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Case Study of Landslide in NH –13 at Kethikal Near Mangalore – India

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Abstract

The disastrous slope failure occurred at the Kethikal hill, in the outskirts of Mangalore city in NH 13, India, during the month of June 1998 soon after the heavy and continuous monsoon rains. Many closely built dwelling houses at the top of hill are damaged and the traffic along the road is also diverted for some duration.

Typical stratified slope with three layers is considered for the stability analysis and the general computer program is developed in C language for optimization of factor of safety. The factor of safety is calculated using Janbu's generalized procedure of slices and Davidon-Fletcher-Powel (DFP) technique for optimization. The road is acting as a berm in Kethikal slope. The program gives factor of safety and the co ordinates of critical slip surface. The program is modified to take the effect of tension crack and the effect of berm. The automated stability analysis program gave least value of factor of safety in base failure with tension crack and a berm. The obtained result matches with the field observation. Kethikal landslide is due to the development of high pore pressure in soil.

1. INTRODUCTION

Human settlements and their subsequent developmental activities, especially in the urban and Semi-Urban areas, are drastically changing the landforms and thereby disturbing the original drainage pattern. The changed drainage pattern could be the primary cause for the failure of the soil masses located beneath the natural soil slopes. The result generally is called 'land slide'. This especially is a serious problem nowadays, with rapid urbanization in the hilly and sloping terrain areas. Landslides are relatively rapid down slope movements of soil and rock, which take place characteristically on one or more, discrete, bounding slip surfaces, which define the moving mass. Landslides constitute an important geotechnical problem that involve a variety of geomaterials in a variety of geological and climatic contexts, and which have a major socio-economic impact on civilization. The failure of slopes leads to considerable loss of life and property. It is therefore essential to check the stability of slopes.

Slope stability analysis is essentially a problem of optimization namely the determination of the slip surface that yields minimum factor of safety. Slope section can be analyzed using the generalized procedure of slices developed by Janbu in conjunction with sequential unconstrained minimization technique. This method is capable of locating the critical shear surface corresponding to the minimum factor of safety without putting any prior restrictions on the shape of the slip surface. In this technique the stability problem is posed as an optimization problem wherein the factor of safety is minimized with respect to the co-ordinates of the slip surface and thus critical surface is located.

Typical stratified slope with three layers is considered for the stability analysis. The general computer program is developed in C language for optimization of factor of safety for the selected stratified slope. The factor of safety is calculated using Janbu's generalized procedure of slices. The program uses Davidon-Fletcher-Powel (DFP) technique for optimization. The program gives the minimum factor of safety and the cross section of critical slip surface in three types of slope failures as Base, Toe and Slope failure. The program is modified to take the effect of tension crack existing at the top layer in cohesive soils.

2. STABILITY ANALYSIS

Evaluation of stability is essential prior to any construction involving natural or man made slopes. Various methods are available to evaluate the stability of slopes. There are at present two basic lines of approach in the slope stability analysis, namely the limit equilibrium approach and the stress-strain analysis using the finite element technique. The later approach is a sophisticated one, since it requires very accurate input data. Otherwise the results obtained from such analysis become as doubtful as the input data itself. On the other hand, limit equilibrium approach is relatively simple and has been widely used by practicing engineers and attracted the attention of researchers. Reservations have been raised against the limit equilibrium approach on the grounds that the factors such as slope deformation, the history of slope formation and initial state of stress are not considered in the analysis. Nevertheless, success in the usage of limit equilibrium methods has been rated as commendable. However, the limit equilibrium method has been used over the years because of its simplicity and reasonable accuracy.

Over the years, limit equilibrium methods have been extensively refined by various investigators. Perhaps the most remarkable refinement has come in the form of development of methods which do not require any priori assumption regarding the shape of the slip surface. Some of the widely studied methods in this category are those credited to Janbu (1957, 1973), Morgenstern and Price (1965) and Spencer (1967, 1973). Subsequently, the refinement, which has so far been concentrated only on the method of analysis, has been extended to the search for critical slip surface. It is now well appreciated that limit equilibrium slope stability analysis is a problem of optimization wherein the shape and location of the critical slip surface which yields the minimum factor of safety, are found out. The use of powerful and efficient minimization techniques available in the optimization literature has been a topic of increasing interest among the researchers in the area.

