relationship for cylindrical explosive charge (height of explosive/diameter of explosive> 6) can be defined for peak particle velocity prediction, as given by,

$$SD = D/W_d^{0.5} \qquad (1)$$

Where, SD, the scaled distance; D, absolute distance between the charge and the monitoring station (m); W_d maximum explosive charge per delay (kg).

The equation extensively used for seismic low of propagation proposed by Devine (1962) and Devine and Duvall (1963) (cited in Carlos et al. [1]) being able to define peak particle velocity (PPV) as given by,

$$PPV\ (mm/sec) = K * (SD)^{-\beta} \qquad (2)$$

Where, K is the ground transition coefficient; β, specific geological constant

3.0 Methodology

This study was done at Hesei Construction company quarry site at Kaduwela Divisional Secretariat division in Colombo District in Sri Lanka. In both types of blasts, ANFO explosive was initiated with a gelatin dynamite primer with electric initiation to produce shock energy. Applied vertical and horizontal bench blasts geometries are shown in Figures 1 and 2 respectively. Blasting parameters are also shown in Table 1 and Table 2 for vertical and horizontal bench blasts respectively.

Ground vibrations were measured for the 38 and 35 horizontal and vertical blasts, respectively with Instantel Blast Mate II seismographs over a period of 7 months. Each vibration reading was measured for three components of three perpendicular directions. Maximum instantaneous charge weight per delay for each blast was also recorded with the respective distance to the point of reference from the charge with GPS equipment.

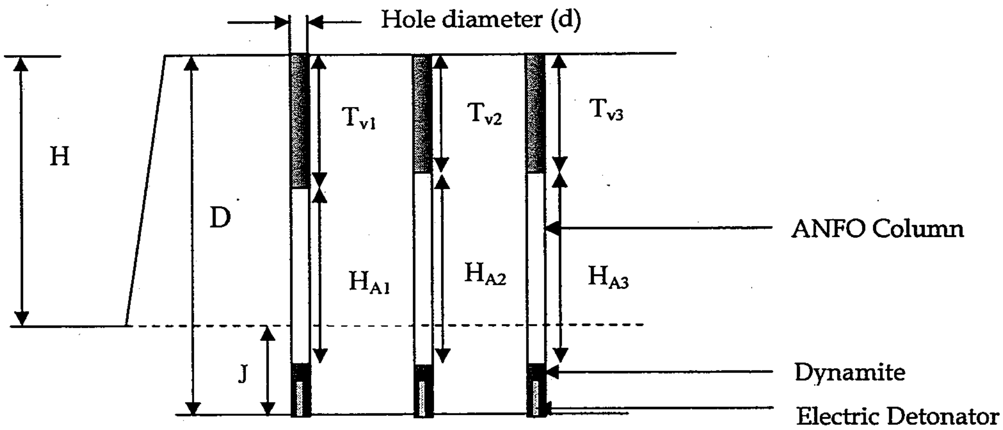

Fig. 01- Blasting geometry of the vertical bench blast

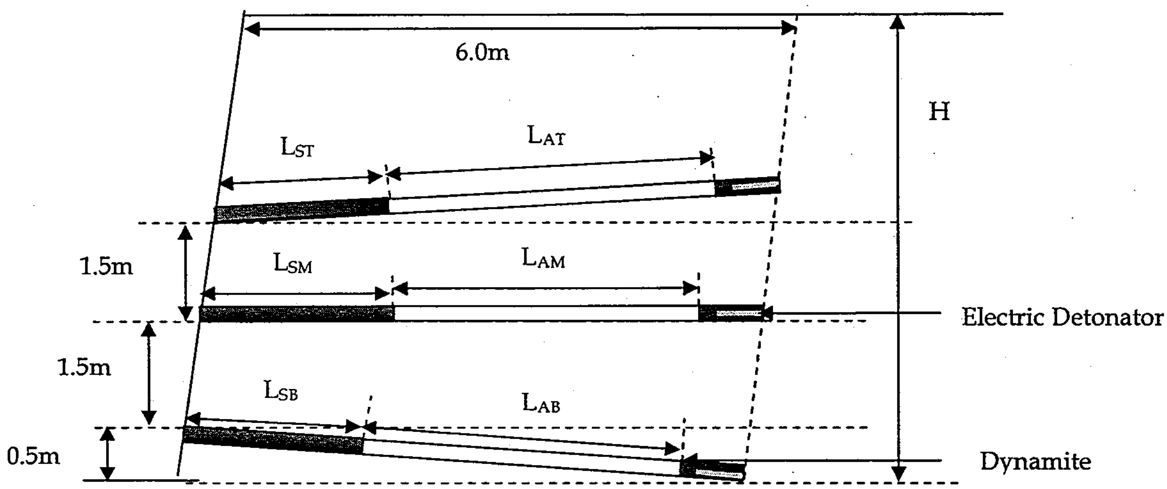

Fig. 02- Blasting geometry of a horizontal bench blast

Table 01- Blasting parameters of vertical blast

Parameter		Value
Drill hole diameter		64mm
Bench height(H)		5.25m
Hole depth (D)		6.00m
Sub drilling (J)		0.75m
Weight of Dynamite	First column	1.00kg
	Second column	1.00kg
	Third column	1.00kg
Height of ANFO column	First column (H_{A1})	2.85m
	Second column (H_{A2})	3.35m
	Third column (H_{A3})	3.60m
Stemming height	First column (T_{v1})	2.75m
	Second column (T_{v2})	2.25m
	Third column (T_{v3})	2.00m

Table 02- Blasting parameters of horizontal blast

Parameter		Value
Drill hole diameter		64mm
Bench height(H)		9.00m
Hole length		6.00m
Weight of Dynamite	Bottom row	1.00kg
	Intermediate row	1.00kg
	Top row	1.00kg
Length of ANFO	Bottom row(L_{AB})	3.60m
	Middle row(T_{AM})	3.30m
	Top row(T_{AT})	3.30m
Length of Stemming	Bottom row(L_{SB})	2.00m
	Middle row (T_{SM})	2.30m
	Top row(T_{ST})	2.30m

4.0 Result Analysis and Discussion

All collected data, for all three component of measurement for each blasting event were recorded and tabulated for data analysis.

4.1 Analysis of the particle velocity data

Peak Particle Velocity (PPV), which is the maximum particle velocity among the radial/longitudinal, vertical, and transverse components recorded from the same blast event, is considered to be a reliable measure for ground vibration caused by blasting. Statistically reliable equations for ground vibration attenuation were obtained for the vertical and horizontal blasts by plotting the data pairs of peak particle velocity and scaled distance in log scale and simple regression analysis as shown in figures 3 and 4 respectively.

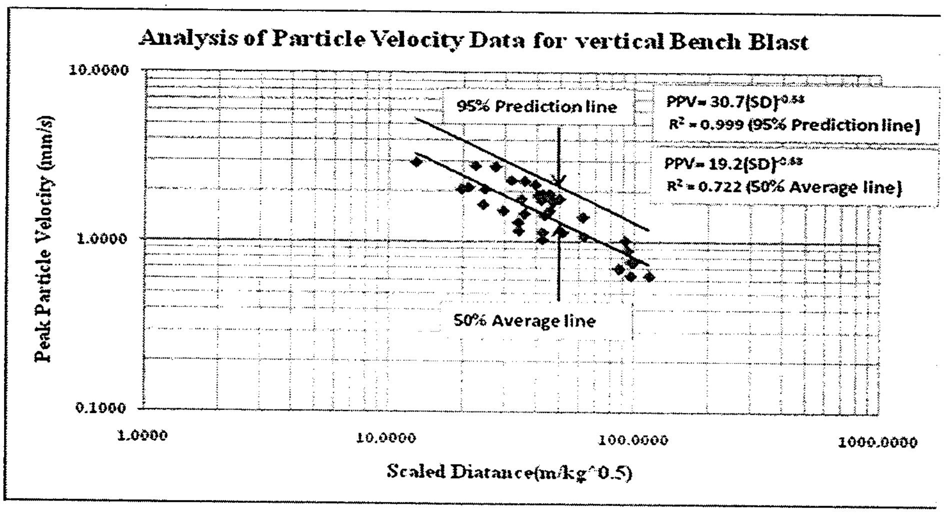

Fig. 03- Peak Particle Velocity Vs Scaled Distance for Vertical Bench Blasting

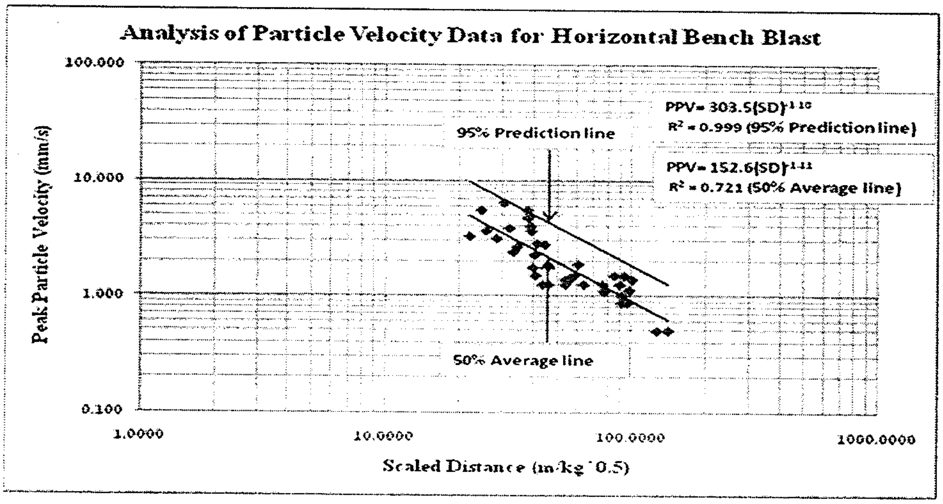

Fig. 04- Peak Particle Velocity Vs Scaled Distance for Horizontal Blasting

The derived upper bound ground vibration attenuation equations, for vertical and horizontal blasts are given in equations 3 and 4, respectively as follows.

$$PPV = 30.70 * (SD)^{-0.68} \quad (3)$$

$$PPV = 303.5 * (SD)^{-1.10} \quad (4)$$

Predicted peak particle velocity for vertical and horizontal blasting based on the maximum explosive utilization (Q) of 13.682 kg per delay time according to Mining License issued by Geological Survey and Mines Bureau, Sri Lanka to this metal quarry is shown in Table 03 with the use of derived ground vibration attenuation equations.

Table 03- Predicted peak particle velocity at 95% predicted level

Given Q(kg)	Distance (m)	Scaled Distance	Peak Particle Velocity(mm/s)-95% Upper bound level	
			Vertical blasting	Horizontal blasting
13.682	50	13.517	5.2255	17.305
13.682	200	54.070	2.0358	3.766
13.682	400	108.140	1.2707	1.757
13.682	600	162.210	0.9645	1.125
13.682	800	216.279	0.7931	0.820
13.682	850	229.797	0.7611	0.767
13.682	875	236.556	0.7462	0.743
13.682	900	243.314	0.7321	0.720

5.0 Conclusion

It was found that predicted ground vibration is higher in horizontal blasting geometry (horizontal bench blast) than that in vertical blasting geometry (vertical bench blast). Therefore, it is concluded that, the risk for ground vibration induce structural damages is less with vertical blasting geometry than horizontal when structures, to be protected are in close proximity to quarry sites.

6.0 Recommendations

Vertical bench blasting is strongly recommended when quarry activities are conducted in sensitive areas (structures, to be protected are in close proximity) giving due attention to peak particle velocity. It is also recommended to conduct the horizontal blasting geometry when quarries are located in isolated area with structures to be protected are in situated at larger distance from quarry sites.

Acknowledgement

Authors would like to thank Hesei Quarry management and the Geological Ssurvey & Mines Bureau Technical service for providing necessary facilities for field investigation, and field data collection during this study.

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Evaluate the proposed rock bench stability using the kinematic analysis of Stero-projection.

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Abstract: *Rock slopes are vulnerable to failure due to various factors such as adverse geological features, steep slope angle, poor drainage, significant weathering condition, etc. However such unstable rock slopes can be stabilized using appropriate techniques of rock bolting, anchoring, installing drainage systems and altering the slope geometry that reshaping.*

For the purpose of widening the Thennekumbura - Ragala road (B 413), at the stretch between 50 + 000 km to 50 + 230 km the prevailing rock layer at the right hand side of the road has to be excavated in a suitable way. As a result of this widening, a most serious disorder of rock fall or sliding of blocks can be expected if the excavation was done in improper way. Therefore a complete mathematical analysis of rock slope stability was done prior to the excavation, using the kinematic analysing method of Stereographic analysis. OpenStereo 0.1.2 software was used to plot the data on Equal-area stereonet hemisphere and analysis was done manually. This analysis reveals that number of location of the rock slope in Ragala road is intersected by well developed joints systems producing wedge failures that critical for unstable. Therefore, angle of proposed rock bench has been redesigned in those locations to prevent sliding of the wedges by increasing the factor of safety.

Keywords: Rock slope stability, Stereographic analysis, Discontinuities, Wedge failure, Factor of safety

1. INTRODUCTION

Landslides in roads, mainly rock mass sliding is one of a critical issue that creates unexpected traffic due to termination of the access, which is severe in low road density areas in Sri Lanka. A central hilly area of Sri Lanka is one of the example which these disasters has been reported with increasing frequency. Mainly four types of rock mass sliding that, plane failure, wedge failure, block failure and circular failure could be generate due to the key disturbance factors of adverse geological features and man made activities.

Thennekumbura – Ragala road (B 413) is also becoming more narrow at the stretch between 50 + 000 km to 50 + 230 km being the width is nearly 2.0 m. This location belongs to Walapane DS Division of Nuwaraeliya district. The left hand side of the road is sloping downwards to so far with approximate slope angle of 50 degrees while a prevailing rock layer is existing at the right hand side of the road. This road has to be widened to improve the transport facilities to Ragala area and to improve the socio economic condition of the residents at the area. For the purpose of widening this road, the existing rock layer at right has to be excavated in an appropriate way. The Hunan Construction Engineering Group Corporation is the contractor of this rock blasting activities and Landslide Research and Risk Management Division of the National Building Research Organization has involved with the monitoring and evaluating of the stability of proposed rock bench.

