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Forensic analysis of Malin landslide in India

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Abstract: A devastating landslide occurred on 30th July 2014, resulting in the burial of a village of about 40 houses called Malin, in western India and also led to about 160 deaths. The landslide was triggered by heavy rainfall in the area and mass movement of debris. The paper investigates slope failure in the Malin area using back analysis and numerical methods. Site investigation was conducted to obtain representative information of the area. Finite difference analyses using FLAC 2D is performed for the failed slope to determine the possible cause of failure. Analysis results show that slope failure occurred due to the loss of suction strength at the interface between rock and local soil.

1. Introduction

Landslides constitute a major threat to both lives and property worldwide. One of the most common triggering mechanisms for landslides is rainfall and the consequent water infiltration. Rainfall induced landslides are the results of a change in the groundwater conditions, especially in the unsaturated zone. The location of groundwater table in most of the high slopes is deep below the ground surface and the pore-water pressures in the soil above ground water table are negative. This negative pore-water pressure is called matric suction when referenced to pore-air pressure which contributes to the stability of unsaturated soil slopes (Fredlund and Rahardjo, 1993; Rahardjo et al. 1995; Griffiths and Lu, 2005). Hence the effect of rainfall infiltration on slope could result in changing soil suction and positive pore pressure, or main water table, as well as raising soil unit weight, reducing shear strength of rock and soil.

Deep-seated rotational and shallow translational slides can often be observed in slopes which used to be stable under normal rainfall conditions due to extreme rainfall infiltration. In these cases deep-seated slides are generated by a rise in the groundwater level and consequently an increase in pore-water pressures and a lowering of the effective stresses in the soil. The failure surface often is located below the phreatic line. This type of sliding occurs in fine-grained clayey soils. On the other hand, shallow translational slides are mainly triggered in the zone above the groundwater level in silty/sandy soils. The thickness of this zone depends much on the climatic conditions of the area. It can be of the order of a meter in temperate regions and up to tens of meters in tropical and subtropical regions. Once the rain-water starts infiltrating this vadose zone of the soil, the negative pore-water pressures will tend to dissipate due to an increase in the soil water content. This process contributes to lowering the shear strength of the soil layers close to the surface. Under critical conditions, the shear strength of the soil can be reduced below the mobilized shear stress on a potential slip surface and failure of the slope may occur. The majority of hill slopes can be considered unsaturated during normal conditions. Due to this unsaturated state, the slopes can often be found at angles steeper than what would be “theoretically possible” by using the common saturated soil mechanics. In practice, most of the slope stability calculations neglect any suction present above the groundwater table. The difficulties associated with the measurement of negative pore-water pressures and their incorporation into the slope stability analysis is the primary reason for this practice. It is difficult to predict the stability of a natural slope subjected to environmental changes due to the many factors involved in the process such as the soil hydraulic properties, climatic data and the initial groundwater conditions within the slope. Neglecting the negative pore-water pressures can be reasonable in many situations where the slip surface is mainly lying below the phreatic line. However, for situations where the groundwater level is deep and/or shallow sliding is of concern, negative pore-water pressures should not be ignored.

In this study, a rainfall induced landslide which occurred along the upslope of Malin village, Ambegaon Tehsil in Pune district of Maharashtra (India) is examined. The landslide occurred in the early hours of July 30th 2014 claiming 151 human lives and damaging 45 houses. Heavy rainfall was reported in the area on that day and over the previous one week. Hence rainfall infiltration is assumed to be the direct cause of landslide in our analyses. Back analysis of the failed slope is performed to determine the shear strength parameters and other conditions at the time of failure. Generally, back analysis is an effective approach to provide an insight into the underlying failure mechanism and improve the understanding regarding the factors controlling the stability of slopes. One of the advantages of back analysis is that it can account for important factors that may not be well represented in laboratory and in-situ tests such as the presence of cracks and pre-existing shear planes within the soil mass. Finally, the cause of slope failure/landslide is explained based on the basic concepts of unsaturated soil mechanics.

2. Site description

Location: The Malin village is located in AmbegaonTaluka of Pune District of western Maharashtra, India. It is located at an elevation of 760 m and about 95 kms away from Pune city. Geographical coordinates of the village are: Latitude N19°09'34.40", Longitude E73°41'19". Around 70 dwelling units existed before landslide. The hill slopes at levels above and below the village were partially converted to terraces for paddy cultivation.