3. CASE STUDY OF KETHIKAL LANDSLIDE

The landslide being discussed in this report occurred on the Mangalore – Hospet Road (NH-13) at Kethikal near Vamanjoor about 20 Kms from Mangalore. There was a major distress associated with the landslide involving more than 60000 m^3 of earth during rainy season of June 1998. The location is in a cutting portion of the steep hillock rising to a height of more than 50 m. The road is in North-South direction (Mangalore towards south) and hillock to the west of the road. The investigation revealed that the failure had a history ever since 1998. It is also noted that a heavy rainfall for 2 – 3 days continuously of intensity more than 1000 mm triggered the landslide.

As a result of landslide a length of 100 m of hill slope including the road suffered a major damage. In the western side of road, the surface sank about 5m with a lateral shift of 6 m and a major heave up to about 2 m. The resultant movement was in NW-SE direction. There was mudflow on the northern hill slope; the area affected being more than a hectare involving more than 60000 m^3 of earth as mentioned earlier.

The detailed observations revealed that near the northern side of the road in the affected portion, the hill slope underwent a vertical slump of about 5 m followed by a lateral mudflow, which pushed the road and the supporting soil by almost 6 m laterally. The most severe damage took place near the location of culvert 1, where the road sunk by about 6 m. Further the mud slide had resulted in to a heaved soil mass on the southern side of slope as shown in Figure 1(a) and (b).

4. CROSS SECTION OF SLOPE

The road is formed on the top of lithomargic clay, which has a tendency to lose much of its shear strength on coming in contact with water, and is highly erodible. This lithomargic clay is locally called as 'shedi soil'; unconsolidated undrained triaxial shear test gave considerable reduction in shear strength parameters on saturation.

The cross section of slope as shown in Figure 1(a) and (b) indicates that the road is present in between the slope. The slope can be taken as a slope with presence of horizontal berm of width about 12 m. The slope has upper slope angle of 65 to 70 degrees and lower slope angle of about 55 to 60 degrees. The height of slope above the berm is about 12.0m and below the berm is continuous and for the study it is taken as 15m. At the top of slope there exists a gentle slope of about 16 to 18 degrees with horizontal. In the analysis it is taken as additional vertical load. There were some small residential buildings at the top of hill. To simulate their effect in the analysis an additional surcharge load is also considered to be acting over the sliding wedge.

