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Study of Nilgiri Hills - A Landslide Prone Area and Its Seepage Analysis

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ABSTRACT

India is among the top ten countries with the highest percentage of landslide fatalities for the past few years. Intense rainfall during the monsoon in 2009 in the hilly district of Nilgiris, in the state of Tamilnadu in India, triggered landslides at more than three hundred locations which affected road and rail traffic and destroyed number of buildings that left more than forty people dead and hundreds homeless. In this report three case histories are investigated: failure of slope in a railway track at Aravankadu , failure of retaining walls supporting buildings at Coonoor, failure of slope and retaining wall along national highway at Chinnabikatty. Laboratory investigations are carried out on soil samples collected at the sites. Soils at all the three locations have high fine content and low values of coefficient of permeability. Finite element analyses of all the three case histories were carried out using PLAXFLOW software to understand the failure mechanism and contributing factors to determine the critical slip surface and factor of safety. Safety analysis using flow field technique is carried out for the slope at Aravankadu..Degree of saturation and pore pressure of Coonoor site revealed that the zone of intense shearing behind the retaining walls due to combined effect of surcharge loading of building and generated pore pressure.

Keywords: Landslide; rainfall; slope; retaining wall; finite element

INTRODUCTION

Landslides are one of the natural hazards that affect fifteen per cent of the land area of India. India is among the top ten countries with the highest percentage of landslide fatalities for the years 2003, 2007 and 2008 (Kirschbaum et al. 2010).

Landslides of different types are frequent in geodynamically active regions in the Himalayan and the north eastern parts as well as in the relatively stable regions of the Meghalaya Plateau, Western Ghats and Nilgiri Hills of the country (NDMG 2009). The phenomenon of landslides is pronounced during the monsoon period (NDMG 2009). Rainfall induced landslides in India in the recent past include slides in Sikkim, Uttrakhand and Western ghats (Anbarasu et al. 2010;Sengupta et al. 2010; Venkatachalam et al. 2002). Landslide disasters have both short-term and long-term impact on society and the environment. In the year 2005 alone, more than five hundred human lives were lost due to landslides in India (NDMG 2009). In the recent past, many studies have been reported on shallow landslides, debris flow and rainfall-induced landslides (e.g. Ozdemir and Delikanli 2009, Teoman et al. 2004, Jotisankasa and Vathananukij 2008). It is generally recognized that the rainfall-induced landslides are caused by excess pore pressures and seepage forces during periods of intense rainfall. It is the excess pore water pressure that decreases the effective stress in the soil and thus, reduces the shear strength of the soil, consequently resulting in slope failure (Anderson and Sitar 1995, Cascini et al. 2010). Xu and Zhang (2010) observed that the rainfall infiltration and a failed drainage system behind the retaining wall resulted in high saturation of the clay and finally led to an undrained failure of slope above a railway line in China. Cascini et al. (2010) presented geomehanical modeling of failure and post failure stages of rainfall induced shallow landslides of the flow-type. The effects of positive pore pressure generation in saturated slopes from intense rainfalls have been analyzed by Johnson and Sitar 1990. Kirschbaum et al. (2010) compiled a landslide catalog for rainfall triggered events for several years.

Slope stability is analyzed by the limit equilibrium methods, boundary element methods (Jiang 1990), finite element methods (Matsui and San 1992, Kellezi et al. 2005), neural network methods (Cho 2009), reliability methods (Dodagoudar and Venkatachalam 2000, Zhang et al. 2011), distinct element method (Bobet et al. 2009) and hybrid continuum-discontinuous methods (Stead et al. 2006). Limit equilibrium methods of slices are the most commonly used methods among others since simplicity and ease of use are the main advantages (Alkasawneh et al. 2008, Cheng et al. 2007).

The finite element method offers an alternative that is more rigorous, free of excessively unrealistic assumptions, and able to provide stress and deformation information. Among the methods for slope stability analysis using finite elements, gravity increase method (Swan and Seo 1992) and strength reduction method (Matsui and San 1992) are most widely used methods (Alkasawneh et al. 2008). The strength reduction method was used for slope stability analysis by various researchers (Matsui and San 1992, Dawson et al. 1999, Griffiths and Lane 1999). The main advantages of strength reduction methods are: the sliding surface does not need to be defined beforehand and is automatically found from the shear strain arising from the application of gravity loads and the reduction of shear strength, therefore, a shear surface closer to the natural sliding surface is defined (Berilgen 2007, Cheng et al. 2007); and can give information such as stresses, movements and pore pressures (Cheng et al. 2007). Deformation analysis using two dimensional finite element plane strain model has also been used to study the behavior of slopes (Berilgen 2007), sheet pile wall (Tan and Paikowsky 2008,

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Georgiadis and Anagnostopoulos 1998) and retaining wall (Shamsabadi et al. 2010).

This report presents three case histories of failures suffered by infrastructural facilities due to rainfall induced landslides in Nilgiris, India in 2009. Soil samples were collected and experimental investigations were carried. Finite element analyses of all the three case histories were carried out using PLAXFLOW.

Fig. 1 Location of study areas within: (a) India (b) Tamilnadu state and (c) Nilgiris District

THE NILGIRIS

The Nilgiris district is located in the southern state of Tamilnadu in India. The location of Nilgiris is shown in Figure 1. The Nilgiris is bounded on the north by the state of Karnataka, on the east by Coimbatore and Erode districts, on the south by Coimbatore district and on the west by the state of Kerala. The Nilgiris is situated at an elevation of 900 to 2636 meters above Mean Sea Level (MSL). The topography of the Nilgiris district is rolling and steep. Tea, carrot, potato, pepper, cabbage, coffee and spices are important agricultural products of the district. The Nagapattinam -Gudalur National Highway (NH67) passes through the district. The Nilgiris is an important tourist center in southern India. Nilgiri Mountain Railway (NMR), Mettupalayam to Ooty line, was declared as World Heritage site by UNESCO. The Nilgiris is in seismic zone III (IS 1893 -2002). According to landslide hazard zonation map prepared by Geological Survey of India, the Nilgiris has moderate hazard (NDMG 2009).