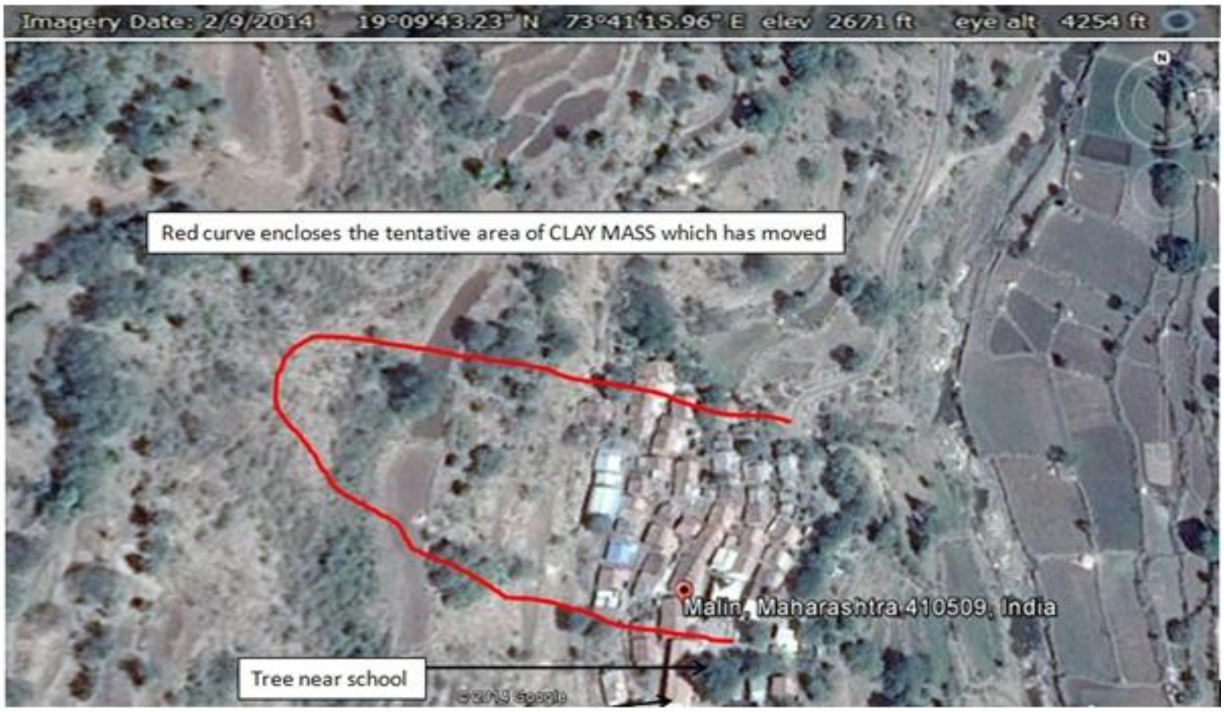

Figure 1: Malin landslide area

Physiography and geology of the area around Malin village

The Malin landslide area is located on the eastern slope of a roughly N-S trending hill. The area is drained by the Ghod River and its tributaries. The area around Malin village is represented by the basaltic lava flows. The seismo-tectonic map of the area revealed the occurrence of two low magnitudes seismic activities (4.0 to 4.9) within 100 km radius of the landslide location. However, these seismic events do not appear to have played any significant role in inducing slope instability in Malin.

Topography

The topography of Malin hill slope is broadly uniform. The entire hill slope where the landslide occurred can be divided into four zones:
Zone 1: from Nala bed level to the road level, which is relatively flatter in its lower part followed by gentler slope in upper part.
Zone 2: from road level to the 2nd slope break/level, which is relatively gentler
Zone 3: from 2nd major slope break/level to the 3rd major slope break, this is moderately inclined.
Zone 4: from 3rd level till the plateau edge/apex of hill slope, this is moderately inclined.