For this evaluation a kinematic analysing method of Stereographic analysis was used. Stereo plot is a particular method that used to have a 2 dimensional projection of a 3D view of the rock slope which makes the analysis easier. For this analysis a detailed evaluation of properties of the rock which contribute to the rock instability, structural attitude of every discontinuities and location details were gathered.

2. METHODOLOGY

2.1 Data Collection and Field Survey

As the first step of our research, all details which contribute to the rock instability were gathered in each five meter interval of the stretch between 50 + 000 km to 50 + 230 km of Thennekumbura-Ragala road. The gathered data consists with structural attitude (dip/strike) of all joints and fractures, rock type, weathering condition of the rock, drainage, geomorphology of the area and location details of GPS values. In addition to this proposed rock bench details including proposed excavate volume, proposed bench angle and bench height were collected from the relevant organization of this road widening that, Hunan Construction Engineering Group Corporation.

2.1.1 Details of Geomorphology and Geology of the area

Main rock types encountered in the whole rock layer are Garnet Biotite gneiss and Charnokite gneiss.

- Between the chainages of 50+000 km to 50+095 km Garnet Biotite gneiss with quartz layer is exists. In between the existing quartz layer and Garnet Biotite gneiss a weak rock layer can be identified which can increase the possibility of sliding.
- Exposing rock layer in between 50 + 045 km to 50 + 075 km is completely weathered, that becoming substantially discoloured with organic fabric and can be excavated by hand easily.
- At the 50 + 075 km a marble band is existing which extends up to 50 + 090 km.
- After the chainage of 50 + 115 km the main rock type is Charnockitic gneiss that embedded with quartz layers in some locations were identified.

The above observed at the foot of the rock stretch and within the vicinity of the area (Figure 01).

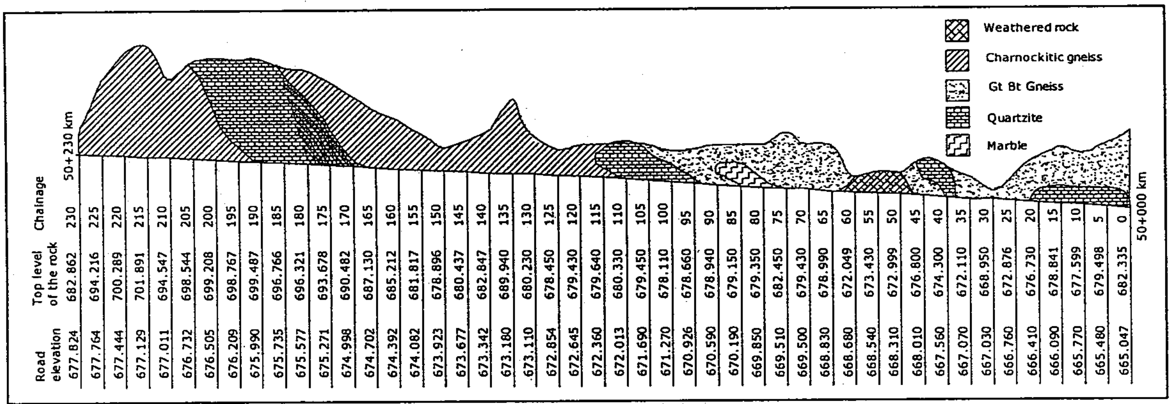


Figure 01: Existing rock types in between 50 + 000 km to 50 + 230 km in road

2.1.2 Details of structural attitude of the discontinuities

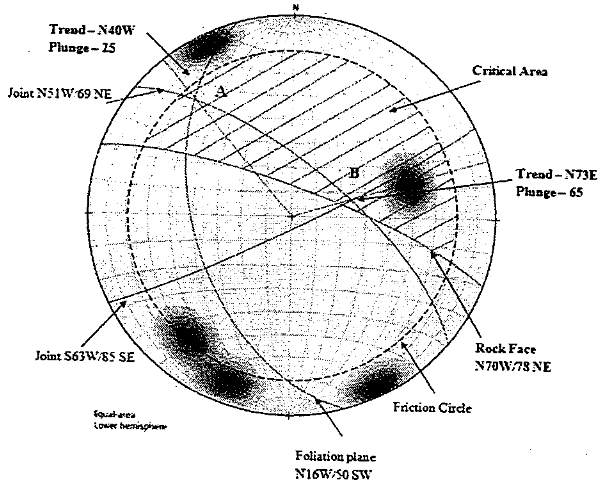
Table 01: Major joints sets in between 50 + 000 km to 50 + 230 km in Thennekumbura – Ragala road

Chainage (km)		Discontinuity (Strike/Dip) – Measured in the field		
50 +000 to 50 +020	In lower layer of Quartzite	Foliation Plane	N20W / 39^0 SW	
		Joints 1	N55W / 78^0 NE	5 joints per meter
		Joints 2	N30E / 90^0	5 joints per meter
	In upper layer of Garnet Biotite Gneiss	Foliation Plane	N16W / 50^0 SW	
		Joints 1	S63W / 85^0 SE	4 joints per meter
		Joints 2	N51W / 69^0 NE	3 joints per meter
50 + 040		Foliation Plane	S30E / 64^0 SW	
		Joints 1	S79W / 81^0 SE	2 joints per meter
		Joints 2	S33E / 71^0 NE	3 joints per meter
50 + 065		Foliation Plane	S10E / 54^0 NE	
		Joints 1	N78W / 90^0	6 joints per meter
		Joints 2	S13E / 54^0 SW	3 joints per meter
50 + 125 to 50 + 175		Foliation Plane	N31W / 42^0 SW	
		Joints 1	EW / 84^0 N	
		Joints 2	S66W / 79^0 SE	
		Joints 3	N14E / 76^0 E	

2.2 Data analysing for rock slope stability using Steronet

The existing in-stabilizations with the available discontinuities were analysed using stereo-net as follows (Figure 02).

Chainage - 50 + 000 km to 50 + 020 km



Chainage - 50 + 110 km to 50 + 155 km

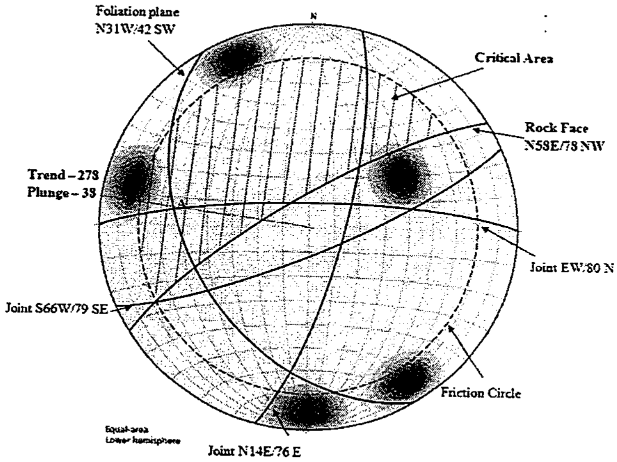


Fig 02: Stereo plot of the discontinuities

2.3 Calculation the Factor of Safety against rock mass failure

Typically, wedge failures are existing due to the intersection of discontinuities and as a result of sliding along the line of intersection. The size of the blocks can be defined by the space between the fracture systems. The critical area has been defined as where the dip of the rock face > plunge of the intersection (φi) > friction angle ($\emptyset$). Here the angle of friction between two layers is assumed as 20^0. However with the increasing of weathering conditions this can be further reduce, that decrease the factor of safety against sliding.

$$FOS = \frac{\sin(\beta)}{\sin(0.5\,x\,\varepsilon)} X \frac{\tan(\emptyset)}{\tan(\varphi i)}$$

ε = angle between 2 slide planes
β = angle between line of intersection and 1 joint plane

3 RESULTS AND DISCUSSION

Table 02: Wedge failure parameters (50 +000 to 50 +020)

Intersection	β	ε	$\emptyset$	φi	Intersection	β	ε	$\emptyset$	φi
A	80.5	61	20	25	B	82	26	20	65

At the intersection A the wedge is formed due to the intersection of a joint plane of N51W / 69^0 NE and the foliation plane with N16W / 50^0 SW. The safety factor for this wedge sliding is as follows.

$$FOS = \frac{\sin(80.5)}{\sin(0.5\,X\,61)} X \frac{\tan(20)}{\tan(25)} = 1.52$$

Here the factor of safety is slightly over than 1.5 without considering the availability of tension cracks and water filling of them. However with the other external factors, other discontinuities and influence of the rock blasting activities a wedge sliding can be predict

At the intersection B the wedge is formed due to the intersection of two joint planes of S63W / 85^0 SE and N51W / 69^0 NE. However, this wedge is overhanging with the slope face and not creating a wedge sliding. That means the intersection dipping outward from the face creating an impossible situation to a wedge failure.

Table 03: Wedge failure parameters (50 +110 km to 50 +155 km)

Intersection	β	ε	$\emptyset$	φi
A	71	58	20	38

$$FOS = \frac{\sin(71)}{\sin(0.5 X 58)} X \frac{\tan(20)}{\tan(38)} = 0.91$$

The wedge is formed due to the intersection of two joint planes of N31W / 42^0 SW and EW / 84^0 N. The factor of safety is bellow 1, which implies an instability for wedge sliding. And also the trend of the intersection of N82W is existing with the same quadrant with dip direction of slope falls which can further increase the possibility of sliding.

4. CONCLUSIONS

The investigation reveals that the slope is intersected by well developed joints systems producing wedge failures with the proposed rock bench as explained above. Therefore, redesign of the proposed rock bench should be done in several locations to prevent sliding of the wedges by avoiding the undercut of the intersection with the slope face as follows.

50 + 000 km to 50 + 020 km
Proposed slope angle = 78^0
Stable slope angle = 65^0

50 + 110 km to 50 + 155 km
Proposed slope angle = 78^0
Stable slope angle = 70^0

At these new conditions, the intersection is overhanging with the rock face and retarded the wedge sliding. When it is difficult to change the bench angle rock bolts should be insert to prevent the expecting failures. These tensioned rock bolts should be with sufficient lengths that increase the inherent strength and holding the unstable discontinuities. At the locations where rock boulders are present (50 + 030 km) they should be first remove by fragmentation. If a weathered rock strata would be exposed even after the blasting in between the 50 + 040 km to 50 + 060 km, the erosion control measures should be installed. It is better to install proper surface drainage system of ditches/diversion drains in this weathered zone to control the dissipation of water into the rock mass. For the purpo seof drain out the surface run off easily without stagnating at the crest and without percolating in to the fractures it is essential to design the minor crest of the each rock bench to slightly dipping towards the rock face as figure 03

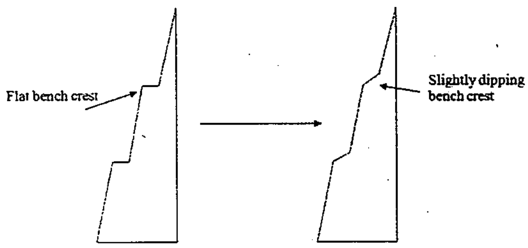


Fig 03: Proposed rock bench and way of it should be changed

5. ACKNOWLEDGEMENTS

Authors wish to express their sincere gratitude to Mr. R.M.S Bandara, Head, Landslide Research and risk Management Division, NBRO for his guidance and granting permission to this research, special thanks should go to Mr HML Indrathilake' Senior scientist, NBRO for his valuable contributing to the project works and to all staff members at NBRO for the their support to the project activities.

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SUITABILITY OF LOCALLY AVAILABLE EXPANSIVE SOIL FOR THE BASE LINERS IN WASTE CONTAINMENT FACILITIES

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Abstract

Due to open solid waste dumping, the water and soil pollution by leachate is now has become a major environmental problem in Sri Lanka. In the engineered landfills, bottom liners are used to prevent the migration of contaminant in to the ground water. Compared to the Geosynthetic clay liners (GCL) and High density polythene (HDPE), Compacted Clay liners are less expensive if materials are locally available. But according to the USEPA recommendations the minimum thickness of CCL liner is 1 m. So with the aim of reducing CCL thickness, expansive soil obtained from Digana, Moragahakanda and soil mixed with 5% and 10% sodium bentonite were tested for hydraulic conductivity. The both unamended soils obtained from Digana and Moragahakanda exhibited lower hydraulic conductivity than the maximum recommended hydraulic conductivity 1 x 10-7 cm/s. The mixing of soil with bentonite resulted much lower hydraulic conductivity than the unamended soil. When the soil mixed with bentonite, the percentage of immobile water within the sample increases, reducing the pores available for water flow. Whether the mixing of soil with bentonite resulted lower hydraulic conductivity than the unamended soil, there is a greater possibility to increase the hydraulic conductivity when exposed to leachate due to aggregation of clay particles.

Key words: Expansive soil, Bentonite, Hydraulic conductivity

1. INTRODUCTION

The bottom liners of the engineered landfill prevent the migration of leachate in to the ground water generated within the landfill. Whether the liner material such as Geosynthetic Clay Liners (GCL) and High Density Polythene (HDPE) are widely used over the world, under local conditions the construction cost of using these synthetic liner materials is exorbitant due to non-existence of local industry producing such materials. Compacted Clay Liners (CCL) are widely used as a liner for waste containment facilities , as the construction cost is less compared to the other liner systems, if the clays are locally available. Usually compacted clay liners of the landfills are constructed of native soil that contains sufficient amount of clay size particles. Instead of kaolinite rich clay, use of an expansive soil is more advantageous due to its very low hydraulic conductivity and its ability to retain more water due to its very high plasticity index and more importantly its self healing ability to seal off the cracks developed during an extremely long dry period. Characteristic expansive or swelling clays are highly plastic clays and they often contain colloidal clay minerals such as montmorillonite. Very small particle size, large internal surface area, diffuse negative layer charge allow montmorillonite clay mineral to absorb large amount of water. Therefore, if it is used to construct base liners, it can withstand longer dry periods without forming cracks.