GEOMORPHOLOGY OF NILGIRIS

The Nilgiri Hills rise aloft from the upland of Coimbatore which is a plateau sloping steeply into the Mysore Plateau towards north and merging gradually with the western ghats in the north-west, west and south-west. The drainage is radial at many points due to dominant high points, and is influenced mainly by the joint pattern and foliation trends of the rocks (Rajakumar et al. 2007). Two types of landforms have been identified in the Nilgiris region. The Dodabetta landform has many high peaks having steep slope and escarpments with or without soil cover, around which a radial drainage pattern occurs. The Ootacamund landform has gentle mounds with a thick soil cover. The Nilgiris region falls in the tropical zone of weathering. Most part of the Nilgiris is deeply weathered and at some places thick soil cover of up to 40 m is found (Rajakumar et al. 2007). Intense precipitations followed by dry periods have helped in the formation of considerable depth of weathering (Seshagiri et al. 1982). The Nilgiris mountain ranges comprise of Archaean metamorphic rocks like charnockite, biotite gneiss, magnetic quartzite, hornblende granulite along with some intrusive bodies like pegmatite, dolerite and quartz veins (Seshagiri et al. 1982, Kandasamy et al. 2002, Jayabalan and Lakshminarayanan 2009). Structurally the area is highly disturbed and is subjected to faulting (Kandasamy et al. 2002). The Nilgiris plateau has been formed by three systems of faults along its peripheries (Seshagiri et al. 1982).

RAINFALL IN NILGIRIS

The Nilgiris district usually receives rainfall both during south west (June, July) and north east (October, November) monsoons. The entire Gudalur, Pandalaur, Kundahtaluks and a part of Udhagamandalam taluk receive rainfall by the south west monsoon and some portion of Udhagamandalam taluk and the entire Coonoor and Kotagiri taluks are benefited by the north east monsoon. The average annual rainfall of the district is about 1700 mm. The average monthly rainfall (IMD 2011) in Nilgiris is shown in Figure 2 for last five years (2006 to 2010). As can be observed from the Fig. 2, the district received a record average rainfall of 658 mm in the month of November 2009 (north east monsoon), which is 415% higher than a long period (50 years) average monthly rainfall (IMD 2011). Daily rainfall data for two weeks before the occurrence of slides in the discussed case histories collected from nearby rain gauge stations and presented in Table 1.

Table 1 Daily Rainfall data

Month	Date	Rainfall (mm)	
		Location of Raingauge station	
		Coonoor (For Coonoor site)	Kethi (For Aravankadu and Chinnabikatty sites)
October 2009	27	23.4	-
	28	2.6	-
	29	2.6	2
	30	27	6
	31	18.8	5
November 2009	1	-	-
	2	1	-
	3	14.4	-
	4	55.2	-
	5	12	12
	6	-	40
	7	3.6	12
	8	141.2	5
	9	366	225
	10	395	312
	11	8.2	820

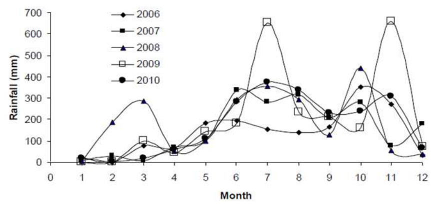

Fig. 2 Rainfall in the Nilgiris during last five years

PAST LANDSLIDES IN NILGIRIS

The major landslides in the past in the Nilgiri Hills were Runnymede, Hospital, Glenmore, Coonoor, and Karadipallam slides (NDMG 2009; Bhandari 2006). More than thirty five people were buried alive in a place called Geddai due to cloudburst and subsequent landslides during October 1990 (Nilgiris 2011). There was another cloudburst during November 1993 in the upper reach of Marappalam of Coonoor taluk which washed away Coonoor –Mettupalayam ghat road for about one km. The road traffic was suspended for more than a fortnight. Twelve persons lost their lives when their huts were washed away (Nilgiris 2011). Due to continuous rainfall during December 2001, two massive landslides occurred near Pudukadu on the Coonoor -Mettupalayam highway (Nilgiris 2011). The railway track between Coonoor -Mettupalayam also got affected. A section of national highway near Burliar was washed off due to landslide during November 2006 (NDMG 2009).

NOVEMBER 2009 LANDSLIDES IN NILGIRIS

The Nilgiris District received heavy rainfall during north east monsoon in November 2009 (from 4th November to 11th November) (Fig. 2 and Table 1). Intense rainfall triggered landslides at more than three hundred locations which affected road and rail traffic and destroyed number of buildings that left more than forty people dead and hundreds homeless (Chandrasekaran et al. 2011, Chandrasekaran 2010, GSI 2011). Mettupalayam –Coonoor –Ooty National Highway, the life line of the Nilgiris, had landslides at many places. The road got completely washed away at few locations and remained cutoff for a period of three months. Nilgiris Mountain Railway (NMR), declared as World heritage site by United Nations, suffered severe damages. The railway track was left hanging at many places with the supporting subgrade washed away and few bridges completely destroyed. In the 27 km long Mettupalayam – Coonoor rack section, about 18,000 cubic meters of debris had to be removed. In this paper, three case histories of failures in November 2009 landslides are presented and investigated.

The three case histories investigated are: failure of a slope supporting railway track at Aravankadu, failure of retaining walls supporting buildings at Coonoor, failure of slope and retaining wall supporting road (NH67) at Chinnabikatty. The locations of the sites: Aravankadu ,Coonoor and Chinnabikatty are shown in Fig. 1. The methodology adopted consists of three phases – field visit and collection of soil samples and required data, laboratory testing of collected soil samples and finite element analysis using PLAXFLOW software and hence seepage analysis is done.

FIELD OBSERVATIONS

A major landslide at Aravankadu which affected the railway track of Coonoor – Ooty section of Nilgiris Mountain Railway (NMR) is shown in Fig. 3 (a).

(a)

The site was located at an altitude of 1812 m above Mean Sea Level (MSL). The slide was of the rotational type and occurred on a curve as can be observed from Fig.3 (b).