The overall slope inclination as measured from the base topographic map is 25°. Considering the similarity of slope configuration for the entire Malin hill slope, a longitudinal section A-A' is drawn in the southern side of the slide from across the river up to just above the crown level of Malin landslide. This section roughly reveals the pre-slide slope configuration of slide portion. Another section B-B' is drawn almost through the middle of the landslide. This reveals the slope modified due to landslide. Slope angles at various zones along the two sections are given in Table 1. Figure 2 shows the longitudinal topographical section along A-A' line of the slope.

Table 1: Slope angles at different slope zones along sections AA' and BB'

Section	1st Zone		2nd Zone		3rd Zone		4th Zone		
A-A'	20°		10°	25°	30°		18°	25°	50°
B-B'	10°	28°	10°	25°	30°	20°	25°	30°	40°

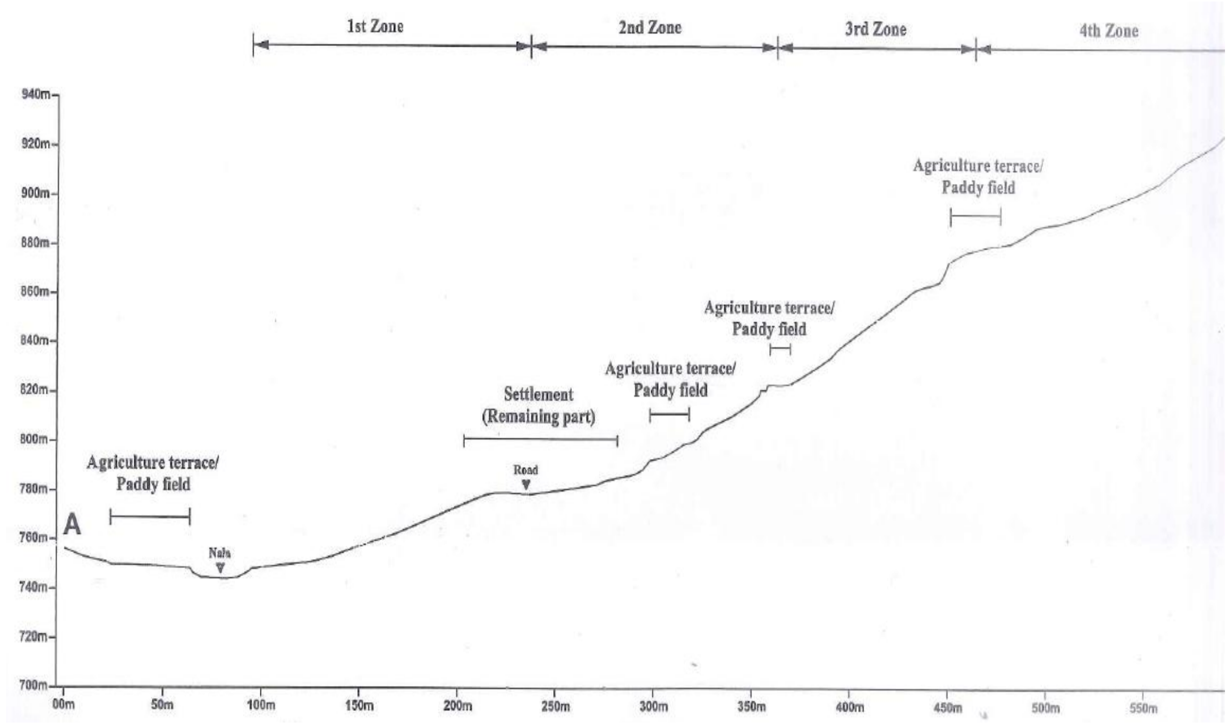

Figure 2: Longitudinal topographical section along A-A'

Features of the Malin Landslide

The height of the landslide is roughly estimated as 190 m while the width of the slide varies from 45 m to 134 m. The entire length of the slide from the crown to toe is 514 m and the area affected is 44245 m^2. The crown of the landslide is at a height of 936 m and marks the top of 4th zone. The width of the landslide in this zone varies from 45 m to 134 m. The Landslide depleted part of 4th zone, entire 3rd zone and maximum part of 2nd zone. Lowest part of 2nd zone and 1st zone form the zone of accumulation. Unfortunately, this zone of accumulation is the settlement area of the Malin village. The maximum thickness of the material slided during landslide could not be established accurately and approximated as 7 m because when the detailed mapping work was carried out, most of the failed slope material has been removed from the place as part of rescue effort.