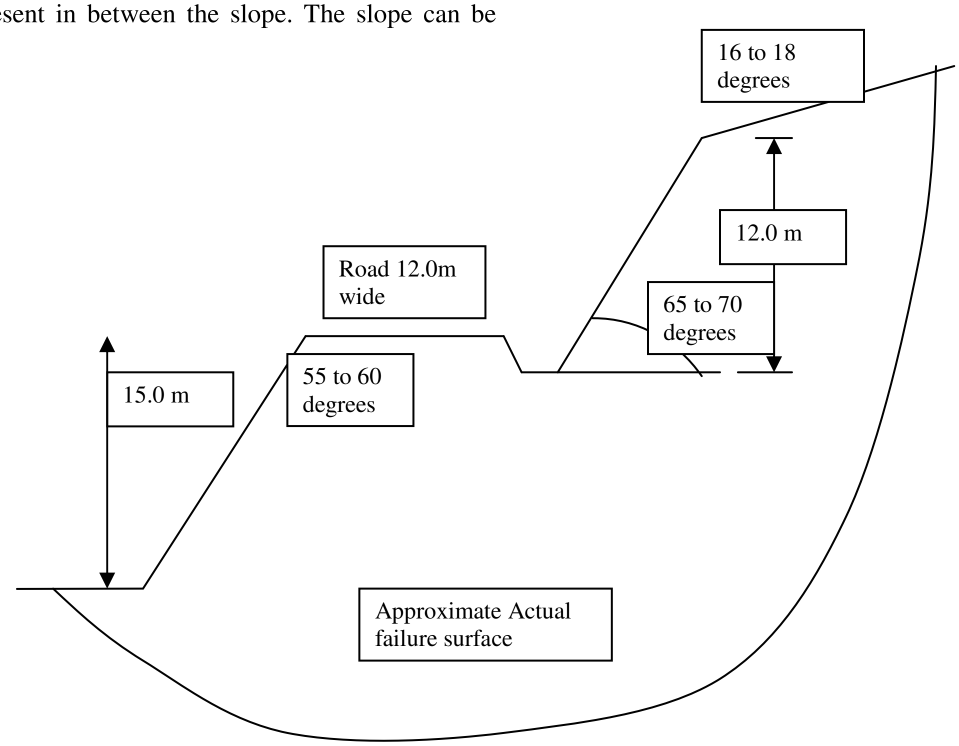

Fig 1(a): Cross section of slope with approximate actual failure surface.

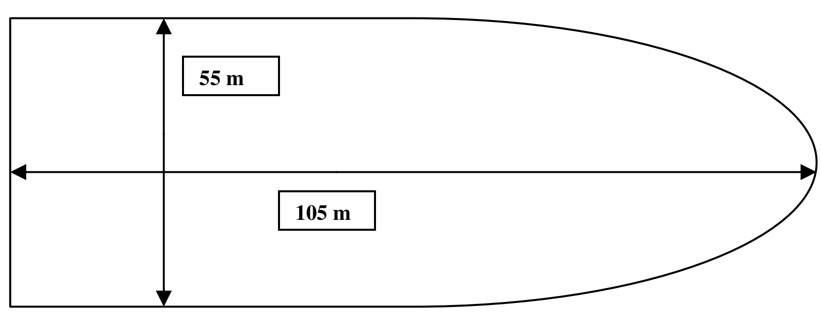

Fig 1(b): Plan of Failure wedge.

5. GENESIS OF DEVELOPMENT OF FAILURE:

The study of records and local enquiry revealed that the failure had a history. In the site, there had been sinking of road surface associated with formation of cracks during rainy season for more than 10 years. They were too small in nature and routinely attended. The movement became prominent in the rainy seasons of 1996 and 1997. During 1996 the width of crack was in the range of few centimeters and in 1997 it grew larger up to nearly a 1.0m.

YEAR 1996: The crack on road surface at two places was in the range of 5 – 6 cm.
YEAR 1997: A major crack was developed on the road. The width initially about 50 mm enlarged to 1100 mm in 5 – 6 days. Road sunk by about 1 m towards eastern side of crack. At 25 m east of first crack another crack of size 750 mm was also observed. A landslide occurred at 150 m east of first crack. Northern hill slopes developed significant cracks, deep cuts and slips. The residential houses at the top were evacuated. The landslide was attended. The failed soil mass was removed and dumped at South – Eastern side of slope.
YEAR 1998: There were two cracks at the same place. The first crack grew in size more rapidly than the second crack. The road surface sunk by 3 m over 6 days and 5 m over a month. Lateral shifting of the road was about 6 m. There was a Landslide resulting in mudflow. It had a slump of 5 m at northern side and a heave of 3 m at the eastern side. The Mangalore tiled bus shelter was moved laterally by about 3 m without any distress to the structure. The culvert was also moved by about 2 m. The longitudinal drain was completely damaged and laterally shifted. The soil mass was lifted and dumped at the eastern side, about 100 m away from the slide. The new road surface was prepared and a new culvert was built. A series of stone piles on either side of the road surface are installed up to the hard rock level.
YEAR 1999: There were cracks of 5 –25 cm in width and of length 75 –100 cm. There was no significant movement of soil mass.
YEAR 2000: There were cracks of 5 –20 cm in width and of length 50 –75 cm. There was no significant movement of soil mass.
YEAR 2001: The upstream side had some movement.
YEAR 2002: There were no appreciable movements.
YEAR 2003: The soil mass on the hillside of the road surface had moved down by about 2 m till the edge of road. This soil mass was cleared and there was no damage to any structure.
YEAR 2004: The soil, which is failed in the year 2000, had moved down by 3 m. The road has developed an upward projection due to the pushing of the soil. The soil is removed from the edge of road.

It is observed that the development of cracks has close relationship with the rainfall as seen from Table 1, which gives the details of amount of rainfall and development of cracks.

Table 1: Development of cracks with Rainfall data.