2. MATERIALS AND METHODS

(2.1) Materials

Expansive soil samples were obtained from Digan and Moragahakanda to carry out soil classification, swelling pressure, 1-D consolidation and hydraulic conductivity tests. Also Moragahakanda soil mixed with 5% and 10% Sodium bentonite (Bentofix®, NAUE GmbH & Co. KG, Germany) were tested for hydraulic conductivity. At the first stage deaired water was used as the permeant liquid. Hereafter the soil obtained from Moragahakanda and Digana will be referred as soil M and soil D respectively.

(2.2) Methods

2.2.1 Soil classification

Initially regular soil classification tests such as liquid limit, plastic limit, specific gravity, and particle size distribution were carried out to identify the soil properties. These tests were carried out conforming to those specified by ASTM and British Standards. To identify the expansiveness of soil obtained from Moragahakanda and Digana, swell pressure test was carried out.

Table 1: Characteristics of tested soils

Test	*Bentonite*	*Soil D*	*Soil M*	*Soil M + 5% bentonite*	*Soil M + 10% bentonite*
Liquid limit/%	*600*	*50*	*44*	*49*	*70*
Plastic limit/%	*55*	*21*	*19*	*21*	*25*
Plasticity index/%	*545*	*29*	*25*	*28*	*50*
Particle density	*2.68*	*2.742*	*2.64*	–	–
Swell pressure/kPa	-	*41*	*126.1*	*267.7*	*357*
pH	*8.76 (1:10)*	*7.3 (1:5)*	*8.06(1:5)*	*8.20*	*8.34*
Electrical conductivity/(ms/cm)	*1.33*	*0.069*	*0.027*	*0.161*	*0.322*

2.2.2 Consolidation and Hydraulic Conductivity

In modern landfills, waste heights over 50 m are becoming more common considering the difficulty in finding suitable sites and high volume of waste generated. Under such loads the investigation of compressibility characteristics of the compacted clay liners also becomes important. In order to evaluate the consolidation and hydraulic conductivity characteristics under different consolidation pressures, the Rowe cell apparatus (Rowe et. al., 1966) having a 15 cm internal diameter was used. Rowe cell apparatus can be used to one dimensionally consolidate saturated soil, allowing drainage from both surfaces and then evaluate the hydraulic conductivity of the consolidated soil sample. In this consolidation cell, the total stress is applied by means of air pressure applied into a convoluted rubber membrane. Initially, the specimen was vertically consolidated up to a pressure of 50 kPa. Then, while keeping the applied consolidation pressure at the same value, the hydraulic conductivity of the consolidated sample was evaluated by allowing a steady seepage of water at a hydraulic head of 30, 50, 100,150 and 200 kPa. The above procedure of evaluating the hydraulic conductivity was repeated on the same sample after being reconsolidated under increased axial pressures.

3. RESULTS

3.1 Soil classification

The particle size distributions of tested soil are illustrated in figure 1. According to the results both clays are gravelly clay of intermediate plasticity. The soil classification results are shown in table 1. According to liquid limit and plasticity index based swell classification criterion both soils obtained from Digana and Moragahakanda exhibit medium swelling capacity.

3.2 Consolidation characteristics.

The relationship between void ratio and consolidation pressure are illustrated in figure 2. The variation of coefficient of consolidation (C_v) with the applied pressure was evaluated using the Taylor's square root of time method. The results are illustrated in Figure 3. For both soils obtained from Moragahakanda and Digana, it can be seen that the coefficient of consolidation increases with the increase of consolidation pressure. However, for the Moragahakanda soil mixed with 5% and 10% of bentonite, the coefficient of consolidation is considerably lower than the unamended soils and it decreases with the increase in pressure. This is in conformity with the observations of Robinson and Allam (1998).

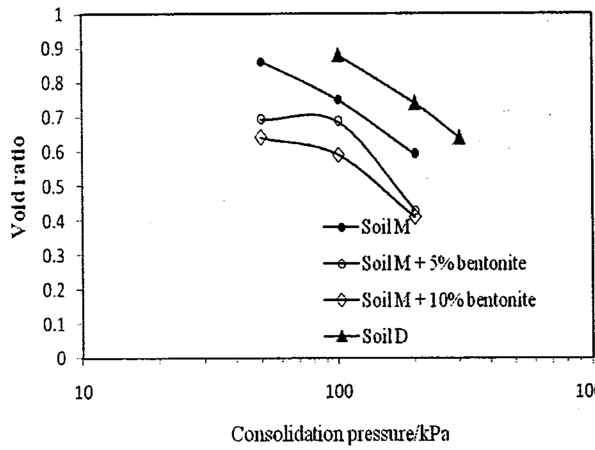

Figure 2: Variation of void ratio with the consolidation pressure.

3.3 Hydraulic Conductivity

With the increase of consolidation pressure, the hydraulic conductivity decreased for all four soils. The mixing of 10% bentonite with the soil obtained from Moragahakanda gave the lowest hydraulic conductivity among all four types of soil. The variation of hydraulic conductivity with consolidation pressure are shown in figure 4. Bentonite in the soil-bentonite mixtures hydrate and swell with the presence of water. Under fully saturated conditions, bentonite has very high swelling capacity and can be expected to fill the voids between the soil particles, reducing the pores available for water flow under hydraulic gradient.

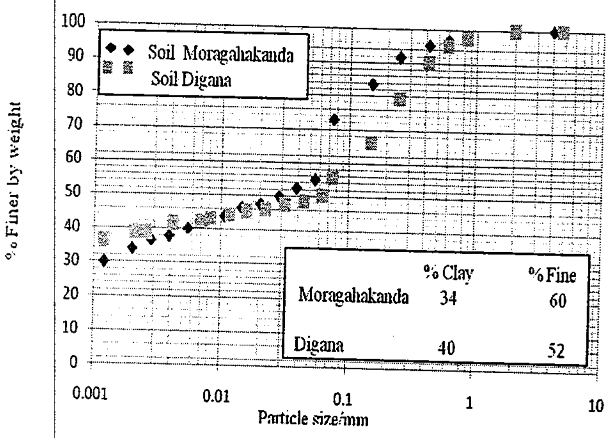

Figure 1: Particle size distribution of soil M and soil D

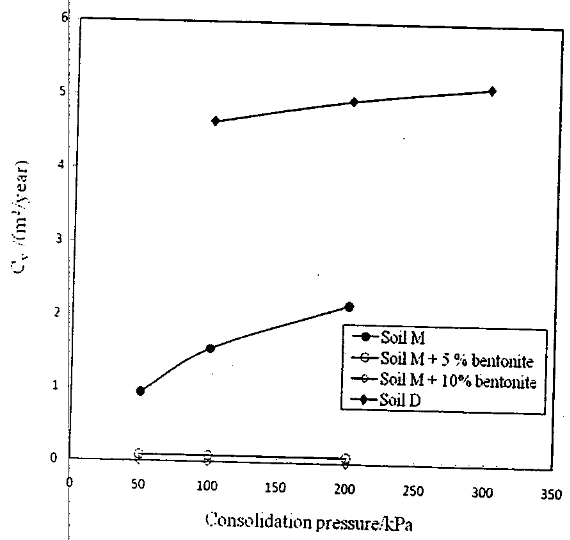

Figure 3: Variation of coefficient of consolidation with consolidation pressure.

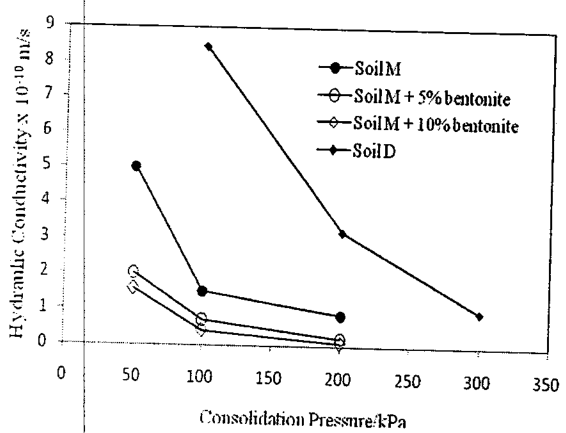

Figure 4: Variation of hydraulic conductivity with consolidaion pressure

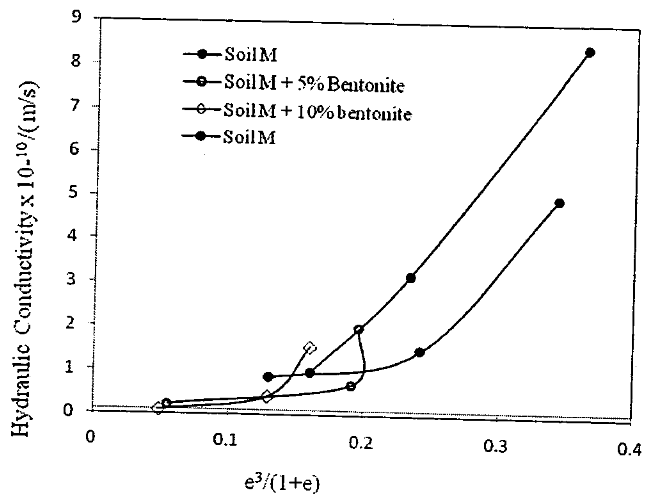

Figure 5: Variation of hydraulic conductivity with void ratio

The relationship between the hydraulic conductivity and the void ratio as given in eq.(1) is plotted in Figure 5.

$$K \propto \frac{e^3}{(1+e)} \qquad \text{Eq. (1)}$$

This shows good agreement of the variation of hydraulic conductivity with the void ratio in accordance with the Kozney Carman equation for both unamended soils obtained from Digana and Moragahakanda while less agreement on bentonite amended soil. (Won-Jin Cho, 2002) Considering 1 m hydraulic head on the liner which is constructed using clay having hydraulic conductivity of 1 x 10^{-9} cm/s with 1 m thickness, the rate of leakage through the liner was calculated using Darcy's law. The variation of rate of leakage with liner thickness for tested soils is illustrated in figure 6. Under same conditions, the tested soils have very low rate of leakage. Hence the liner thickness can be reduced from 1 m to much lower thickness.

4. CONCLUSIONS

Since both soil samples obtained from Digana and Moragahakanda exhibited lower hydraulic conductivity values than the maximum recommended, these soils can be satisfactorily used to construct compacted clay liners in municipal landfill sites. The mixing of soil M with bentonite resulted lower hydraulic conductivity than the unamended soil. With the increase of percentage bentonite added in to the soil, the hydraulic conductivity decreased.

The hydraulic conductivity of a soil depends on both mechanical and physic-chemical variables. The water within the pores of soil can be categorized in to two groups: mobile water and immobile water. Water within the interlayer of clay is considered as immobile and these pores

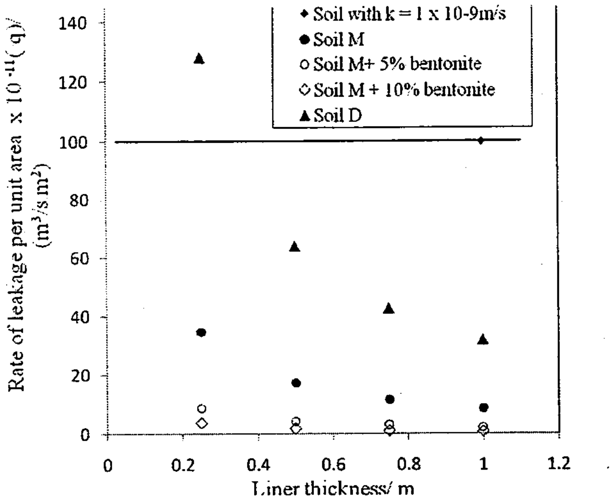

Figure 6: Variation of rate of leakage through the liner with liner thickness.

are not contributed for the water flow. When the soil mixed with bentonite, the percentage of immobile water within the sample increases, reducing the pores available for water flow. Whether the mixing of soil with bentonite resulted lower hydraulic conductivity than the unamended soil, there is a greater possibility to increase the hydraulic conductivity when exposed to leachate due to aggregation of clay particles. The main clay mineral in expansive soil and bentonite is montmorrilonite, in which have weak interlayer bonding. Due to this weak interlayer bonding water or any other liquid can easily go through the layers. Depending on the amount of exchangeable cations in the clay surface, swelling or shrinkage may result when exposed to leachate.

Considering the rate of leakage through the liner it can be concluded that by using the all four types of soils, the liner thickness can be reduced to 0.5 m. By using bentonite mixed soil, the liner thickness can further reduced.

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Mitigating Distress in Lightly Loaded Structures in Matale District

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ABSTRACT

Development of cracks on walls and foundations of lightly loaded structures in the Matale district is a great concern to its residents. This is attributed to the presence of karstic topography in Matale district that has been subjected to high grade metamorphism during pan African orogeny at ca.550 Ma. NBRO as the main government organization carrying out studies on landslides and subsidence has developed a hazard map for the district identifying areas that should not be considered for human settlement and major development activities. In this study, 75 houses from three Grama Niladari Divisions in the Matale district namely, Imbulandanda, Puwakpitiya and Dorakumbura were selected to conduct a field survey on the severity of cracks and collect information on the structural soundness of the buildings categorized as "modern", "Semi-modern" and "old" (poor) and their GPS locations. The severity of the cracks was categorized as "negligible" and "significant". Subsurface geological condition of the lightly loaded structures was obtained using geological maps and the GPS coordinates of the structures. The severity of the cracks was categorized as negligible for the cases where there are no cracks or slight cracks (< 2mm) and significant for the cases where there are moderate to severe cracks (> 2 mm). The type of construction was categorized as modern, semi-modern and old depending on the structural soundness of the construction. The analysis of the data show that 82% of the houses that are structurally sound had negligible cracks though they are lying on the limestone belt. However, out of the old type of houses located away from the limestone belt, 54% of the houses developed significant cracks. Based on the hazard map of NBRO, it is also revealed that 83% of the modern type of houses built on the hazard area developed negligible cracks while 77% of the old type of houses built on non-hazard area developed significant cracks. Therefore, it can be concluded that the influence of unfavourable geological conditions can be mitigated by adopting structurally sound designs in the construction of lightly loaded buildings and that the hazard maps developed by the NBRO can also be used to allow constructions in hazard areas that are properly designed and constructed.