Representative soil samples were collected from Malin landslide area. These samples were collected from three different levels in main landslide. Table 2 shows the results of geotechnical tests performed on soil samples from failed slope.

Table 2: Results of geotechnical tests on soil samples

Parameter	From slided area		
Zone	2	3	4
Moisture content (%)	36.41	27.92	38.70
Dry density (g/cc)	1.32	1.36	1.24
Liquid limit (%)	53	51	46
Plastic limit (%)	31	29	28
Plasticity Index (PI)	23	23	18
Specific Gravity	2.57	2.51	2.58
Grain Size analysis (Silty clay, %)	57	78	31
Cohesion (Kg/cm^2)	0.125	0.391	0.339
Friction angle (°)	4.06	10.31	41.41

The grain size analysis indicated that the samples collected from slide area belong to silty-clay group. The plasticity index values vary from 18 to 23 %. The average bulk density of the slope material is taken as 1560 kg/m^3.

Rainfall process

In the Malin village there are no rain gauge stations; therefore rainfall data were recorded from different rain gauge stations located around Malin. Figure 3 shows the rainfall data of Malin village from 22nd July to 30th July. In the figure it can be noted that during previous few days i.e from July 22 to 28, the antecedent rainfall was nothing extraordinary. However the rainfall record for 29th July i.e after 168 hours shows high amount of rainfall (108 mm). This may have played a significant role in slope instability.

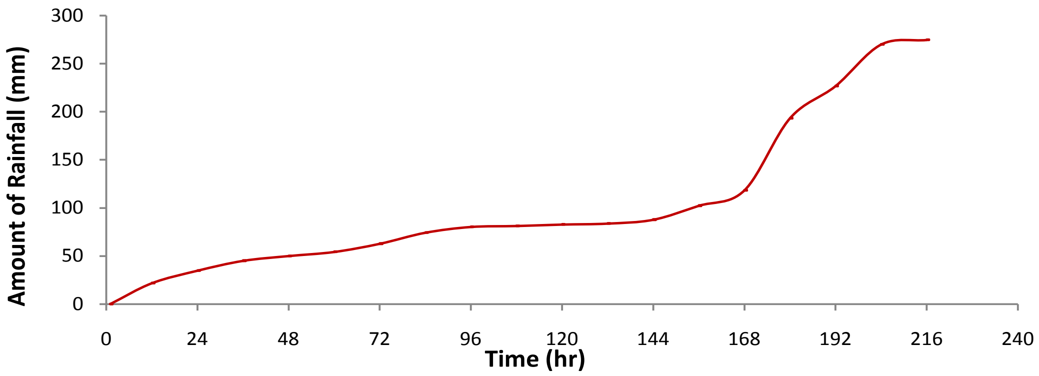

Figure 3: Rainfall data of Malin village

3. Method of Analysis
Back Analysis of the failed slope

A slope failure implies that the factor of safety of the slope at the moment of failure is unity. Based on this information, back analysis is often carried out to improve knowledge on slope stability parameters. Stability parameters may include both soil strength parameters and pore water pressure parameters at the moment of slope failure. The landslide in Malin village provides us with information that factor of safety of the slope at the time of failure is unity and the length of failure surface is 514 m. To achieve a safety value of unity, the soil resistance can be varied which is usually represented by a combination of c' and φ'. Back analysis requires determining appropriate values of cohesion and friction angle which controls failure in the original slope. To determine the shear strength parameters of the slope at failure, the method proposed by Duncan and Stark (1992) is used. This method enables the estimation of friction angle from the known values of plasticity index and hence one can back-calculate the cohesion parameter at the moment of failure. Known parameters: Factor of safety =1, Length of failure surface = 514 m, Plasticity Index = 23. Unknown parameters: Friction angle, cohesion.