(b)

The landslide completely moved away the supporting subgrade and left the railway track hanging for a length of about 100 m (Fig. 3 a). Heavy rainfall occurred from

5to 11November 2009 (Table 1) during north east monsoon, which was preceded with high intensity rainfall in July 2009 during south west moon (Fig. 2). Intense rainfall and blocking of drainage culvert led to the saturation of soil. Mud flow (Fig. 3 c) of about 200m down the slope was observed upto a stream running below the slope.

(c)

Fig. 3 Landslide along railway track at Aravankadu (a) view of slide and hanging of railway track, (b) view of failed surface and (c) view of mud flow

Soil samples, both undisturbed and disturbed, were collected from the exposed sliding surface.

Figure 4 (a) shows buildings at Coonoor, located at the crest of a steep slope of 15 m high.

(a)

The buildings were supported on three levels of reinforced concrete retaining walls as shown in the Fig.4(b). No weep holes were provided in the retaining walls.

(b)

Heavy rainfall occurred from 3to 10November 2009 (Table 1) during north east monsoon, which was preceded with high intensity rainfall in July 2009 during south west moon (Fig. 2). Intense rainfall, blocking of drainage paths and no provision of weep holes led to the saturation of soil. Sliding of the soil on the side and accumulation of soil on the backfill added to the surcharge on the retaining walls (Fig.4 c).

(c)

The top two retaining walls got severely tilted and failed and the retaining wall at bottom cracked (Fig. 4d) such that the foundation of the buildings got exposed.

(d)

Fig. 4 Building supported on retaining walls in Coonoor: (a) Overall view of buildi with retaining walls, (b) view of three levels of retaining walls, (c) view of sliding and surcharge load on retaining wall and (d) view of tilting and failure of retaining

Soil samples were collected from behind and in front of the retaining walls. The data regarding retaining wall – thickness, grade of concrete, height of wall and depth of embedment were assessed.

Figure 5 (a) shows the failure of a slope supporting road (National highway NH67) at Chinnabikatty, in the Coonoor to Ooty section of the National Highway.

(a)

Intense rainfall (Table 1) and blocking of drainage ditches led to the saturation of soil. This was one of the major slides which led to disruption of traffic for a period of about two months. Soil samples were collected from the exposed sliding surface.

Mudflow surrounding the building below the road and construction of stone masonry retaining walls are depicted in Fig. 5 (b).

(b)

This is the site of repeated failures with slide occurring again during monsoon season of 2010. Figure 5 (c) shows the overall view at present (2011), with the remedial measure-the stone masonry retaining wall supporting the road, the building above the road and service road and building below.

(c)

Fig. 5 Slope and retaining wall failure along national highway at Chinnabikatty: (a) of failure of slope supporting road in 2009, (b) view of mud flow and retaining under construction in 2009 and (c) overall view at present (2011) showing retai walls, road and building above

EXPERIMENTAL INVESTIGATIONS ON SOIL SAMPLES

Extensive laboratory investigations are carried out on soil samples, collected from all the three sites mentioned. Specific gravity, liquid limit, plastic limit, sieve analysis of soil samples were carried out as per relevant ASTM standards. Falling head permeability test as per IS 2720 (17) – 2002 was carried out on soil samples to find coefficient of permeability. Consolidated drained direct shear tests on undisturbed soil samples were carried out as per ASTM D3080 -04. The properties of the soil samples tested are given in Table 2.

Table 2 Properties of soil samples at different locations

Location / Property		Aravankadu (Railway)	Coonoor (Building)	Chinnabikatty (Road)
Specific gravity G		2.71	2.65	2.58
Liquid Limit LL (%)		33	45	44
Plastic Limit PL (%)		28	25	29
Platicity Index PI (%)		5	20	15
Saturated unit weight (kN/m^3)		19.8	17.9	18.7
Coefficient of permeability k (m/s)		$2.03 * 10^{-7}$	$7.2 * 10^{-10}$	$5.9 * 10^{-8}$
Soil Classification		SM Silty Sand	CL Lean Clay with Sand	ML Sandy Silt
Effective cohesion c' (kPa)	Experiment (CD Direct shear)	7	7	18
	Back analysis	7	7	8
Effective angle of internal friction φ' (Degrees)	Experiment (CD Direct shear)	30	22	23
	Back analysis	30	28	23

Soils at all the three locations have high fine content (less than 75µ) – 40%, 77%, and 54% for the sites in Aravankadu, Coonoor and Chinnabikatty respectively. The fines have low plasticity, liquid limit value less than 50% (Table 2). The soil at Aravankadu has low (10-7 m/s) and at Coonoor has very low (10-10 m/s) values of coefficient of permeability. High fine content and low permeability values indicate that the dissipation of pore water, during rise in groundwater table due to rain fall, takes long time and soils have undrained condition. The effective shear parameters obtained from consolidated drained direct shear tests indicate low values of cohesion (c') and angle of internal friction (f') (Table 2). The soil samples were classified as per Unified Soil Classification System (ASTM D2487–11). The soil types are**: silty sand (SM) at Aravankadu, lean clay with sand (CL) at Coonoor and sandy silt (ML) at Chinnabikatty.**

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FINITE ELEMENT ANALYSIS

Finite element analyses of all the three case histories were carried out using PLAXFLOW software , *Slope Supporting Railway Track at Aravankadu.* Two dimensional plane strain modeling of the slope at Aravankadu, as in 2009, is carried out. The plane strain model assumes that the strains in out of plane direction are zero.

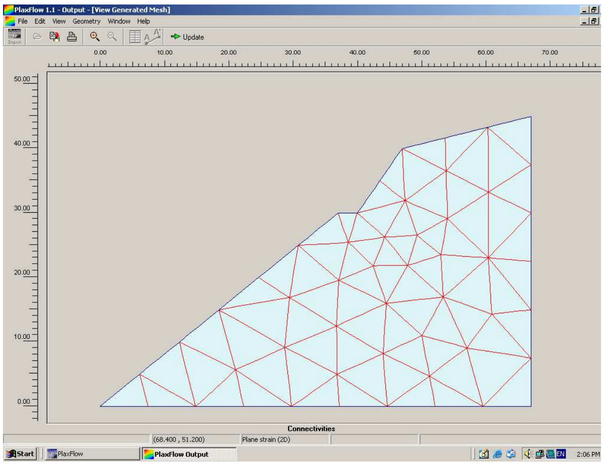

Fig 6a) The model geometry along with generated mesh at Aravankadu.