Introduction

The study area of Matale District is located in the Central Province of Sri Lanka. Matale area has been subjected to high grade metamorphism during pan African orogeny at ca.550 Ma (Pitawala A. et al., 2008, Fernando et.al., 2011). The study areas selected are characterized by having relatively narrow belts of marbles, garnet-silimanite gneisses, charnockitic gneisses and quartzite as the uppermost rock Three Grama Niladhari Divisions in the District were selected for the survey of cracks developed in walls of the lightly loaded structures with the objectives of its relationship to the underlying geology and the type of construction and to identify a suitable construction methodology for its mitigation.

Methodology

Study Area

The Matale District has eleven Divisional Secretariats (Fig.1). Three Grama Niladari Divisions, out of 52 in the Matale Divisional Secretariat are selected as the study area where severe cracks had appeared on walls of houses.

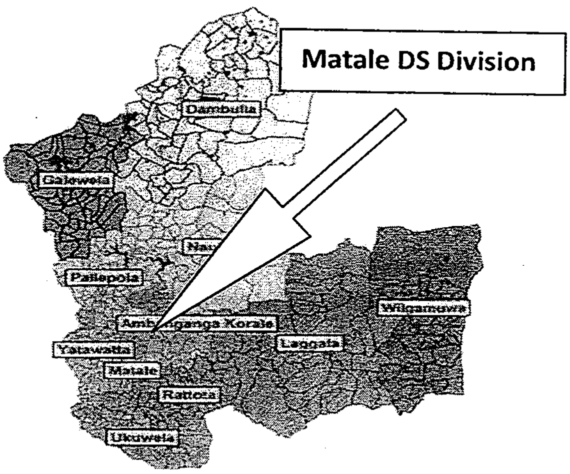

Fig.1 Matale District Divisional Secretariats and the study area

Seventy five Lightly loaded houses in the three GNDs, namely, Imbulandanda, Puwakpitiya and Dorakumbura were selected for this study.

Data Collection

During the field visits, data on the house as well as the site, such as GPS coordinates, type of crack and width, type of construction, materials used and in addition, information on the structural system of the building and geological features in the vicinity were collected.

Data Analysis

The magnitude of the cracks developed in houses was categorized as "negligible" for those having no cracks or cracks having less than 2 mm wide and "significant" for those having cracks of more than 2 mm in width.

Lime stone

Fig.2 Locations of all 75 houses surveyed in the Imbulandanda, Puwakpitiya and Dorakumbura GN Divisions on the Geology map

The construction type of the houses surveyed were categorized into three groups; Old, Semi-modern and Modern. The houses in which low quality materials have been used for the construction with poor workmanship or in a dilapidated condition due to poor maintenance, or non-usage of masonry foundation and walls that have been plastered with clay putty were categorized as "Old" type of construction. Houses where use low quality materials with evidence of poor workmanship in construction, however, having used Random Rubble Masonry for the foundation and cement/lime for the wall plastering were categorized as "Semi-modern". A house was categorized as "Modern", if it has used high quality construction material with evidence of good workmanship, constructed using appropriate technology, reinforced concrete framed structure, brick/block walls with lime/cement plastering.

The GPS locations of each house visited was plotted on the Kandy geology map produced by the Geological Survey and Mines Bureau to ascertain the type of underlying rock (Fig.2). Therefore, it is possible to categorize the houses based on whether the underlying bedrock is limestone or not.

Results

The above analysis shows that out of the 75 houses surveyed, 53% of the houses (40 houses) were on the limestone layer (Fig.3).

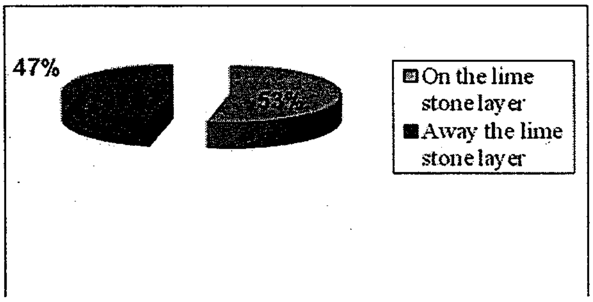

Fig.3 Number of houses located on the limestone layer and away from it.

A summary of field survey of the distribution of the extent of the development of cracks in relation to the type of construction of the houses is given in Table 1.

The variation of the development of significant cracks and the type of construction for the houses constructed on limestone and away from it is shown in Fig.4.

Table 1. Summary of results of the field survey

Underlying Rock type	Construction type	Category of Cracks	
		Significant (%)	Negligible (%)
Limestone	Modern	9	91
Limestone	Semi-modern	67	33
Limestone	Old	86	14
Other than Limestone	Modern	8	92
Other than Limestone	Semi-modern	27	73
Other than Limestone	Old	86	14

It is clearly seen that the development of significant cracks is at a very high percentage value for "old" or poor type of constructions especially when it is on limestone. However, it can also be seen that the development of significant cracks can be reduced if modern construction methods are used. This observation holds true irrespective of the type of underlying rock type.

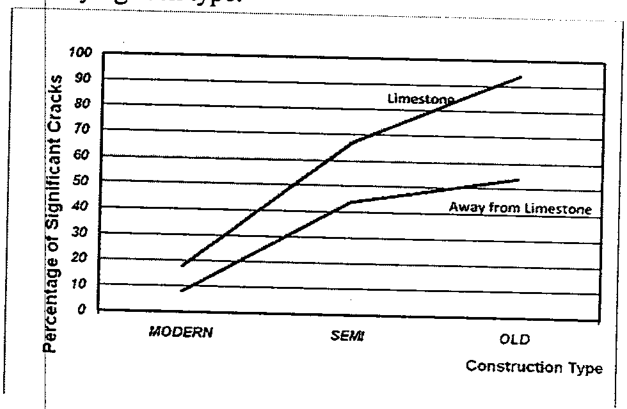

Fig.4 Variation of development of significant cracks and the type of construction for houses constructed on limestone and away from it.

Comparison the results with those of previous NBRO study

A study had been carried out by NBRO in Matale District in 2006 based on 2882 houses in 24 GN Divisions which also included Puwakpitiya, Imbulandanda and Dorakumbura GN divisions. Based on this study NBRO had prepared a map in which a hazard and non-hazard areas have been identified. Therefore, in this comparative study, the 75 houses in the 3 GN divisions were located in the map produced by the NBRO as given in Figs.5, 6 and 7 for Dorakumbura, Imbulandanda and Puwakpitiya GN divisions respectively. Table 2 gives the comparison of the results of this study with that of NBRO.

It can be seen that 83% of the houses constructed using modern construction techniques developed only "negligible" cracks though the locations of the houses had been classified as "hazard" in the hazard map of NBRO whereas, 77% of the houses constructed using "old" or poor construction methods in "non-hazard" areas developed "significant" cracks.

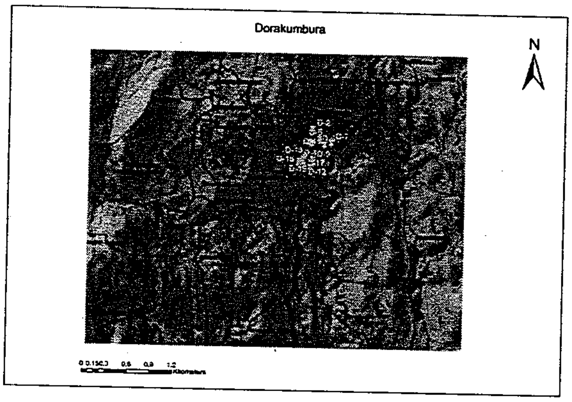

Fig.5 Locations of the houses in Dorakumbura GN division in the Hazard map of NBRO

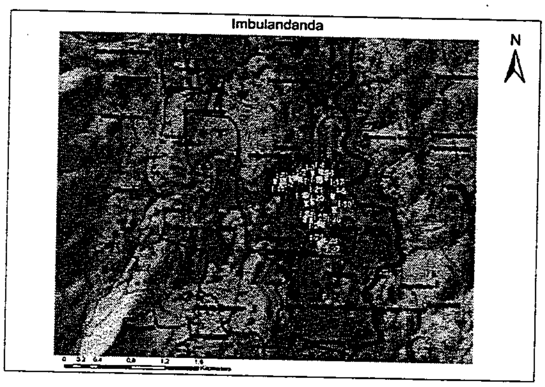

Fig.6 Locations of the houses in Imbulandanda GN division in the Hazard map of NBRO.

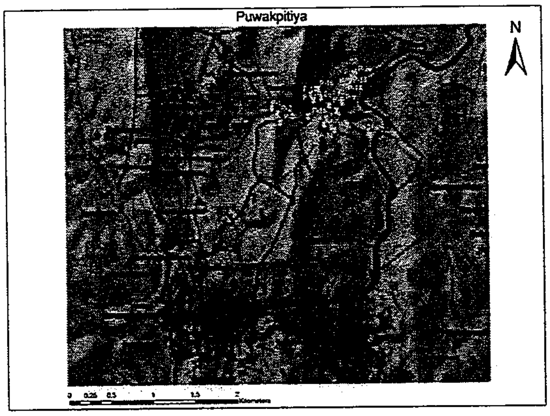

Fig.7 Locations of the houses in Puwakpitiya GN division in the Hazard map of NBRO

Table 2. Comparison of the severity of cracks and the hazard status based on NBRO map

Type of house	Location of the house in NBRO map	Total No. of houses	Severity of cracks	No. of houses	% of houses
Modern	Hazard area	18	Negligible	15	83%
			Significant	3	17%
Old	None hazard area	13	Negligible	3	23%
			Significant	10	77%

Conclusions

Based on the study carried out on the houses in three GN divisions of Matale Divisional Secretariat, the following conclusions are made.

(1) The development of cracks is influenced not only by the unfavourable geological conditions but also by the quality of construction and structural integrity of the buildings.

(2) The influence of unfavourable karstic topography in Matale district on the development of cracks in buildings can be mitigated by using proper construction techniques and sound structural system.

(3) The severity of the cracks of houses built in hazard areas as classified by NBRO can be reduced if modern techniques of construction and sound structural system are used.

(4) The hazard map of NBRO can be used to allow construction in hazard area only if the proposed structure is to be properly designed and constructed.

Acknowledgements

The support extended by Mrs.Thejani Thilakaratne, Assistant District Secretary, Matale and Mr.Indika Ranaweera, Assistant Director of Disaster Management Centre, Matale in the process of field surveys is gratefully acknowledged.

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Vertical displacement and stress behavior below the strip foundation on underground cavity

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ABSTRACT: Simulation of the behavior of foundations over underground cavities and identification of factors affecting the design of the foundations over such cavities are valuable to foundation design engineers. Matale area in the central province of Sri Lanka provided the background for this work, where many problematic subsurface foundations are giving rise to distress in building. The various factors and parameters used in this work where based on actual data obtain previously by studies conducted in the Matale area by NBRO. The objective of the study is to investigate numerically the interaction between strip footing and circular unlined underground cavities. Particular emphasize is placed on the vertical displacement, stress distribution and extent of influence zone.

1 INTRODUCTION

Managing difficult ground could effectively reduce uncertainties and hence minimize risks.

Weak foundation soils have always been a challenge to Geotechnical Engineers. When designing foundations over underground cavities, bearing capacities, vertical movements, and deferential settlements are some of the major concerns. Structural damage due to cavities under buildings has been a major problem in geotechnical engineering. Ground subsidence may occur due to some internal change within the subsurface such as the extraction of fluids or solids, solution of rock or a cementing agent in soils, erosion, or physiochemical changes.

In accordance with field studies and experience, several empirical formulae have been developed to predict the settlement of foundations over cavities such as Zoning method used in Europe and Dimensionless empirical method used in Britain. These empirical equations do not take into account all the factors such as type of the foundation, soil type and characteristics, and cavity geometry, which would improve certain limitations on the accuracy of these empirical methods.

In the world many countries are faced with problems of ground subsidence, and Sri Lanka is among them. Matale area in Sri Lanka contains ample evidence for cavities that may be present under existing foundation or at potential construction sites.

Therefore, this paper addresses the simulation of the behavior of foundations over underground cavities and identifies factors effecting the design of the foundations over such cavities. The interaction between footing and cavity is analyzed using a commercial Finite Element software program named Plaxis (Brinkgreve, 2002).

Very few studies on foundation vs. cavity interaction are available, and some work had been done on the effect of underground cavities on the performance of overlying footings. In some instances, soluble bedrock dissolved at the interface of the soil and bedrock leave void spaces. Estimation of ultimate bearing capacities of shallow foundations constructed over these voids, as well as stability of such structures are becoming important issues. Only a few studies have been reported in published literature so far.

Baus and Wang [Das, 2007] have reported some experimental results for the ultimate bearing capacity of shallow rough continuous foundations located above the voids. The laboratory model test of Baus and Wang [Das, 2007] were conducted with a soil having the specific properties.

The effect of underground cavity on the interaction between two closely spaced strip footings was investigated using a finite element computer program by Wang et al., (1994).

Interaction between strip footings and circular shallow soft ground tunnels were investigated by Badie and Wang (1994).

2 NUMERICAL MODELING

The numerical study carried out here confirmed the generally excepted behavior of a footing placed on ground with a cavity. A parametric study was carried out using finite element analysis, which yielded more specific quantitative data on the interaction between a footing foundation, and ground with cavities.