Table 3: Typical values of φ' at different values of plasticity index

Plasticity Index	Values of φ' (in degrees)	
	Fully softened	Residual
0-10	30-40	18-30
10-20	25-35	12-25
20-40	20-30	10-20
40-80	15-25	7-15

For the given value of plasticity index (PI = 23), friction angle is expected to in the range of 20-30 degree. Hence a value of friction angle value of 22° for the slope considered in the study. The rotational model is adopted in the analysis. The circular failure surface extends down to the weathered/unweathered interface and has a long radius to represent long shallow failure surface as observed in the failed slope. Bishop's method of slices is applied to evaluate the shear strength parameters of the slope at failure. Knowing the length of failure surface, a slip circle is drawn along the slided slope from the crown in the 4th zone till the 2nd zone. The soil mass above the assumed slip circle is divided into a number of vertical slices of equal width. The number of slices taken is 14 in this case. Failure length = 514 m, Radius of arc taken = 295 m, Number of slices = 14, width of each slice = 25 m.

Considering the whole length of slip surface L as 514 m, the total driving and resisting forces are:

Driving forces = $\sum T$; Resisting force = $\sum c'L + \tan \varphi' \sum N$

Hence, the factor of safety against sliding is

$$F = (c'L + \tan \varphi' \sum N) / \sum T \quad (1)$$

Where, N is the normal component of weight and T is the tangential component of weight. Normal force and Tangential component are calculated asN = W*Cos α and T = W*Sinα.

From the analysis: $\varphi' = 22°$, $\sum N = 86452.17$kN/m, $\sum T = 201075.1$kN/m, L = 514 m, F = 1

Since factor of safety is one, the left hand side of equation (1) equals the right hand side. Assuming the value of φ' as 22° and solving equation (1), we obtained the value of cohesion mobilized along the slip/failure surface as 10.14 kPa.

To validate the results obtained using this limit equilibrium method of slope stability, the stability of same slope is examined using finite difference program FLAC 2D (Fast Langrangian analysis of continua). The geometry of the slope with finite difference grid is shown in figure 4.Mohr coulomb model is used as material model in the analysis. The cohesion and friction angle values considered are 10.14 kPa and 22°. Other properties of soil are taken from table 2. Boundary conditions are such that both the vertical boundaries are fixed in x-direction and the bottom boundary is fixed in both x and y directions. FLAC organizes its zones in a row and column fashion. However, the grid can be distorted to fit complicated shapes. As a general rule, the aspect ratio (i.e height to width ratio) of the zone should be close to unity and the ratio of areas between adjacent zones should be less than 4:1. Considering the above rules, different mesh sizes from coarser to finer grids were used in the problem. It was noted that due to the complexity of slope geometry in our problem, finer grids got distorted under the influence of gravity conditions and could not simulate the physical system. Hence coarser grids with gradual variation in gird size from zone 2 to zone 4 were used to solve the problem. Finer grids were used in zone 4 to ensure maximum accuracy in the displacement and stress values. The number of zones used is 830. The analysis is performed in gravity conditions only. The entire slided slope is divided into 2nd, 3rd and 4th zones, each having different slope inclinations as given in Table 1. Different zones are depicted in different colour shades. FLAC evaluates factor of safety using strength reduction method. It is found that for the combination of shear strength parameters considered, factor of safety evaluated is one. Also the modelled slip surface agrees well with the observed one as the failure surface covers part of 4th zone, whole of 3rd zone and part of 2nd zone as observed in the site. Figure 5 shows the Malin slope at the time of failure.