Dates & Year	Cumulative rainfall up to dates in col.1, from start of raining season mm)	Average rainfall during the dates in col.1 (mm/ day)	Length of crack (mm)	Vertical movemet of failure Wedge (mm)
10–14 July 1997	1410	101	760	600
27 June 2 July 1998	1300	85	3100	3100
10-14 June 1999	580	63	120	350
29 June 10 July 2000	120	94	60	305

6. FIELD INVESTIGATIONS:

In the sliding area, at western side, hillock rises to a height of about 30 meters above the road surface over a distance of 80 meters. The eastern side of the slope dips to a depth of 20 – 30 meters over a distance of 50 meters. A length of 100 meters of road suffered a major damage. The minor damage was seen over a length of about 200 meters.

The Figure 2 shows the bore log positions and bore log details of the area. The borehole 1 and 2 drilled at the western side of the road and borehole 3 drilled at the eastern side. The weathered rock is available at ground level at the top of ridge in western side of hill. The bore log of BH1 indicates the presence of 4.5m of depth hard laterite at top, 9.0m of depth shedi soil, 3.3m of weathered rock and then hard rock. The bore log of BH2 indicates the presence of 5.3m of depth hard laterite at top, 3.7m of depth soft laterite soil, 8.0m of shedi soil, 1.5m of weathered rock and then hard rock. The bore log of BH3 indicates the presence of 6.0m of depth Shedi soil at top, 2.5m of depth weathered rock, 2.0m of disintegrated rock and then hard rock.

Abandoned laterite quarries are also found on the western part at the top of the hill. These quarries are filled with water in rainy season. The road is formed in the top of lateritic material. Soil below is clayey sandy silt which is essentially Kaolinitic in nature, locally called as 'Shedi Soil'. The type of soil wedge movement indicates that there was a base failure. The width of failure wedge is about 55.0m and the length of failure wedge is about 105.0m as shown in Figure 1 (b).