The variation factors and parameters used in this work where based on actual data obtained previously by studies conducted in the Matale area by NBRO (National Building Research Origination). These studies had yielded the profile depth and size of cavities, Young's Modulus and Poisson's Ratio of the subsurface.

Considering the type of the sub soil at the Matale area the range of Poison's Ratio was determined as 0.3 ~ 0.45 for saturated clayey soil and as 0.1 ~ 0.3 for rock/ sand/gravely sand.

The finite element analysis was conducted by assuming linear elastic properties for all soil/rock material. Analysis were made under a uniform surface load of 100 kPa (applied on the top of the footing), and the parametric study was conducted by varying the Width of the Foundations (B), Depth to the cavity from bottom of the footing (H), Young's modulus (E), Cavity radius (R),Poisson ratio (v). In this analysis the above parameters are identified in the legend B-H-E-R-v.

The cavity and footing were located at a minimum distance of 20m (from the domain boundaries to minimize boundary effects.

Stress Distribution and Settlement Analysis

First the case of footings of different widths located on continuous soil media (no cavities) was considered. Widths of foundation considered were 1m, 2.5m, 5m, 10m and 15m. Vertical normal stress distribution along the centre plane of the footing without cavity is shown in the Fig 01. The shape of the curve is expected to follow that of Boussinesq distribution. All curves start from the 100kN/m^2 at the ground surface, the vertical stress along the cavity centre line decrease when the footing width is increased, the vertical normal stress at given depth increases, but the basic shape of the curve remain the same.

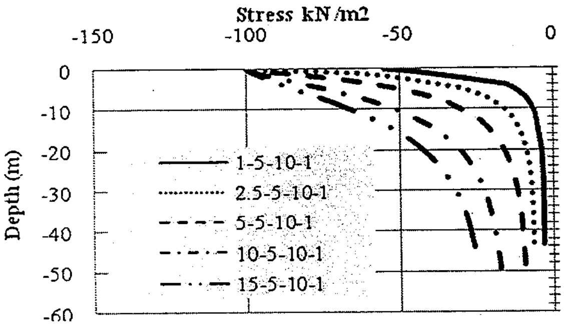

Fig 01 Vertical stress distribution of centre plane of the footing without cavity

Variation of stress and settlement with width of footing

The vertical stress distribution (VSD) and extreme vertical settlement (EVS) along the vertical plane were analyzed with different values of B.

The vertical stress distribution analysis (VSD) along the vertical plane of footing centre above single cavity is presented in Fig 02 for H =10m. Extreme vertical settlements (EVS) along the vertical plane of centre of the footing above single cavity for different values of footing width (B) are shown in Fig 03 for without cavity.

It is seen that vertical settlement increases with increasing footing width (B). The EVS different with and without cavity (ΔS) is presented in Fig 04. Considering the Fig 04, ΔS also increases with footing width. Therefore small footing widths exert very low influence on the cavity.

VSD vs. radius of single cavity (R)

The configuration of a 15m wide footing located 10m above single cavity of radius (R), the vertical stress distribution analysis (VSD) along the vertical plane of footing centre shown in

Fig 5 for B = 15m. This analysis shown that below the bottom of the cavity the maximum stress point and shape of the stress distribution consequently change with the cavity radius.

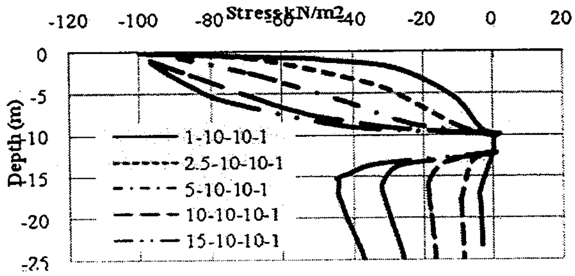

Fig 02 The VSD along the vertical plane of footing centre for different footing width (B = 1, 2.5,5,10 & 15) for H= 10m.

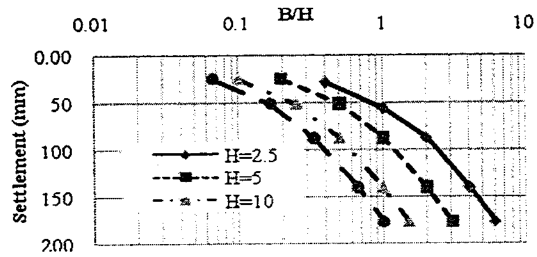

Fig 03 EVS vs. B/H without cavity

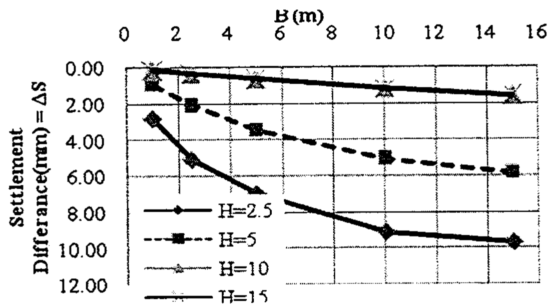

Fig 04 Settlement Difference (ΔS) of with and without cavity

The maximum stress point at the bottom of the cavity is decreased from about 45kPa to 12kPa when the cavity radius is increased from 1m to 10m.

The ground settlement trough for difference cavity radius (R) above single cavity due the footing stress is shown in Fig 6 for B = 15m. It is seen that as the cavity radius increases from 1m to 10m, the extreme vertical settlement increases from 51mm to 100mm for 15m footing width.

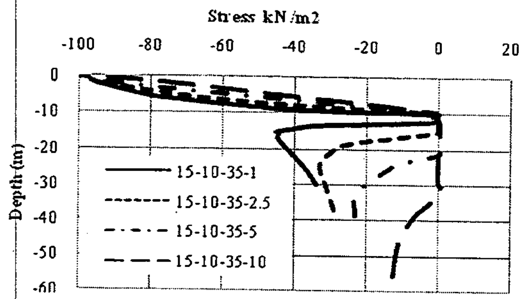

Fig 5 - The vertical stress distribution analysis along the vertical plane of footing centre for different radius of single cavity (R = 1, 2.5, 5, 10)

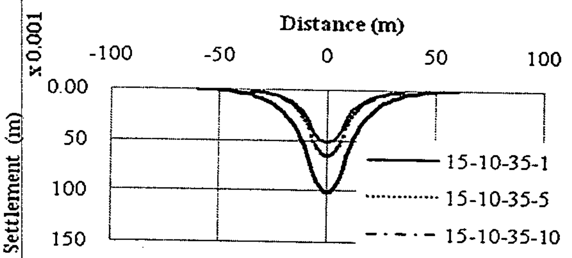

Fig 6 The ground settlement though for different radius of single cavity (R = 1, 5, & 10).

Stress and settlement vs. depths of cavity (H)

The configurations for stress and settlement analysis by different depths of cavity (H) are shown in Fig 7.

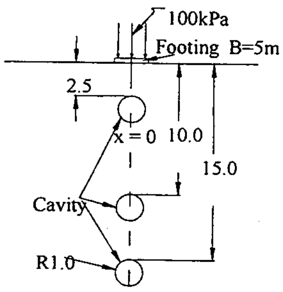

Fig 7 The subsurface geometry for stress and settlement analysis for different depths of cavity (H)

The vertical stress distribution with depths to the different cavity are showing in Fig 8 for B=15m.

The extreme vertical settlement for 5m wide footing (Fig 7) is shown in Fig 9. In this analysis revealed when the depths to the cavity (H) increase from 2.5m to 10m, the settlement decrease about 6.8mm to 6.2mm for 2.5m wide footing and 100mm to 92mm for 15m wide footing.

Stress and settlement analysis vs. value of E & v

The Young's modulus (E) and Poison's ratio (v) directly influence the stress distribution and settlement in liner elastic material. Thus, stress and settlement behavior under footing above single cavity with different value of elastic properties of subsoil are useful for foundation design engineers, as foundation design and analysis is mainly based on equivalent elastic properties of subsoil.

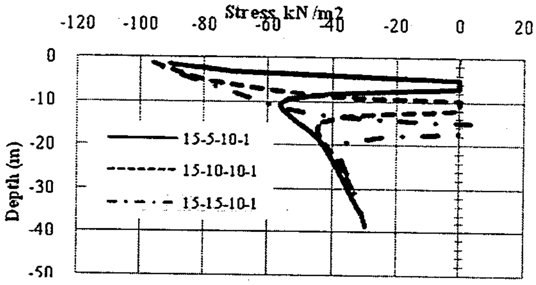

Fig 8 The vertical stress distribution analysis along the vertical plane of footing centre for different depth to the cavity (H=5, 10, 15) for B = 15m.

The Fig 11, shown that the VSD for different values of Young's modules graphs are same because of it is dependent of the value of Young's Modulus (E). when Poisson's ratios of foundation soil change in 0.1 to 0.45, the maximum vertical stress below the bottom of cavity change from about 52kPa to 61kPa (Fig 12)

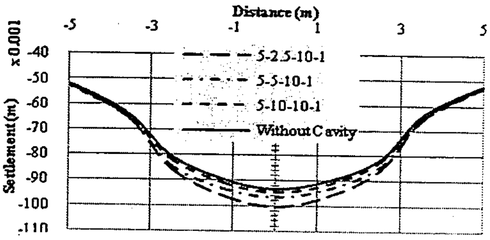

Fig 9 The settlement for 5m width of footing for different depth to the cavity (H=2.5, 5, 10)

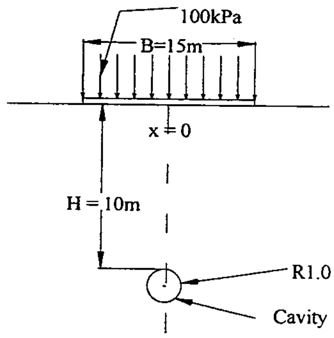

Fig 10 The different values of Young's Modulus (E) vs. VSD

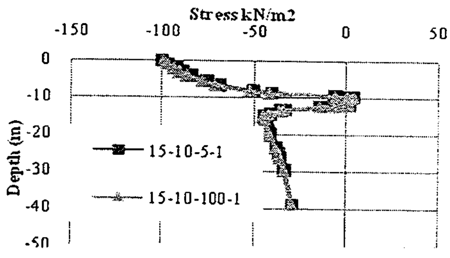

Fig 11 The different values of Young's Modulus (E) vs. VSD

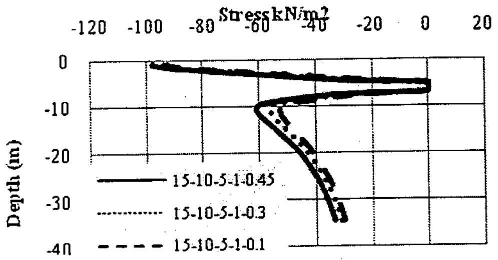

Fig 12 The different values of Poisson's Ratios (v) vs. VSD

The VSD with different values of Young's modules and Poison's ratios were analyzed using the configuration shown in Fig 10.

Extreme vertical settlement (EVS) along the vertical plane of centre of the footing above with and without single cavity for different values of Young's Modulus (E) and Poisson's ratio (v) are shown in Fig 13 and Fig 14.

The settlement difference with and without cavity (ΔS) is in the secondary vertical axis of graphs in Fig 13 and Fig 14. The influence of the cavity is directly indicated by ΔS. It is seen that when the E value changes from 5MPa to15MPa the ΔS decreases from 5.86mm to 1.85mm and the E value increases 15MPa to 35MPa the ΔS decreases from 1.85mm to 0.8mm. As would be expected, for low values of E, the extreme vertical settlement of the centre of the footing above single cavity is much greater than in the case without cavity.

When the Poison's ratio change from 0.1 to 0.45, the EVS value changes from 136mm to 201mm for the case with cavity and from 195mm to 130mm for the case without cavity. However when the Poisson's ratio increases from 0.1 to 0.45, the ΔS decreases from 5.95mm to 5.72mm. Therefore the Poisson's Ratio influences the settlement in ground with cavity.

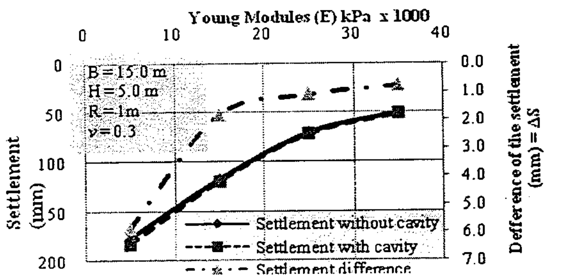

Fig 13 Extreme settlement values vs. E

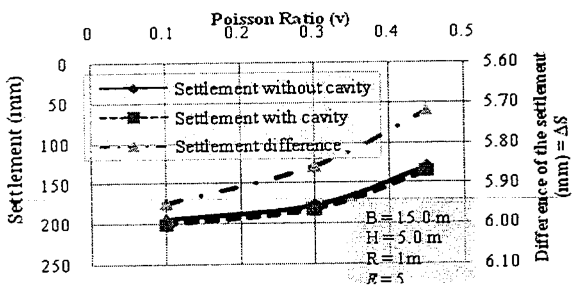

Fig 14 Extreme settlement values vs. *v*

3 CONCLUSION

The numerical study carried out here conformed the generally excepted behavior of a footing placed on ground with a cavity. A parametric study carried out using finite element analysis, which yields more specific quantitative data on the interaction between a footing foundation, and ground with cavities.

The presence of the cavity modifies the stress distribution in the soil medium in the vertical planes varying through the cavity, as well as outside the cavity but in its vicinity. High vertical normal stresses exist at the extreme edge of the cavity, on the horizontal plane through the center of the cavity.

The distribution of vertical normal stress underneath the footing is influence by the depth of the cavity (Fig 02).

Footing of the layer width shows a greater settlement at the centre than footing of smaller width, with or without the presence of a cavity underneath (Fig 03 and Fig 04).