Fifteen nodded triangular elements are used to model the soil. The boundary conditions – fully fixed (in both X and Y directions) at the base of the geometry and roller conditions (restrained in X direction) at the vertical sides are selected. The Mohr-Coulomb elastic-perfectly plastic constitutive model is used for soil.

Table 3 Input parameters

Location	Aravankadu (Railway)	Coonoor (Building)	Chinnabikatty (Road)
Property			
Soil			
Young's modulus E'_s (kPa)	10000	1400	1600
Poisson's ratio v'	0.3	0.3	0.3
Dilatancy angle Ψ (Deg)	0	0	0
Retaining wall			
Material	-	Reinforced cement concrete	Stone masonry
Thickness of wall (m)	-	0.3	0.6
Poisson's ratio v		0.15	0.15
Young's modulus E (kPa)	-	19365000	200000
Normal stiffness EA (kN/m)	-	5809000	120000
Flexural rigidity EI (kN-m^2/m)	-	43600	3600

The analysis is based on effective stress approach. The input parameters used in the analysis, (in addition to the parameters given in Table 2), are given in Table 3. Poisson's ratio of 0.3 is assumed since for undrained material behaviour the effective Poisson's ratio should be smaller than 0.35 (Brinkgreve 2002). Dilatancy angle of 0 degree is assumed since for most cases of soil the angle of dilatancy angle is zero for f' values of less than 30° (Brinkgreve 2002).

According to permeability tests (Table 2), the soil (silty sand SM) is having low permeability (2.03*10-7 m/s). Hence undrained material behaviour is taken for the analysis. The initial stresses are calculated by means of gravity loading. During 2009 prior to very heavy rainfall in the north east monsoon in November (Table1), the Nilgiris region experienced heavy rainfall during south west monsoon (in July) as well (Figure 2).

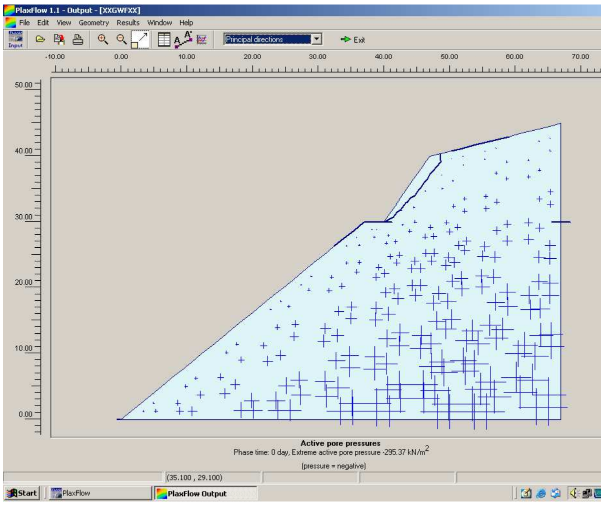

Fig. 6(b) Active pore pressure before rainfall (due to present water table)

Pore pressures are incorporated in the analysis by specifying the phreatic level (Brinkgreve 2002). Safety analysis using Strength reduction (Phi-c reduction) technique is carried out to obtain the factor of safety. In the Phi-c reduction approach the strength parameters tanf and c of the soil are successively reduced until the failure occurs (Brinkgreve 2002). The Phi-c reduction approach resembles the method of calculating safety factors as conventionally adopted in limit equilibrium analyses (Brinkgreve 2002, Griffiths and Lane 1999, Alkasawneh et al. 2008, Cheng et al. 2007).

High pore pressure generated in the embankment, due to high water table and undrained condition, is shown in Fig. 6 (b).Active pore pressure is a combination of steady state pore pressure and excess pore pressure (Brinkgreve 2002). Steady state pore pressures are calculated based on input data, generated on the basis of phreatic level (hydrostatic). Excess pore pressures are pore pressures that occur due to loading (including external loading) of soil clusters in undrained condition. The shape of the deformed mesh and the pattern of the displacements give an indication of the shape of the failure surface. Complete movement of the slope is observed from the Fig. 7a)

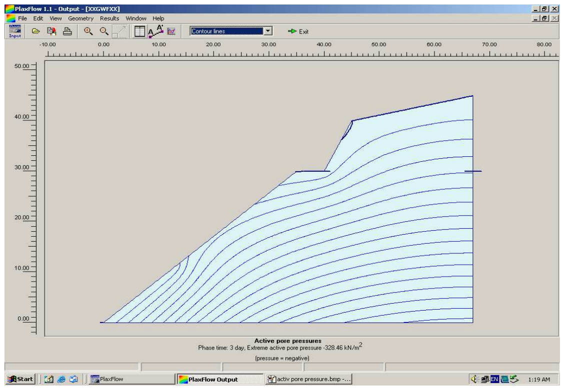

Fig 7a) The pore pressure contours in the embankment

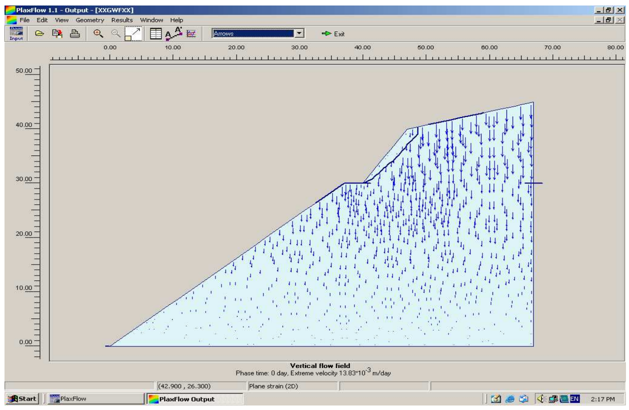

Fig 7b) Vertical flow field at the Aravankadu site

It can be seen that the vertical flow fields are very less, nearly zero near the toe of the slope. This is due to increase in pore pressures as shown in Fig 7a) and led to reduction in shear strength of the soil which consequently resulted in the slope failure.