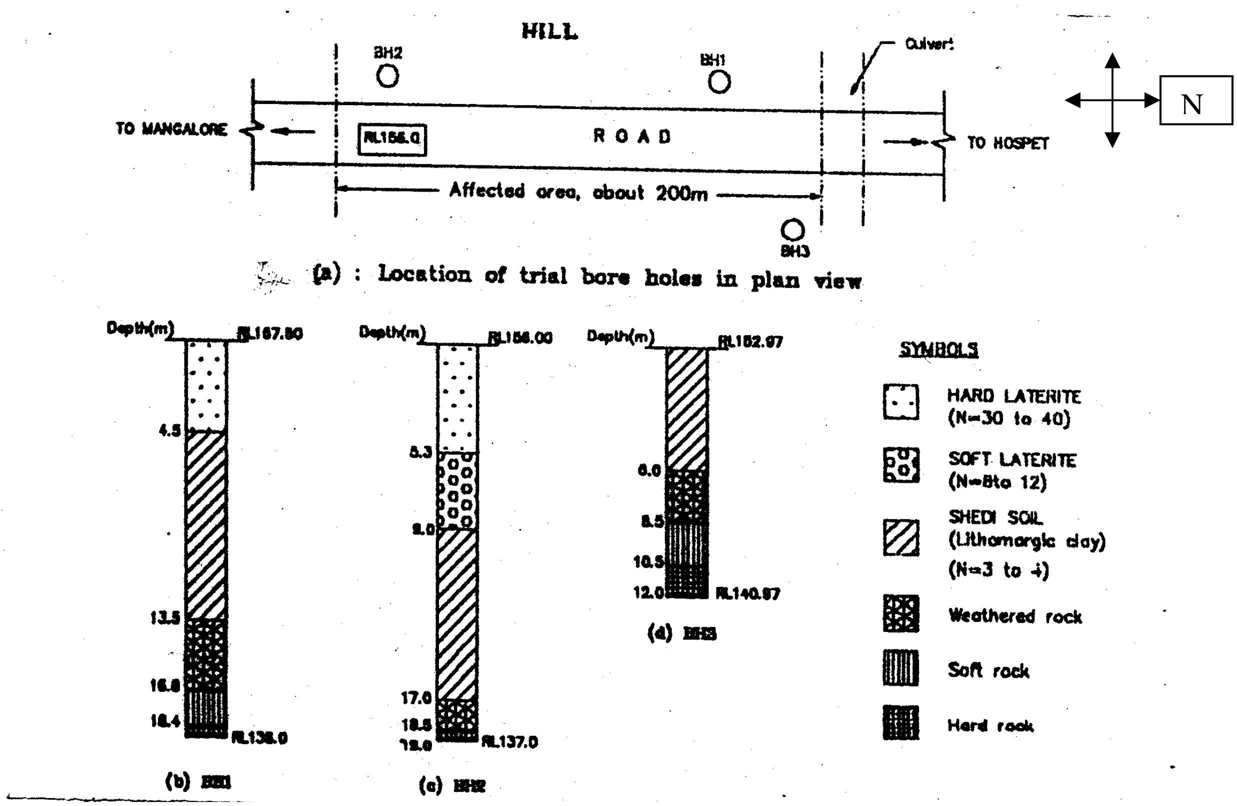

Fig 2: __Borehole details at Kethikal.__

7. LABORATORY TESTS AND RESULTS:

The samples collected are divided into three types

1. Disturbed samples.
2. Undisturbed Vertical Samples.
3. Undisturbed Horizontal Samples.

The samples are tested in two different sizes as given below.

1. Small size Samples of 37 mm diameter and 76 mm height.
2. Large size Samples of 76 mm diameter and 160 mm height.

The samples are collected during the month of November using thin tube samplers in both horizontal and vertical directions at a depth 1 meter. The samples are tested for their index properties and engineering properties, especially shear strength parameters. The Table 2 gives the results of in situ properties of the soil Samples. The Table 3 gives the results of identification and classification of soil samples taken from Kethikal landslide.

Table 2: Results of In situ properties and classification of the soil Samples at Kethikal.

Sl. No.	Field Unit weight (kN/M ^{3})	Specific Gravity	Dry Unit weight (kN/M ^{3})	Liquid Limit	Plastic Limit	Shrinkage Limit	%Passing in 75µ Sieve	Group Symbol
1.	17.19	2.54	14.68	33.00	30.59	26.89	50.49	**SL-ML**
2.	17.41	2.53	14.40	43.50	40.63	28.41	44.73	**SM**
3.	19.75	2.57	11.86	37.30	35.27	29.34	50.56	**SI-MI**
4.	17.85	2.60	16.21	35.60	31.90	30.11	59.13	**MI**
5.	17.05	2.56	14.59	34.50	31.52	25.79	62.24	**ML**
Ave	**17.85**	**2.56**	**14.35**	**36.78**	**33.98**	**28.11**	**-----**	**--**

A strain controlled triaxial test was carried out under unconsolidated undrained condition. The sample was allowed to saturate by connecting through a standpipe and supply of water from bottom of sample. The porewater pressure is also measured at 100% saturation. The Table 4 gives the results of shear test of saturated soil samples.

Table 4: Results of shear test of saturated soil samples.

Type of Sample	Small size sample		Large size Sample	
Shear strength parameters	Unit cohesion (kN/m^2)	Unit cohesion (kN/m^2)	Unit cohesion (kN/m^2)	Angle of internal friction (Degrees)
Remolded Sample	30	20	20	17
Undisturbed Vertical Sample	50	16	16	12
Undisturbed Horizontal Sample	60	12.5	12.5	11

8. AUTOMATED STABILITY ANAL;YSIS USING THE OPTIMIZATION PROGRAM:

Slope stability analysis is essentially a problem of optimization namely the determination of the slip surface that yields minimum factor of safety. Slope section can be analyzed using the generalized procedure of slices developed by Janbu in conjunction with sequential unconstrained minimization technique. This method is capable of locating the critical shear surface corresponding to the minimum factor of safety without putting any prior restrictions on the shape of the slip surface. In this technique the stability problem is posed as an optimization problem wherein the factor of safety is minimized with respect to the co-ordinates of the slip surface and thus critical surface is located.

8.1 Formulation of the problem:

For the given geometry of the slope and soil properties the factor of safety is a function of shape and location of the potential slip surface. The problem is to determine the shape and location of the shear zone that yields the minimum factor of safety.