Also it shows that the grater the depth of location of cavity, the lesser is its influence on the footing settlement. For example, the increase in settlement due to the presence of a cavity underneath as compared to the case of no cavity underneath shows no difference for depths of cavity exceeding about 10m.

It is found that stiffer the soil (higher the Young's Modulus) the lesser is the settlement of footing, with or without cavity (Fig 13). Also higher the Poison's Ratio, the lesser is the settlement (Fig 14)

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GEOGRID REINFORCED PILE SUPPORTED EMBANKMENT FOR BRIDGE APPROACHES

By Wedikkarage. W.S.N.M.

Abstract: Southern Transport Development Project (STDP) in Sri Lanka is the second expressway project which is connecting southern part of the island to the main city of Colombo. Almost *18 km* s of first *34 km* s are resting on the soft soil deposits. Some of the bridge approaches also lies in these soft ground area. To eliminate the settlement and differential settlements it was proposed to have a geogrids reinforced pile supported embankment (GRPE). Finite element method was used to estimate the settlements in GRPE. It was evident that the specified settlement criterion can be achieved by the application of GRPE.

Key words; Soft soil, Embankment, Bridge Approach, Geogrid reinforced pile supported embankment (GRPE), Finite element analysis.

INTRODUCTION

There are few shortcomings due to the settlement phenomenon at bridge approaches. The vehicles moving along this section would feel discomfort caused by sudden upward leap at the bridge approach. Then the regular topping up or surfacing would be needed. This is a very common problem on highway pavements.

Southern Transport Development Project (STDP) in Sri Lanka is the second expressway project which is connecting southern part of the island to the main city of Colombo. Almost *18 km* s of first *34 km s are resting on the soft soil deposits. The highway trace* stretches along the Kalu Ganga river flood plain. There are 18 river bridge approaches in STDP and four of them are on 6 m or more thick soft soil deposits. The location map is in Figure 1.

Highway bridges constructed over rivers, usually located rested on piled foundations due to the presence of soft alluvial deposits in flood plain. The piled foundations are designed to eliminate almost all the settlement or settlement to be in a range of *10-20 mm*.

In STDP (Kottawa to Dodangoda section), the soft soil deposits less than *5 m* depth at bridge approaches are replaced using rock boulders. For more than 5 m depths, For others Geogrid Reinforced Piled Embankment (GRPE) was selected as the best option.

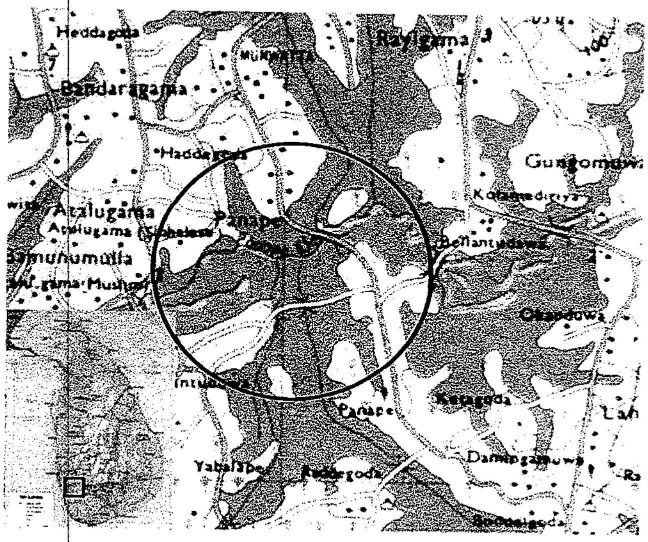

Figure 1 –Location map

SUBSOIL CONDITION AT STDP

In this paper, Panape Ela bridge approach design using a GRPE system *is considered. The maximum height of the embankment in* Panape area is about 7m s and the sides slopes are designed to have 1:1.5, vertical to horizontal.

Two borehole investigations were carried out. Undisturbed samples were also collected and were subjected to laboratory tests.

Consolidation tests, triaxial tests and other index tests were carried out. Summary of test results are given in Table 1.

A lateritic fill was already placed at the site and the thickness of the lateritic fill varies from 1 m to 2 m at two borehole locations. Below the lateritic fill, the original ground surface was found and investigations revealed that the sub soil is composed of soft organic clay layer followed by very soft peat layer with some decayed wood pieces. Total thickness of the above organic clay and peat layers is around 6.5 m and these compressible layers are underlain by layers of sand, about 3 m in thickness.

At a depth of 11 m from the original ground surface, partly weathered hard rock is encountered. The SPT values in organic clay and the peat layer is typically zero. The recorded high N values such as 4, 6 at some depths of peat layer might be due to the presence of undecayed timber particles.

Idealized subsoil profile in longitudinal direction of the expressway is presented in Figure 2.

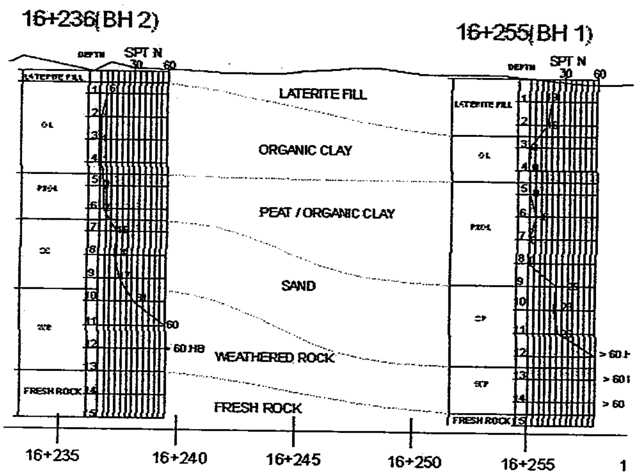

Figure 2- Idealized subsoil profile

Table 1-Sumary of laboratory tests

Borehole location	Consolidation test			Triaxial test		
	Depth /m	C_c	σ'_c	Depth /m	c'	ϕ'
16+235	4.0~4.6	1.8	65	-	-	-
16+236	5.5~6.1	1.24	68	3.5~4.1	8.85	25.4

MAIN COMPONENTS AND MECHANISMS OF THE GRPE

As in name "GRPE", it consists of piles, pile caps and load transfer platform (LTP) which is reinforced by geosynthetics (geogrids/geotextiles). The load from the embankment, road pavement, and vehicles are transferred to piles through the LTP and piles transfer *that to hard bearing stratum bypassing the soft soil deposits.*

The mechanism of transferring load to the piles in this system is arching action of the soil in LTP. Arching action is further enhanced by the geosynthetics in the LTP. The arching action is effectively mobilized in the granular soils and hence it is very important to use a granular material in the LTP.

DESIGN CONSIDERATIONS

The main design criterion is to limit the settlement near bridge approach. It is intended to limit the settlement less than 20mm near the bridge and less than 150mm at 20m away from the bridge. These limits are specified considering rider comfortability. Long etal (1998) classified the approach embankment –bridge structure interface movements qualitatively as in Table 2 and the approximated differential settlements related to each rating are also given in the same table.

The pile spacing, load transfer platform thickness and required tensile strength of the geogrids/geotextiles should be determined to comply with the design requirements.

PRELIMINARY DESIGN

Arrangement of piles

There are wide ranges of piles, which can be used in a GRPE system. Gravel piles, precast concrete piles, lime-cement mixing piles

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are few of those. These pile types should be selected considering the factors such as availability, cost, construction methodology, subsoil conditions and embankment height etc. Pile arrangement will be based on required settlement profile other than aforementioned facts.

400×400 mm² square precast concrete piles were used in two different spacing, 2.5m and 3m in Panape GRPE system.

Table 2- Classification of approach/bridge interface description (Long etal, 1998)

Qualitative visual rating	Approach/ Bridge Interface Description	Approximate differential settlement/(mm)
0	No bump	0
1	Slight bump	25
2	Moderate bump, readily recognizable	50
3	Significance bump, required repair	75
4	Large bump, safety hazard	>75

Design of load transfer platform

As discussed in the above section, the LTP transfers the load to piles. It is assumed that this reinforced LTP acts as beam, which transfers the load to piles (Colin 2004). Based on that assumption, following criterions was considered in the design of LTP.

- A minimum of three layers of reinforcements is used to create the platform
- Spacing between layers of reinforcement is 200- 450 mm
- Platform thickness equal to one half the clear span between piles
- Soil arch is fully developed within the depth of the platform

Therefore, the thickness of the LTP was selected as 1.2m, which is tentatively half of the pile spacing.. A granular material is compulsory to be in the LTP and hence dense graded aggregate base course (ABC) material was planned used for LTP.

Length of the treatment from the bridge

The specified differential settlement criterion is maximum 0.6% gradient to a minimum length of 6m. As discussed in previous sections, it was intended to use GRPE for average length 25m from the bridge, which will comply with the specified limits. The last 10m of the GRPE considered as a de-skewing section due the skewness bridge abutment.

Estimation of tensile strength of geosynthetics material

There are few methods developed to determine the tensile strength of geogrids. *The method proposed in BS 8006 is widely used.* There are few other methods also, namely Terzaghi method, Hewlett and Randolph method, etc.

Han (1999) proposed a method based on case studies related to GRPE and the information in Figure 3 is developed based on that information. Figure 3 was used to estimate the tensile strength of the geogrid material.

As in the Figure 3, if there is no contribution from foundation soil and for 6.5 m embankment, tension in reinforcement will be 600 kN/m. With bulk safety factor of three considering the damage during installation, long term creep, chemical degradation and biological degradation, ultimate tensile strength of geogrids was taken as 1800 kN/m. This much of capacity is very difficult to achieve with one geogrid layer. Therefore, 3 numbers of geogrids with 600 kN/m ultimate tensile strength were embedded in the LTP at 300mm vertical spacing.

DETAILED ANALYSIS

The current problem is mainly related to deformation of soft soil under a very complex load transferring system and even more complex geometry due to the skewness of the bridge approach. There is very few or almost no any closed form solution to estimate deformations of this kind of a problem.

The application of three dimensional finite element (FE) analyses would be ideal for the analysis of deformations. However, the analysis would be required very powerful computers and longer time. Even if it is used, the results may not be differ very much when compare with a two dimensional model. Therefore carrying out a two dimensional FE Analysis is worth and software package called PLAXIS 8.2 2D was used for the analysis.

MODELLING THE PROBLEM IN PLAXIS

Foundation soils

Soft peaty clay and organic clay was modeled using soft soil model which readily available in the software program. It is based on the Cam-Clay theory and required parameters other than shear strength parameter (cohesion, c' friction angle (ϕ') and dilatancy angle (φ)) are modified compression index (λ^*), modified recompression index (κ^*) and initial void ratio, e_0. The relationship of λ^* and κ^* with laboratory consolidation test results are as follows.

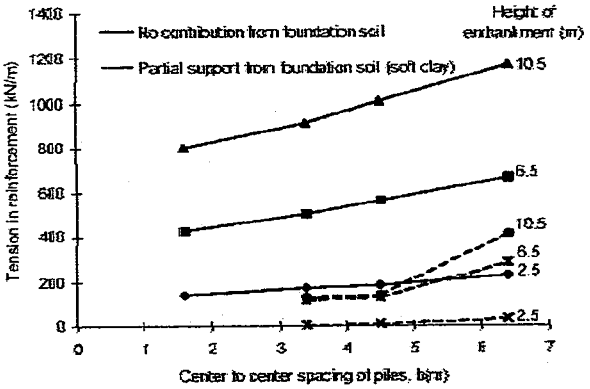


Figure 3- Variation of tensile strength of the reinforcement with other parameters

$$\lambda^* = \frac{C_c}{2.3(1+e_0)} \qquad \kappa^* = \frac{2\,C_r}{2.3(1+e_0)}$$

Clayey sand, bedrock, embankment and gravel mat was modeled assuming linear elastic-perfectly plastic behavior which was modeled using Mohr Coulomb model. Relevant soil parameters are tabulated in Table 2.

Piles and geogrids

Piles were modeled as plate element and geogrids were modeled using geogrid element in the software program. Relevant material parameters are tabulated in Table 3 and 4 respectively.

Construction sequence

PLAXIS has the capacity of analysis the problem by stage loading. This facility was used to simulate the embankment filling in layers. The analyzed stages are as follows.

- Initial condition: It is assumed that piles were installed in the unimproved ground and gravel mat with geogrids (load transfer platform) was constructed on it.
- Stage 1 to Stage 4: Application of embankment and pavement load
- Stage 5 to Step 6: Simulation of Road open to traffic

Analysis 1- settlement of the embankment without any treatment

FE model was developed considering the normal embankment construction without any ground improvement i.e. Consolidation settlement of the soft soils due to embankment loading. The total settlement after the application of full load was 3.6m s. Karunarathna (2007) developed some curves for the variation of consolidation settlement of peat/peaty clay deposits with the embankment height and thickness of said soft soil deposit as in Figure 4. From that curve also, same settlement can be obtained and hence it can be considered as the validation of the developed FE model. Figure 5 presents deformation contours for the analysis 1.

Analysis 2- analysis of GRPE

The schematic diagram of the GRPE system, which was analyzed, is presented in Figure 6. The main components of GRPE and relevant material properties are given in that Figure.

Figure 7 and Figure 8 gives the analysis results of GRPE for different pile spacing, namely 2.5m and 3.0m respectively. Pile spacing of 2.5 m represents the area close to the bridge and 3.0 m spacing away from the bridge. As mentioned in the above, stage construction procedure was used in the analysis in order to simulate actual construction process.

Results of the analysis

The predicted maximum settlement is about 20 mm near the bridge and 31 mm away from the bridge. Settlement after three years after completing the construction of GRPE can be estimated using stage construction option in PLAXIS. Accordingly, after the construction of the embankment, only 4 mm of settlement will occur in three years time near the bridge approach and only 7mm away from the bridge approach. Table 5 gives the predicted settlement after completion of each construction phases.