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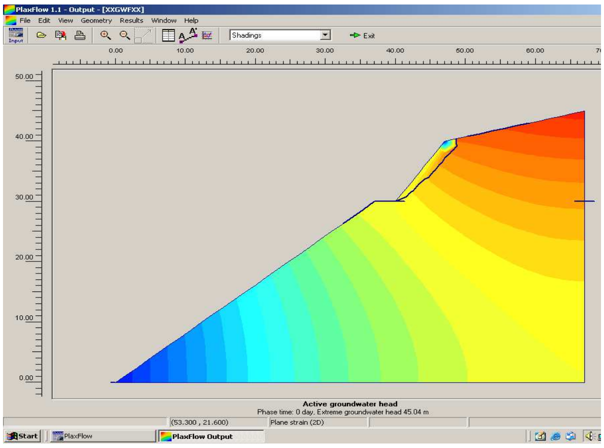
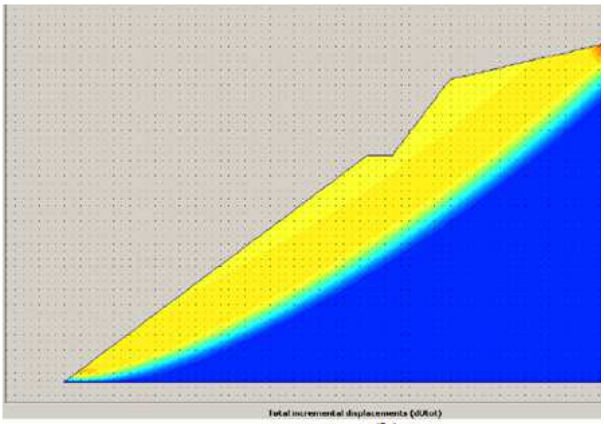
The total active groundwater flow head is shown in Fig. 8(a). The high active groundwater head at the toe of the slope is clearly observed from Fig. 8(a). Figure 8(b) clearly indicates the most appropriate failure mechanism, rotational and translational and the critical slip surface .

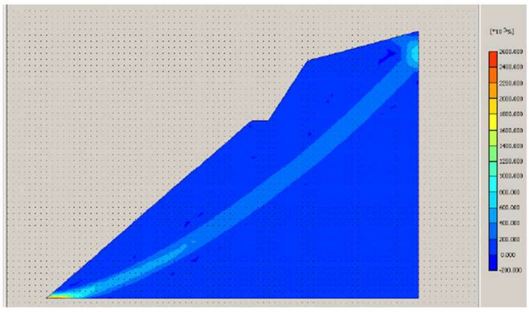


(a)



(b)

The predicted failure-surface shape (Fig. 8b) is in reasonable agreement with that occur red at the site during November 2009 (Fig. 3 a, b). The concentration of shear strain in the critical slip surface is shown in Fig. 8 (c), in light shading.



(c)

The generation of high pore pressure (Fig 6b) and consequent reduction of effective stresses near the toe (Fig 7b), along with high shear strain at toe (Fig 8c) indicate the failure mechanism of progressive failure. The slope fails at toe first and slip surface progressed upwards. The value of sum of the load multipliers obtained at the toe of the slope represents the safety factor. Near to the failure state of the soil, the deformations increase dramatically. The value of factor of safety, evaluated from the analysis, is 0.95.

ANALYSIS CONSIDERING RAINFALL

SITE 1. ARAVANKADU

Analysis considering rainfall intensity and duration is carried out using PlaxFlow(version 1) software (Brinkgreve 2003). During rainfall, it saturates the slope and progressively increases the pore pressures and this is by a process of seepage.

The geometry of the model used in the analysis showing boundary condition, water table and precipitation is depicted in Figure 9 (a).

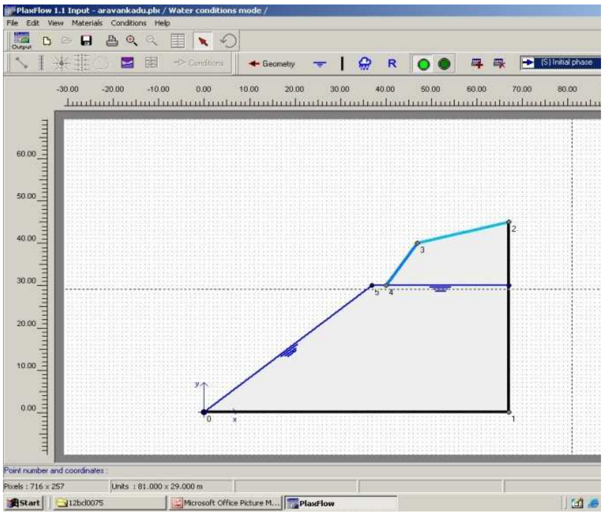


Fig 9a) High water table as is considered to simulate the rise in water table due to prior heavy rainfall occurred during south west monsoon in July 2009 (Fig. 2). The bottom boundary is considered as closed flow boundary, flow across this boundary does not occur. Transient groundwater flow analysis is carried out to consider the variation of rainfall intensity in time. Three days of heavy rainfall 225 mm, 312 mm and 820 mm during November 9, 10and 11respectively (Table 1) are modeled using precipitation option by specifying the intensity of rainfall as recharge (Figure 9a). Infiltration boundary condition is considered at top, where precipitation is applied.