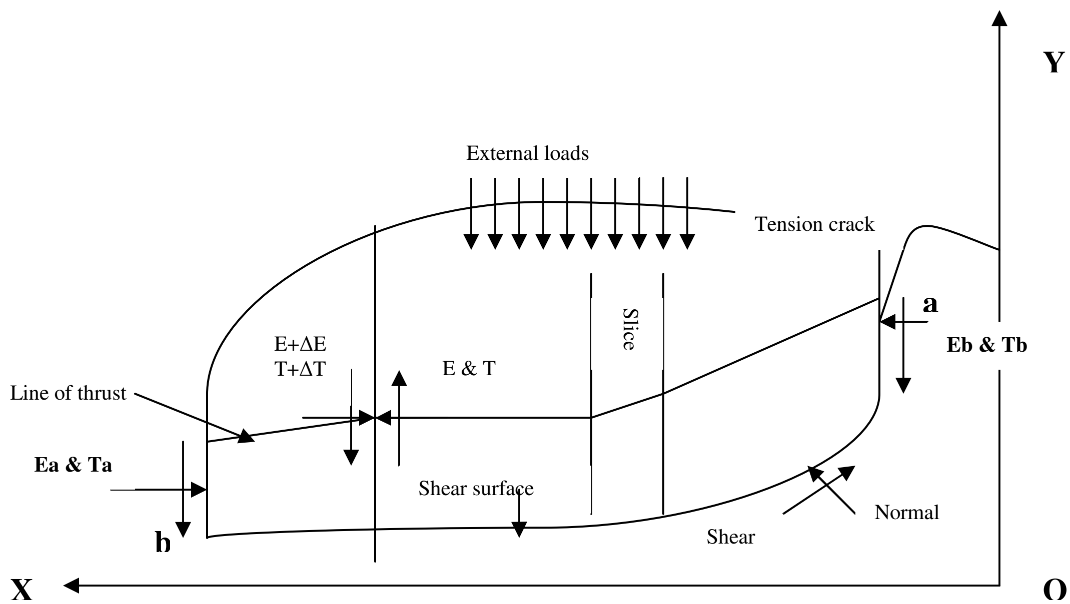

Fig 3: Definitions and Notations used for the generalized procedure of slices.

8.2 Design variables and objective function:

The co-ordinate system chosen is as shown in the Figure 3. Let X[0], X[1], X[2],……X[6] be the design variables of a general potential slip surface. Given the positions of the initial design variables, respective X and Y co-ordinates of all the points which completely defines the slip surface can be calculated, since thickness of each layers, width of slice and number of slices are known. With this, the co-ordinates of all the points along the slip surface are completely defined and the factor of safety can be expressed as a function of these co-ordinates.

In the adopted procedure, the factor of safety is minimized with respect to these design variables. Once the optimal design vector i.e. D = f(X[0],X[1],X[2]……X[6]) is found out, the design vector along with the X co-ordinates of the slice interfaces will define the actual critical shear surface. Thus determination of the critical surface involves the optimization of the factor of safety, now treated as an optimal function F, with variables as geometrical co-ordinates defining each admissible slip surface. Consequently, optimizing the factor of safety function will yield the minimum factor of safety and the corresponding set of co-ordinates defining the critical slip surface.

The objective function is the factor of safety and an expression for the same can be obtained from the Janbu's method. The factor of safety can be expressed in terms of the design vector as
F = f(D) = f(X[0],X[1],X[2],…..X[6]).

8.3 Optimization formulation:

We are now in a position to state the problems of finding the critical slip surface and the corresponding the minimum factor of safety as a mathematical programming problem as follows.

Find the design vector d such that F = f(D) is minimum of f(D) subject to

G_j (D) <=0;

Where j = 0, 1, 2,…….(M-1), where M is number of constraints. The best way of deciding the initial design vector is to choose the initial slip surface in such a way that a major portion of it lies along the existing shear plane. If the initial design vector is chosen in any other way then during the optimization process it is likely that the weak shear plane will be bypassed and the obtained solution would not be the global minimum. This is the major disadvantage with this type of analysis.