Differential settlement

Differential settlement is defined as the difference in the settlement at the center of the pile and at the mid span of the pile spacing. The observed differential settlement at the embankment top is almost zero. Differential settlement at pile top level is about 10 mm and 14 mm for 2.5m and 3.0 m pile spacing respectively. The differential settlement within the embankment is quite alright then.

Table 2 – Material parameters of soils for FE analysis

Parameter	Symbol	Unit	Peat /Organic Clay	Clayey Sand	WR /fresh rock	Embankment fill	LTP
Saturated unit weight	γ_{sat}	kN/m³	10.3	16	22	20	20
Dry unit Weight	γ_{dry}	kN/m³	3	14	22	18	20
Horizontal Permeability	k_x	m/Day	1.5	0.864	0.009	0.086	8640
Vertical Permeability	k_y	m/Day	1.5	0.864	0.009	0.086	8640
Young's Modulus	E	MN/m²	10	40	4200	30	175
Poisson's Ratio	ν	-	0.4	0.25	0.2	0.35	0.2
Cohesion	c'	kN/m²	5	5	1000	20	0
Friction Angle	ϕ'	°	20	30	45	30	40
Compression index	C_c	-	1.25	-	-	-	-
Swell Index	C_s	-	0.1	-	-	-	-

Table3 – Material parameters of piles

Parameter	Symbol	Unit	Value
Young's Modulus	E_{conc}	MPa	30000
Poisson's Ratio	ν	-	0.15
Area of a single pile	A	m^2	0.16
Moment of inertia of the pile section	I	m^4	21.3

Table 4- Material parameters for geogrids

Parameter	Symbol	Unit	Value
Tensile Strength	T_g	kN/m	200
Strain (at 600 kN/m)	ε	%	6

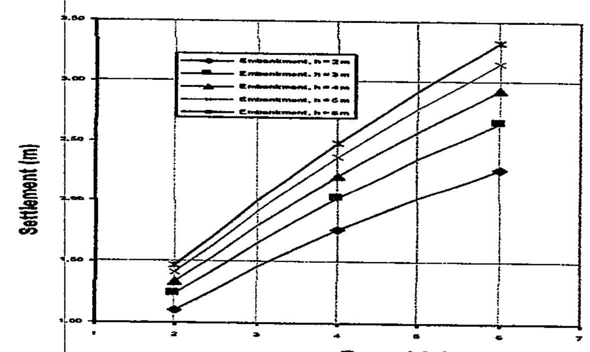


Figure 4- Variation of consolidation settlement with peat thickness and embankment height (Karunarathna, 2007)

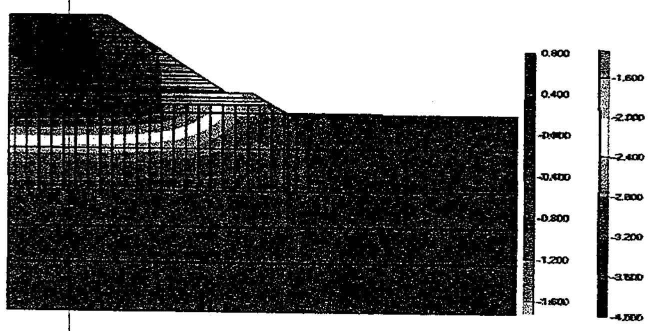


Figure 5- Settlement color contours without any treatment

It can be seen that the differential settlement decreases from the pile head to the top of the embankment. This is due to the development of soil arching at the pile head and the efficient load transfer mechanism created by the load transfer platform constructed with geogrids on the top of the pile. According to the analysis, there has been no discernible differential settlement noticed at the site since the construction.

The approximate settlement profile in longitudinal direction is presented in Figure11.

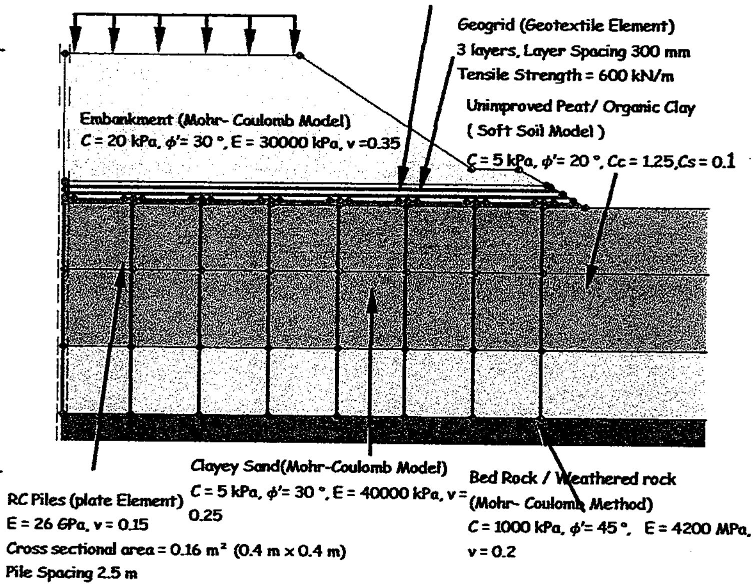


Figure 6- Schematic diagram of GRPE model

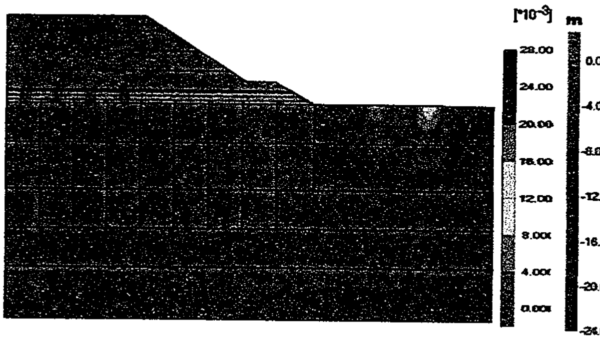


Figure 7- Plot of vertical displacements of 2.5 m pile space section

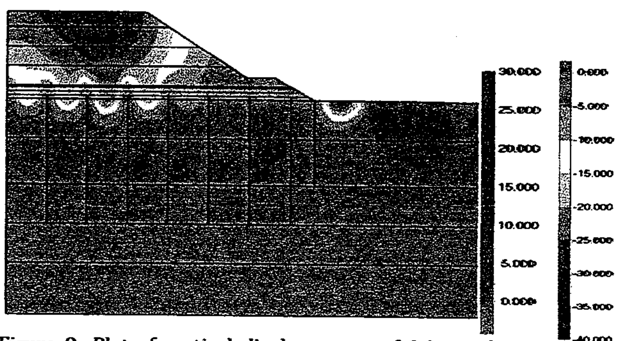


Figure 8- Plot of vertical displacements of 3.0 m pile space section (shadings)

Stresses and the deformation in Geogrid

The load transfer platform consists of 1.2 m gravel mat comprising of three layers of biaxial geogrids with strength of 600 kPa. The mobile tensile strength of geogrids placed on 3m pile spacing is shown in Figure 9. It indicates that more tensile strength has mobilized in the geogrids at the edge of the pile.

Mobilized Strain

The estimated vertical displacements of the geogrid placed on 3.0 m *pile spacing are shown in Figures 10. The calculated maximum total* stain is 0.3 %. Gangakhedkar (2004) reported that to avoid long term localized deformations at the surface of the embankment, the long term strain should be kept to minimum strain value of 2 % for the permanent construction work, which agrees with calculated value.

Table 5- settlement after each construction stage

Stage	Load Application	Time Interval of Load Application /(Days)	Settlement of the top of the embankment	
			2.5 m Pile Spacing	3.0 m Pile Spacing
Initial stage	Existing ground + Driven piles + Geogrid + Gravel Mat (1.2 m Thick)	0	0	0
Stage 1	Initial stage +Embankment(1.5m)	21	2	2
Stage 2	Stage 1+Embankment (1.5 m)	21	5	6
Stage 3	Stage 2+Embankment (1.5 m)	21	10	15
Stage 4	Stage 3+Embankment (0.8 m)	21	16	24
Stage 5	Stage 4+ Open to traffic	21	20	30
Stage 6	Stage 5+ Open to traffic	200	20	31
Stage 7	Stage 6+ Open to traffic	800	20	31

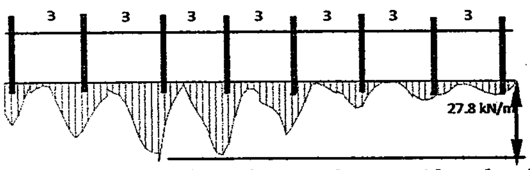


Figure 9 - Mobilized Tensile Strength in geogrid number three at section where pile spacing is 3m

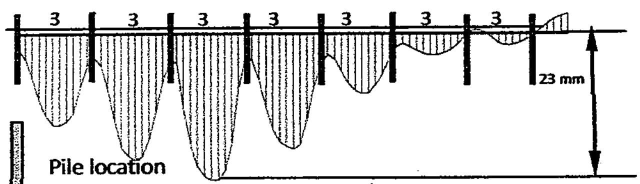


Figure 10- Vertical displacements in 1st geogrid from top.

CONCLUSIONS AND RECOMMENDATIONS

For comfortable rides, elimination of differential settlement between the embankment and the approach slab is very important. At a distance of about 20 m from the slab, the settlement should be comparable with the treated ground settlement under the embankment fill which is ≤ 150 mm. Settlement of the untreated ground is large and soft ground treatment is therefore essential to limit the differential settlement in the approach embankment.

The proposed GPRE system for construction of bridge approaches reduces the total settlement of the embankment; more importantly, the differential settlement between the pile and foundation soil is very small and hence upward lunges, which may be created due to the settlement between the bridge deck and the embankment, will be eliminated.

The conceptual design or preliminary design for the situation was carried out based on published data and guidelines. Then system was analyzed with a validated FE model and it was found the specified design criteria were met.

To verify the design and to check the performance of the embankment, mentoring of settlement is being carried out. Therefore, the aforementioned performance and verification of design can be done in the future.

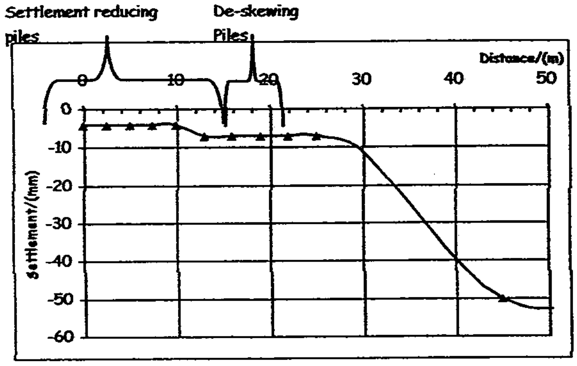
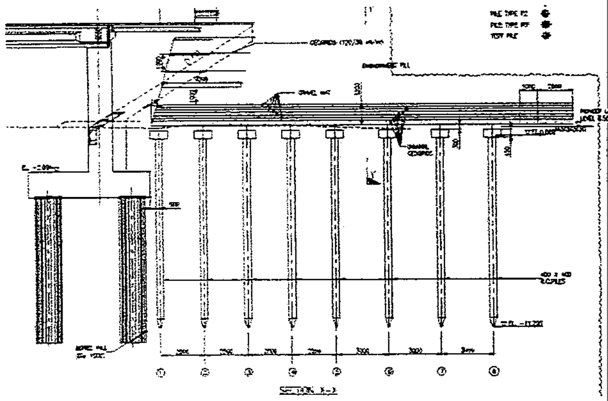


Figure 11-Settlement in longitudinal section of embankment



Longitudinal section
Figure 12- Final design Drawings

ACKNOWLEDGEMENT

Prof. S.A.S. Kulathilaka is acknowledged for giving this valuable opportunity to prepare this paper.
This design and analysis was done under the supervision of Mr. Mazakazu Toki, Team leader, Dr. Asiri Karunawardena, Senior Geotechnical Engineer, and Mr. S. Vasanthan, Geotechnical Engineer of Oriental Consultant. Co, Ltd. Their kind support is acknowledged here.

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Exploration of indigenous knowledge used in Galle Fort construction with respect to Geotechnical Engineering

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ABSTRACT:

Galle fort is the dominant living heritage within the Southern Province of Sri Lanka. However, attention for the work of engineering is less, while a lot of architectural investigations are in a highly active stage. In this study, it was investigated about such ancient geotechnical engineering aspects through exploring the subsurface soil profile. Further, engineering properties of backfill materials were evaluated using conventional laboratory experiments.In addition, a computer based finite element procedure was used to evaluate slope stability of the rampart.

Results show that, rampart is entirely constructed on the bed rock and there was a moat in northern part of the rampart to enhance the security of the fort In addition, it was found that there are some cavities in the western side of the rampart, which can be concluded as the internal transport corridor between bastions within the rampart and prisons or store rooms that has been used in Dutch era. Further, analysis illustrated that stability of the rampart against a rainfall induced slope failure is endurable. This implies the geotechnical engineering knowledge adopted in civil engineering construction even before four hundred years.

Key words: *Indigenous knowledge,Moat, Rainfall, Rampart, Slope stability*

1. INTRODUCTION

The historical studies play a vigorous role for the progression of civil engineering. Therefore many advanced nations give priority for the historical studies in the civil engineering.

Galle fort is the only living heritage within the Southern Province of Sri Lanka which is in the Bay of Galle on the south-east coast. It was built first in 1558 by the Portuguese, and then comprehensively fortified by the Dutch during the 17th century [1]. It maintains a genteel appearance even after four centuries.

Even though such a significant structure is existed within us, attention for the work of engineering is very poor for the Galle fort, despite the fact that a lot of architectural investigations are in a highly active stage.

Albeit being a world heritage, it hardly exist the details of the rampart, such as cross sectional details of the retaining walls, details of backfill material and construction procedures used in different stages of the historical era (Portuguese, Dutch and British). In this research study it was intended to investigate about such ancient geotechnical engineering aspects, which is a social service to be scrutinizing that information and preserved for the future generation for further studies.