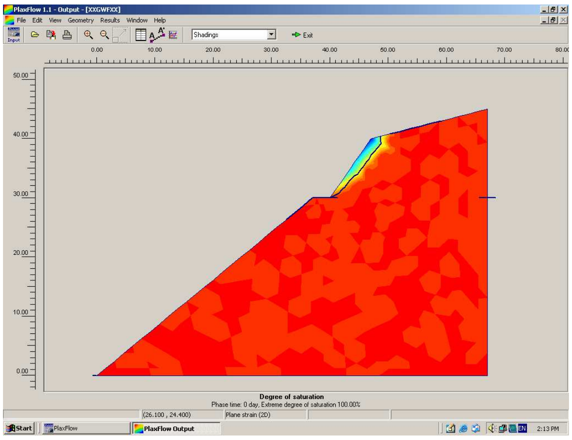


Fig 9b) Degree of saturation after heavy rainfall. The intensity of color is directly related to saturation extent.

The coefficient of permeability value of 2.03 *10-7 m/s (Table 2) is used in the analysis. Approximate Van Genuchten material model (Brinkgreve 2003) is used in the analysis. The flow field intensity at the end of three days of heavy rainfall is shown in Figure 9 (c).

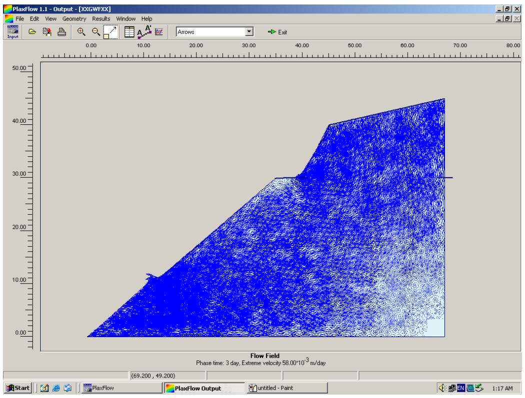


Fig 9c) Flow field after three days of heavy rainfall

The active pore pressure contour, at the end of three days of heavy rainfall, shown indicates high pore pressure due to seepage

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during rainfall. It indicates specific discharge at element stress points. The length of the arrow indicates magnitude of specific discharge and arrow direction indicates the flow direction .The pore pressure calculation in finite element is based on the generated mesh, permeability of soil and time dependent boundary conditions applied**. When the analysis is carried out considering the vertical boundary (right side) as closed flow boundary**, the resulting flow field, shown in Figure 9 (d) correlate well with the failure (Fig 3a) and mud flow (Fig 3c) observed at the site. **Clearly the horizontal flow is directed toward the toe causing its failure.**

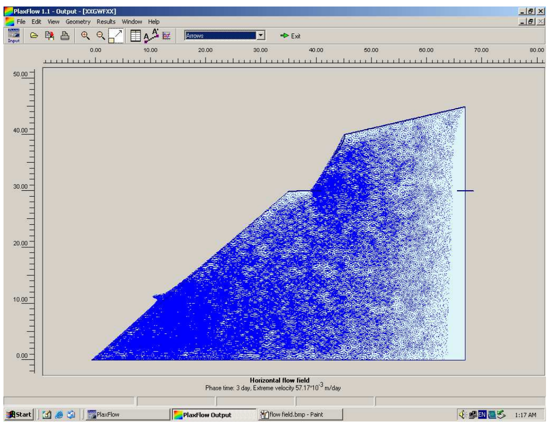

Fig. 9d) Horizontal flow field with vertical wall closed.

SITE 2. COONOOR

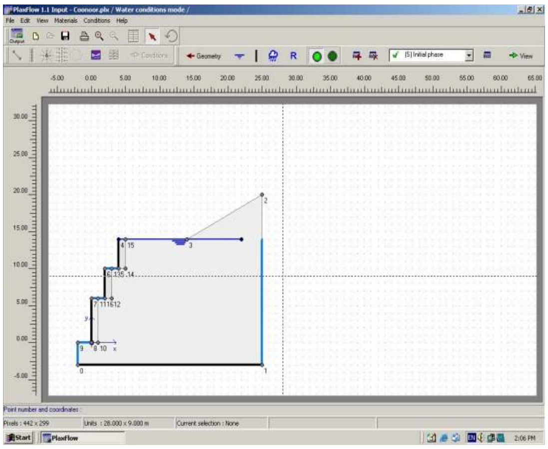

Fig 10) Geometry of Model using PlaxFlow for the Coonoor site

The retaining walls supporting buildings at Coonoor, as in 2009 (Figures 4(a) and (b), are modeled as shown in Fig.10. Two dimensional plane strain modeling is carried out. The soil modeling methodology is same as in the previous case .Retaining walls are modeled using plate elements. The input parameters for retaining wall: thickness, stiffness and flexural rigidity are given in Table 3. Young's modulus value of Reinforced concrete is based on characteristic compressive strength (f_{ck} = 20 N/mm2) of concrete used (M20 grade) and using relation E = 5000*(f_{ck})^0.5 (IS 4562000). The interaction between the wall the soil is modeled by means of interface elements. Strength reduction factor of 0.67 is considered to assign a reduced interface cohesion and friction. Since the soil (lean clay CL) has very low permeability (10^-10m/s) value, the material behavior for soil is taken as undrained condition. Also, at the site, there were no weep holes provided in the retaining walls and no drainage layers provided in the backfill. .

High water table, corresponding to top of the upper retaining wall, is considered in the analysis is shown in Fig.10. The initial stresses are calculated by means of gravity loading. Displacement analysis was carried

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out by plastic calculation using back analysis (section 5.4) c' and f' parameters (Table 2). Plastic calculation is an elastic-plastic deformation analysis.

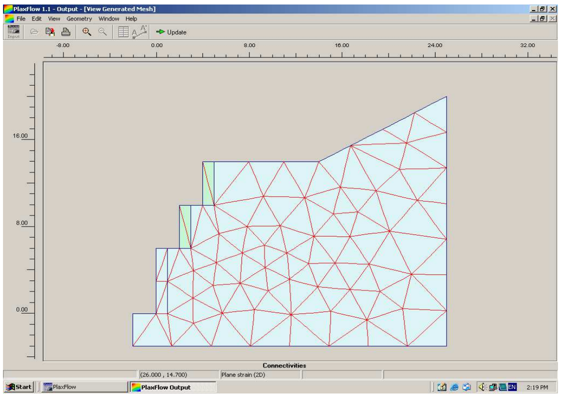

Fig.11) Generated mess of the Coonoor site with the retaining walls considered

The deformed mesh reveals that the bottom two retaining walls get completely displaced and titled; bottom retaining wall got displaced. The deformed mesh (Fig. 11) correlated very well with the observed failure pattern in November 2009 [Fig. 4(d) and 4(b)].