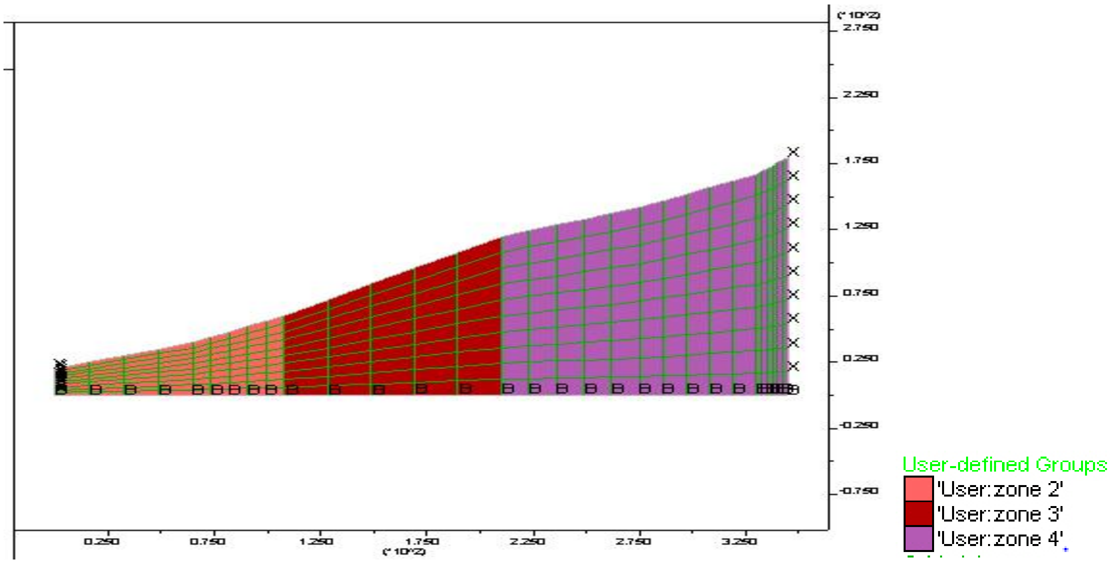

Figure 4: Finite difference grid

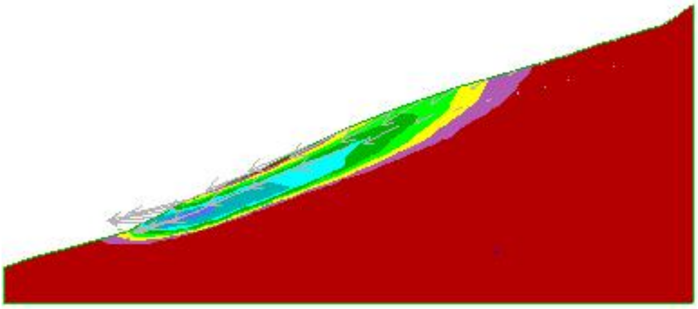

Figure 5: Malin slope at the time of failure (Factor of safety = 1.00)

Analysis performed using FLAC confirms the accuracy of back analysis performed on the failed slope. Figure 6 shows the displacement contours of the slope and it is found that the maximum thickness of slip surface is 10 m. During the site investigation, the thickness of material depleted from various zones during landslide was approximated as 7 m although the value was not measured accurately because when the detailed mapping work was carried out, most of the failed slope material was removed from the place as part of rescue efforts.

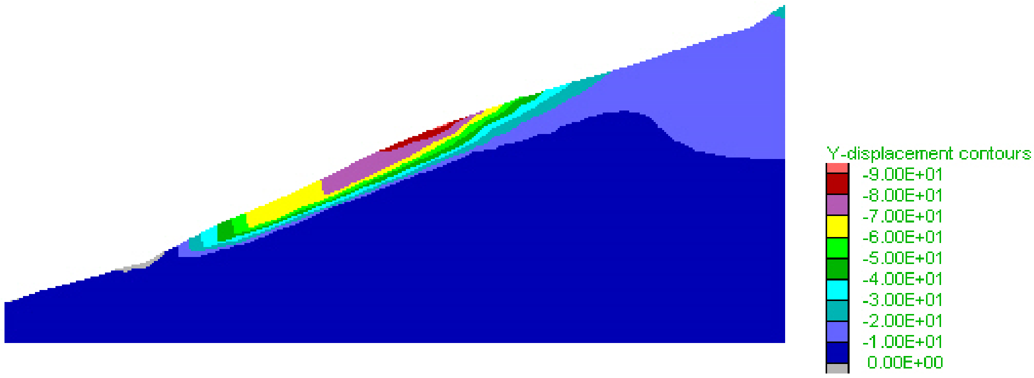

Figure 6: Displacements contours at the time of failure

Using FLAC, one can determine the pre-slide shear strength parameters of the slope. It is understood that stable slopes have factor of safety greater than 1 and in most natural soil slopes the conventional factor of safety assumed for stable slope is 1.5 or greater than 1.5. In FLAC one can perform an analysis to determine the combination of cohesion and friction angle which gives a factor of safety of 1.50 or above. For the same slided slope we have carried out the stability analysis. Mohr coulomb model is used as material model in the analysis. Boundary conditions are same as the previous problem and the analysis is performed in gravity conditions only. The entire slided slope is divided into three zones, each having different slope inclinations and is depicted in different colour shades. From the analysis it is found that for values of cohesion = 36 kPa and friction angle = 22°, the factor of safety of slope is 1.50. Hence the pre-slide strength parameters of the slope are considered as 36 kPa and 22°.