2. METHODOLOGY

Research study was conducted in three separate stages. Subsurface soil profile of the rampart was evaluated at first stage. Then engineering properties of the materials within and surrounding the rampart were determined using laboratory experiments. Finally, slope stability of the rampart under different rainfall intensities was evaluated.

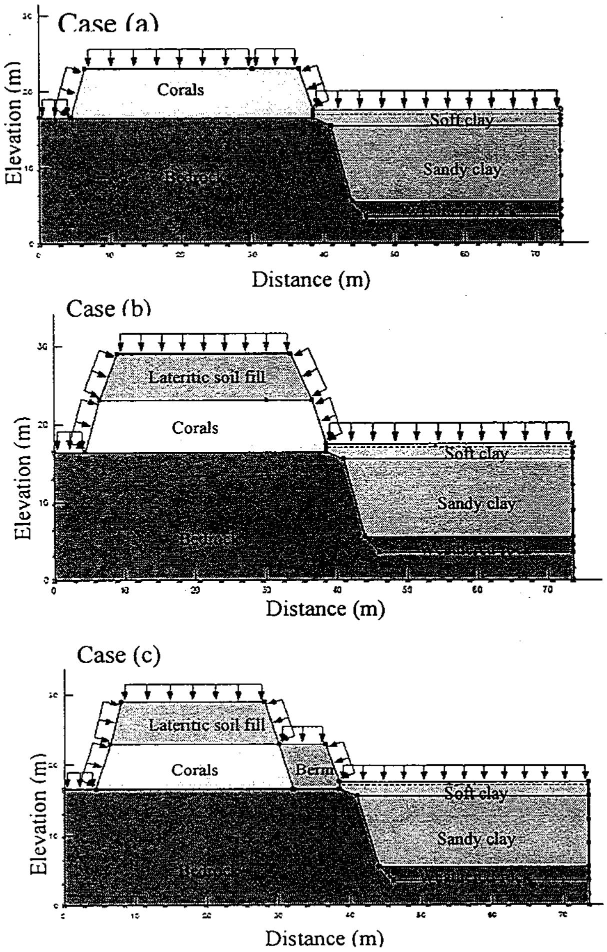
2.1 Evaluation of subsurface soil profile

Initially, electrical resistivity survey, which is one of the most commonly used geophysical exploration techniques, was conducted covering most areas of the rampart. Based on the results of electrical resistivity survey, three locations were selected for further investigation using wash boring method [2]. It is well known that sub soil exploration technique is very useful to calibrate the electrical resistivity survey results.

After the field investigation, engineering properties of subsurface soil, which were collected during subsurface soil exploration, were determined using laboratory experiments and results are summarized in Table 1.

2.2 Evaluation of slope stability of rampart

The analysis was performed for three cases using the data collected through subsurface soil investigation and the historical data, paying more attention to the evolution of the rampart throughout the four centuries. The three cases used for the stability analysis is shown in Figure 1.They represent (a) Cross section of the rampart built by Portuguese (b) Cross section of the rampart after fortified by Dutch (c) Cross section of the rampart with a berm.



~~Figure 1: Slope geometry and boundary conditions~~

Infiltration of rainwater through the unsaturated soil slope and the consequent changes in the pore pressure regime was analysed by the SEEP/W_2007 computer package. Thereafter, the stability of the slope was analysed using the SLOPE/W_2007 package incorporating the pore water pressures derived from the SEEP/W analysis as described by [3]. Stability analysis was carried out for different time steps ranging from 1 day to 5 days, for rainfall intensities of 5mm/hr., 20mm/hr., and 40mm/hr. respectively. The occurrence of both circular and non-circular modes of failure were analysed using Bishop's simplified method and Spencer's method respectively.

3. RESULTS AND DISCUSSION

3.1 Evolution of rampart

The subsurface profile of the rampart based on soil investigations is illustrated in Figure 3. It can be noted that rampart is entirely constructed on the bed rock. It is clear that Portuguese constructed the rampart up to a height of 6.0 m using corals. Further, according to the bedrock dipping pattern, it confirms the fact of the existence of a moat in front of the rampart, which is shown in Figure 2 [1].

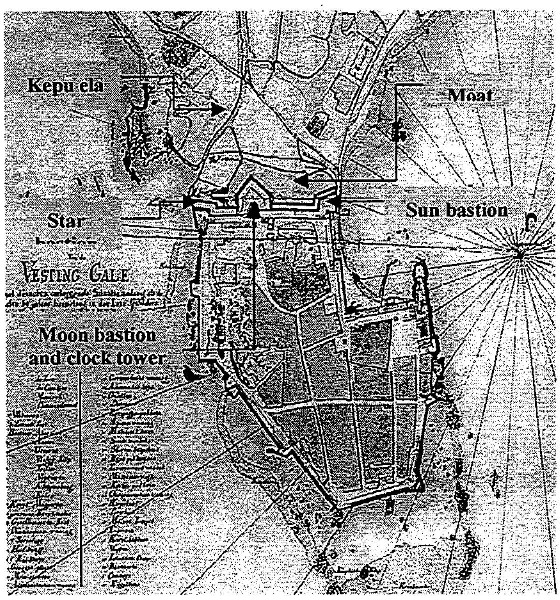


Figure 2: Plan view of the Galle Fort in 1790's [1]

As depicted in Figure 3, the moat was constructed up to a depth of 12.0 m by excavating until the weathered rock level. According to the soil investigations, it was revealed that this moat was later filled with uncompacted sandy clay as shown in Figure 3.

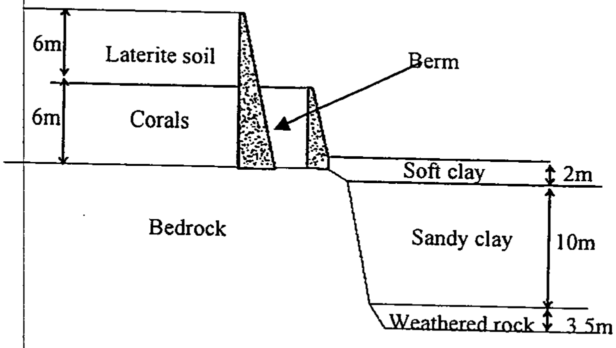


Figure 3: Subsurface profile of rampart

When the fort came under the control of the Dutch, they considered the old fortifications built by the Portuguese is unsafe and they raise the elevation of the rampart by another 6.0 m using laterite soil as shown in Figure 3. Due to 12.0 m height of the rampart, berm was constructed as illustrated in Figure 3 in order to stabilize the slope.

Table 1: Engineering properties of subsurface soil

Soil type	Unit weight (kN/m^3)	Cohesion (kPa)	Friction angle (^{0})
Coral	20	6000	38
Soft clay	14	5	20
Laterite soil	16	10	22
Sandy clay	18	12	28
Weathered rock	19	16	35
Bed rock	22	11590	45

Further, according to the Electrical Resistivity (ER) survey results, it can be seen that many cavities are present in western part of the rampart as shown in Figure 4 and Figure 5.

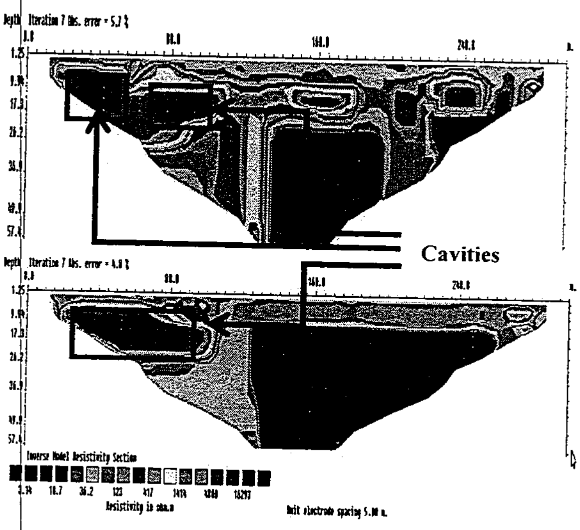


Figure 4: Results of Electrical Resistivity Survey on the western part of the rampart

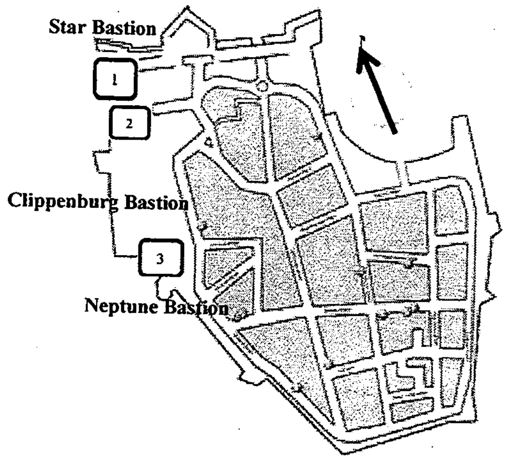


Figure 5: Locations of the cavities on the western part of the rampart

Most probably these cavities show the internal transport corridor between bastions within the rampart [1]. By an another angle, it can be concluded that these cavities have been used by the Dutch as prisons or some kind of a store rooms (ex. Explosive store room) due to longer size of the cavities.

3.2 Slope stability of rampart

The slope stability results for three cases, which illustrate the reduction of the Factor of Safety (FOS) as rain progressed, are summarized in Figure 6.

As shown in Figure 6(a), the rampart constructed by Portuguese has sufficient FOS against rainfall induced slope failure, even under the most critical situation. However, when the elevation of the rampart raised by Dutch as shown in Figure 6(b), FOS has been significantly reduced and which may lead to fail the slope. Therefore, Dutch has constructed a berm in order to improve the stability. As depicted in Figure 6(c), 12.0 m height rampart has sufficient FOS under the most critical condition with the introduction of the berm. This clearly indicates the adoption of indigenous knowledge on geotechnical engineering for civil engineering construction.

From the reconnaissance survey, it was found that the top surface of the rampart is constructed having a slope towards outside of the fort and can see the accommodation of the flow of rain water seepage by providing weep holes at the top of the rampart (Figure 7).Hence, it is well understood that the Dutch knew the adverse effect of the pore water pressure development within the soil structure, and they have taken necessary precautions to minimize this adverse effect. Therefore, this determination clearly illustrates the geotechnical knowledge even in 400 years before.

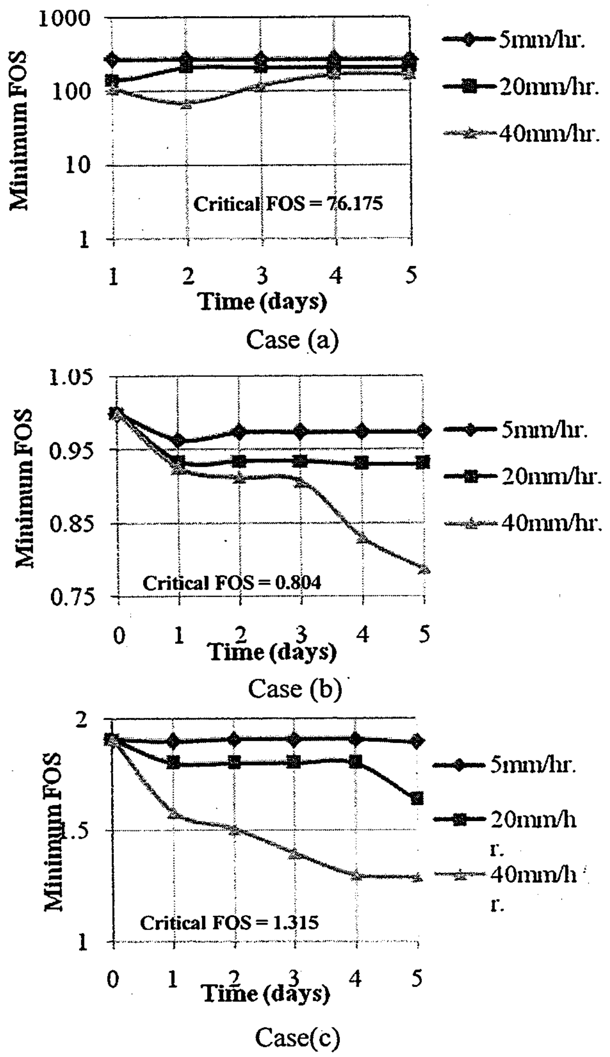


Figure 6: Variation of minimum FOS vs. time

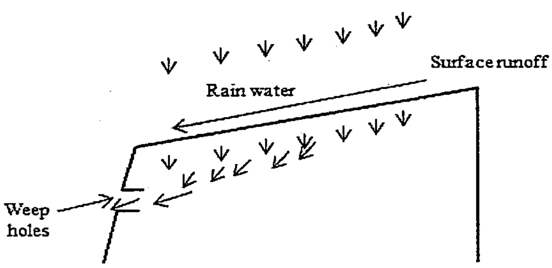


Figure 7: Cross section of top of the rampart

4. CONCLUSIONS

Based on soil investigations, it was revealed that the rampart is entirely constructed on the bed rock and it avoids the undesirable settlement of this huge and heavy structure with time. Northern rampart is the oldest part of the rampart which has been initially built by Portuguese and later fortified by Dutch. In addition, there are some internal transport corridors between bastions and prisons or store rooms within the western side of the rampart which has been used by Dutch. The Portuguese constructed the rampart using corals, which has sufficient FOS against rainfall induced slope failure. With the fortification done to the rampart by Dutch, the FOS has been significantly reduced. Then, they use a berm to improve the stability of the rampart indicating their knowledge on geotechnical engineering. By providing weep holes and constructing a sloping top surface Dutch have reduce the rainfall infiltration to the soil structure. It decreases the development of pore water pressure within the soil structure while increasing the slope stability. Further, by taking the advantage of bedrock dipping pattern, a moat has been constructed by the Portuguese and afterwards closed down by the British.

ACKNOWLEDGMENT

This study was supported by the research grant received under the transforming University of Ruhuna into International Level.

Special gratitude is conveyed toNational Building Research Organization (NBRO) for the assistance in conducting the Electrical Resistivity (ER) Survey.

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