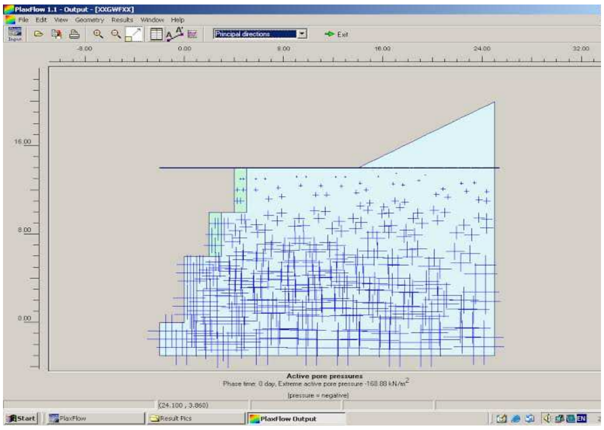

Fig 12) Active pore water pressure at the Coonoor site

Active pore pressure is a combination of steady state pore pressure and excess pore pressure (Brinkgreve 2002). Steady state pore pressures are calculated based on input data, generated on the basis of phreatic level (hydrostatic). Excess pore pressures are pore pressures that are calculated due to loading (including external surcharge loading) of soil clusters in undrained condition.

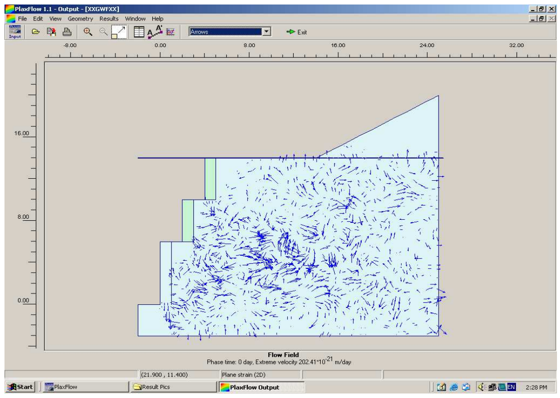

Fig. 13) Flow field (represented by arrows) at the Coonoor site

The plot clearly brings out the zone of intense flow field towards the retaining walls as represented by arrows. This is due to high water table due to intense rainfall and generated pore pressure in undrained condition. The critical slip surface is clearly marked in Fig.13a. The total displacements are shown in Fig.13 (b). The displacement pattern is well established in the figure with the length of arrows indicating the relative magnitude. Concentration of large deviatoric stresses behind the retaining walls, particularly near the bottom retaining wall, can be observed from Fig. 14.

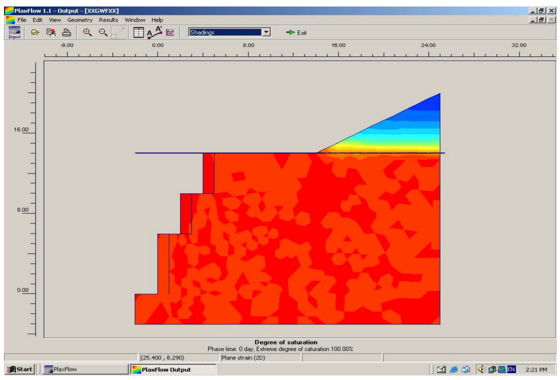

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Fig 14) Degree of saturation after rainfall considering closed boundary

SITE 3. CHINNABIKATTY

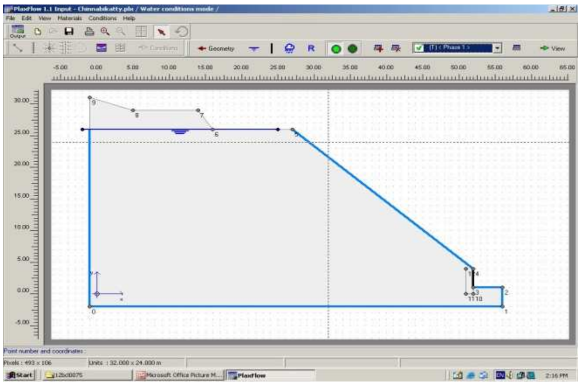

Figure 15a) Slope Supporting Road in Chinnabikatty shows the geometry of the section of slope supporting National Highway at Chinnabikatty, as in 2009 with the retaining wall at toe of the slope.

The soil and retaining wall modeling, initial stresses, displacement analysis are same as explained in the previous case . The input parameters are shown in Table 2 and 3. Since the soil (sandy silt ML) has low permeability (10-8 m/s) value, the material behaviour for soil is taken as undrained condition. Site observations revealed that no drainage arrangement was provided in the backfill. The traffic loading and surcharge due to building above the road are modeled as uniformed distributed loads as shown in Fig. 15(a). The dynamic effects from the traffic are not taken into account. High water table, corresponding to bottom of the road surface, is considered in the analysis (Fig. 15a).

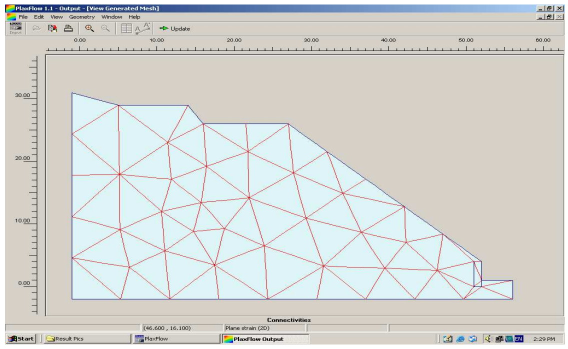

Fig 15b) Generated Mesh for the Model at Chinnabikatty

The generated mesh is shown in Fig. 15(b).). The active pore pressure shown in Fig.16 is consistent with the failure observed in 2009 (Fig. 5 a). The severe titling and failure of retaining wall at toe of the slope is clearly evident from the Fig. 16 (a).