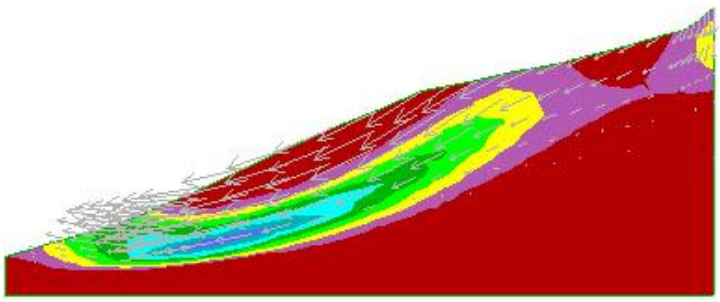

Figure 7: Pre-slide Malin slope (Factor of safety = 1.52)

Figure 7 shows that the pre-slide slope suggested deep rotational slip circle, however the rainfall infiltration into the soil profile decreased the strength parameters and caused shallow slide. Table 4 compares the strength parameters of pre-slide, failure state and post slide state of the same slope where landslide occurred. It is observed that for the same friction angle value of 22° there is a considerable decrease in the value of cohesion.

Table 4: Pre-slide and failure state strength parameters

Parameter	Pre-slide	Failure state	Post-slide
Cohesion (kPa)	36	10.14	38.3
Friction angle (°)	22	22	10.31

It is evident that there has been a decrease in strength parameters from pre-slide state due to failure of slope (parameters of post-slide are taken from Table 2). Excessive rainfall in the area for more than a week has been identified as the triggering mechanism for the failure. Chowdhury and Flentje (2002) observed that high pore water pressures that are generated after prolonged and intense rainfall trigger most cases of significant landsliding. The mobilized shear strength as low as c’ = 0 kPa and φ’ = 15° have been determined from back analyses and for deep-seated landslides, the shear strength parameters determined are c’ = 0 and *φ’* = 9°.SivakumarBabu and Dasaka (2005) showed that decrease of matric suction with time is a time dependent process and evaluated the reliability of a typical landslide in Himalayan terrain. Zhang et al. (2009) performed back analysis of slope failure using probabilistic methods. A cut slope failed due to heavy rainstorm was back-analyzed in a probabilistic way. The mean value of strength parameters determined from the analysis were c’ = 8kPa and φ’ = 38°. Soon Min Ng et al. (2014) investigated the slope failure triggered by rainfall using numerical back analysis method. Back analyses were performed via finite element shear strength reduction method. The results showed that the shear strength parameters at failure were c’ = 11 kPa and φ’ = 20°. The slope consisted of silty soil in large amounts. Hence the results obtained by back analysis based on limit equilibrium and finite difference methods are comparable with the results from Soon Min Ng et al. (2014).

This decrease in strength parameters which resulted in failure of slope can be attributed to the loss of matric suction in the unsaturated part of the slope due to excessive rainfall infiltration. Once the rain-water starts infiltrating the vadose zone, the negative pore-water pressures will tend to dissipate due to an increase in the soil water content. This process contributes to lowering the shear strength of the soil layers close to the surface. Under normal conditions the negative pore water pressure creates suction in the vadose zone and increases the strength of the soil.

4. Role of matric suction in the stability analysis of unsaturated soil slopes

To study the stability of natural slopes subjected to extreme rainfall, one needs knowledge of the pore pressures in normal conditions in addition to those during and after the event. The pore-water pressures, together with the shear strength parameters, govern the stability of a slope. In unsaturated soils, pore-water pressures are dependent upon the flux of water infiltration and upon the hydraulic properties of the soil. The most important hydraulic parameters for unsaturated soils are the storage capacity characterized by the soil-water characteristic curve and the water coefficient of permeability of the soil.