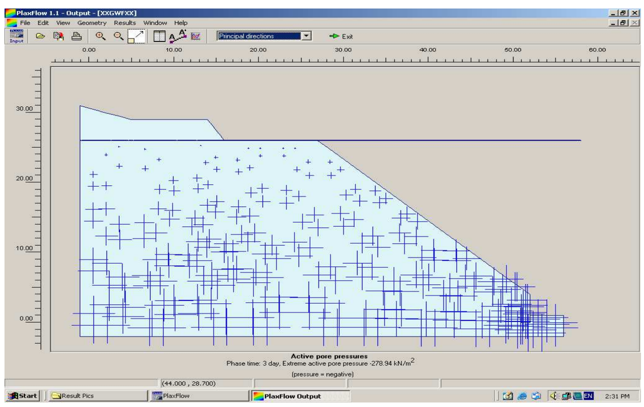

Fig 16) Active pore pressure generated

Active pore pressure is a combination of steady state pore pressure and excess pore pressure (Brinkgreve 2002). Steady state pore pressures are calculated based on

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input data, generated on the basis of phreatic level (hydrostatic). Excess pore pressures are pore pressures that are calculated due to loading (including external surcharge and traffic loading) of soil clusters in undrained condition. High pore water pressure at toe of the slope is depicted in the Fig. 16.

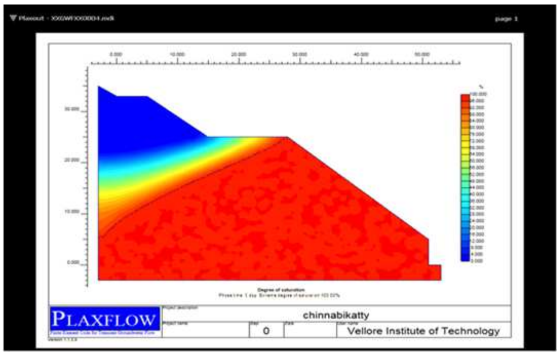

Fig 17) Degree of saturation after heavy rainfall . The curved line indicates the boundary between saturated and unsaturated zone. It can be seen that the degree of saturation at the retaining wall is very high hence sufficient enough to cause failure as was observed at the site.

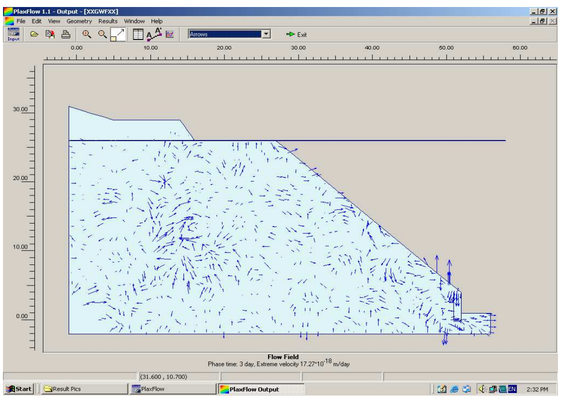

Fig 18)Flow field of the Chinnabikatty site. The arrows indicated the flow field direction. Flow field directed towards the retaining wall is a direct indication of failure.

As a immediate measure after slide in 2009, stone masonry walls were constructed at top face of the slope in addition to wall at toe. Failure occurred again during monsoon in 2010, which led to closure of road for traffic. Ten meter high and 0.6 m thick stone masonry retaining walls are constructed at the top face of the slope (Fig. 5c) as a permanent mitigation measure and the road was opened for traffic.

CONCLUSIONS

During north east monsoon in November 2009, the Nilgiris District, in Tamilnadu, India received very heavy rainfall, 415% higher than a long period average monthly rainfall, that triggered landslides at more than three hundred locations which affected road and rail traffic and destroyed number of buildings that left more than forty people dead and hundreds homeless. The three case histories: failure of slope supporting a railway track at Aravankadu, failure of retaining walls supporting buildings at Coonoor, failure of slope and retaining wall at Chinnabikatty were investigated. Soil samples, both undisturbed and disturbed, were collected from all three sites. Laboratory investigations were carried out on samples, suggested that soils at all the three locations had high fine content and low values of coefficient of permeability. The soil types are: silty sand (SM) at Aravankadu, lean clay with sand (CL) at Coonoor and sandy silt (ML) at Chinnabikatty. Finite element analyses of all the three case histories were carried out using PLAXFLOW software to understand the failure mechanism and causes. Safety analysis using strength reduction technique is carried out to obtain the factor of safety of slope at Aravankadu. It was observed that the effective stresses are very less, nearly zero near the toe of the slope.

The degree of saturation level indicated most appropriate failure mechanism, rotational and translational and the critical slip surface. The generation of high pore pressure and consequent reduction of effective stresses near the toe, along with high shear strain at toe indicate the failure mechanism of progressive failure. The predicted failure-surface shape is in reasonable agreement with that produced during November 2009. The value of factor of safety of 0.95 is evaluated from the analysis. Analysis considering rainfall intensity and duration for Aravankadu slide carried out using PlaxFlow software indicated high pore pressure in slope due to seepage during rainfall. The resulting flow field correlate well with the failure observed at the site.

Degree of saturation analysis was carried out for the site in Coonoor also. The deformed mesh revealed that the top two retaining walls got completely displaced and titled; bottom retaining wall got displaced. The deformed mesh correlated very well with the observed failure pattern in November 2009. The analysis revealed that the zone of intense shearing behind the retaining walls due to combined effect of surcharge loading of building and high water table.

The flow field and degree of saturation of slope supporting National Highway at Chinnabikatty is also carried out. The direction of flow field revealed the occurrence of large displacements at the face and toe of the slope. The curve line represents the boundary between saturated and unsaturated regions and these supports the failure at the toe as represented by the high active pore pressure at the toe.

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