In a saturated soil, the pore space is entirely filled with water i.e the degree of saturation is 100%. But in an unsaturated soil, the volume of water stored within the soil depends upon the negative pressure or suction within the pores. This negative pressure can be varying with time and space in the soil. A function is therefore required to describe the changes in water content related to different suction pressures in the soil. The soil-water characteristic curve (SWCC) can be viewed as the continuous sigmoidal function describing the amount of water in the soil as it is subjected to changes in soil matric suction. The shear strength of a soil is an essential parameter for numerous types of stability analyses. In a saturated soil, the stress state variable is called the effective stress and is often represented by the following:

$$\sigma' = \sigma - \mu_w \qquad (2)$$

Where σ' = effective normal stress, σ = total normal stress, μ_w = pore water pressure. The shear strength of a saturated soil is most often expressed using the Mohr-Coulomb failure criteria and the effective stress concept as:

$$\tau_{ff} = c' + (\sigma_n - \mu_w)_f \tan \varphi' \qquad (3)$$

Where τ = shear stress, c’ = cohesion, $(\sigma_n - u_w)$ = effective normal stress, φ’ = friction angle. Unlike saturated soils, the shear strength of unsaturated soils cannot be described by a singlestress-state variable. Fredlund et al. (1978) justified the need for two independent stress state variables known as the net normal stress (σ-μ_a) and the matric suction (μ_a-μ_w). They proposed that the shear strength of an unsaturated soil could be expressed as:

$$\tau_{ff} = c' + (\sigma_n - \mu_a) \tan \varphi' + (\mu_a - \mu_w) \tan \varphi^b \qquad (4)$$

Where φ^b = angle indicating the increase in shear strength relative to the matric suction and all other parameters have the same meanings. The parameter φ^b is determined experimentally, and is normally found to be somewhere between φ' and φ'/2 (Fredlund and Rahardjo 1993). This angle describes the increase of shear strength due to an increase in matric suction, while the friction angle (φ') describes an increase in shear strength caused by an increase in net normal stress. Hence from equation (4), an unsaturated soil tends to give larger shear strength than that of the same saturated soil. Because it gets strength from three components: cohesion, frictional strength and suction strength.

In this study, the cause of Malin landslide was the rainfall over the previous one week. This antecedent rainfall may have played a role in saturating the slope. Due to increase in saturation, the matric suction in the soil decreased which is clearly evident in the analysis as for the same friction angle value, the value of cohesion decreased from 36 kPa in pre-slide state to 10.14 kPa at the moment of failure. The most convenient form for expressing the decrease in shear strength due to suction is a decrease in the cohesion.

$$c = c' + (\mu_a - \mu_w) \tan \varphi^b \qquad (5)$$

Where c = total cohesion of the soil and all other parameters are the same. Considering that the matric suction has the same effect as that of confining pressure and hence the friction angle with respect to matric suction is the same friction angle. Solving the above equation for c = 36 kPa and 10.14 kPa, the matric suction in the soil is calculated and found that it decreased from 90kPa to 25 kPa. This decrease in matric suction is attributed to the rainfall infiltration. Hence it is important to study the stability of normal slopes affected by rainfall using unsaturated soil mechanics concept which may help in understanding many problems of rainfall induced landslides. Further analysis of the problem stated in this paper is carried out using the concepts of unsaturated soil mechanics and intensity-duration-frequency analysis of rainfall.

5. Conclusions

The catastrophic landslide which occurred along the upslope of Malin village in Pune district of Maharashtra, India in the early hours of July 30th 2014 and claimed the lives of 151 humans and damaged 45 houses was studied. Heavy rainfall over the previous one week was identified as the triggering mechanism of slope failure. Back analysis is performed on the failed slope to identify the cause of slope failure/landslide. Using Bishop's method of slices, the shear strength parameters at the time of failure is determined by assuming factor of safety of unity and known length of failure surface. A combination of cohesion and friction angle values of 10.14 kPa and 22° is reported as the shear strength parameters at the time of failure. To validate these values a numerical analysis was performed using FLAC2D. FLAC2D program is used to analyze the stability of the pre-slide Malin slope. For pre-slide state, cohesion and friction angle values is evaluated as 36 kPa and 22°. Hence there has been considerable decrease in soil strength due to rainfall infiltration. This is due to the loss of matric suction in the unsaturated zone of soil slope which causes lowering of strength of soil. Hence stability of soil slopes subjected to rainfall must incorporate the concepts of unsaturated soil mechanics and rainfall intensity-frequency analysis.

Acknowledgements

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