In the present analysis, one must be careful in choosing the penalty parameter(R); it should be chosen initially in a manner such that the objective function and penalty term have equal weightage. If it is not done, there is possibility of increase in the objective function value during the optimization process even though the penalty function gets minimized. This will result in lifting off the shear surface plane and it will be almost impossible to hit back the same shear surface again and the solution converges to local minimum. Moreover, the optimization scheme did not have the freedom to move away from the constraint surface i.e. the obtained surface depends on accuracy in selecting the constraints.

8.4 Minimization procedure:

The sequential unconstrained minimization technique using the interior penalty function formulation coupled with Davidon – Fletcher – Powel variable metric method (DFP) and cubic interpolation method of one dimensional search for linear minimization. Interior penalty function needs feasible point for starting the solution. As in this it is possible to get a feasible initial design vector, the interior penalty function method has been chosen for the analysis. The basic object of the penalty function method is to convert the original constraint problem in to one unconstrained minimization by blending the constraints in to composite function (Ø). The detailed backgrounds of these methods are available in any standard text books on optimization. For problems with inequality constraints only

$$\Psi (D,r_k) = F(D) - r_k \sum_{j=0}^{M-1} (1/g_j(D))$$

Where, F is to be minimized over all D, satisfying

$g_j(D) <=0; J = 0,1,2,\ldots\ldots\ldots.(M-1)$

The penalty parameter r_k is taken as 1.0 in the present analysis. A computer software in C language is developed to search the shear failure plane and is associated factor of safety of the given slope section. In order to ensure that the slip surface is physically reasonable and acceptable the following constraints are imposed on the shear surface.

8.5 Details of computer program

The rapid development of computers has completely revolutionized research and practice in every scientific and engineering field. The process of solving the slope stability problems is essentially an interactive process and time consuming. Hence development of program is advantageous for an automated slope stability analysis and is presented in the following section.

The general computer program in FORTRAN language for solving any type of nonlinear mathematical problem was developed by Rao (1989). This program is modified and developed in C language for calculating minimum factor of safety of three layered slope section in association with Janbu's method (1973) of analysis. The solution procedure is based on the entire penalty function method coupled with Davidon-Fletcher-Powel method of unconstrained minimization and cubic interpolation method of one dimensional search. This program is applicable to much type of slope stability problems i.e. for homogeneous and non homogeneous slopes for finding minimum factor of safety. It consists of main program and five functions as explained below. The program can evaluate two types of slope as

1. Slope with absence of berm.
2. Slope with presence of berm.

It consists of main program, three main functions and five sub functions in each of the main function. The three main functions which is used to evaluate three types of slope failures based on the location of slip surface are

1 Base failure where failure surface is passing through bottom layer of slope and a point beyond toe.
2 Toe failure where failure surface is passing through bottom layer and toe.
3 Slope failure where failure surface is above toe of slope.

The above three main functions are combined in to main program which calls each of the main function and gets the factor of safety. Three functions returns three factor of safety in three types of failures and the main programs selects the least among them. Similarly another program is developed which takes care of tension crack at the top layer of slope. The main function has five sub functions and the main function performs the function of organizing the following set of five sub functions to get the final solutions i.e. calculation of factor of safety of critical slip surface. The set of five sub functions perform specific work and they are;

1 UNCON: For implementing the Davidon – Fletcher – Powell method of unconstrained minimization.
2 ONEDIM: For implementing the cubic interpolation method of one dimensional search.
3 GRADT: For evaluating the gradient of function using forward finite differences method.
4 FTN: For providing the values of Φ, f and G_j, j = 0, 1, 2… M-1 corresponding to any design vector X. It also performs automatic calculation of input data and expressing objective function and constraints in terms of design variables (X [0], X [1], X [2]……..X [N-1]).
5 PRESSURE: For the given slope cross section it calculates the total earth pressure at the bottom of each slice and also the porewater pressures of each slice. The function returns the pressure values in an array called PR [ ] and pore water pressure in U [ ].

In each case the effect of tension crack is also considered in a separately. The output obtained by the program is validated with the factor of safety of slopes as published in the literature. The output is also validated using Plaxis 8.0 which is a finite element program for Geotechnical applications.

8.6 Stability analysis using the developed computer program

Figure 1(a) shows the cross section of slope at Kethikal can be a slope with berm. The slope has upper height (HU) as 12m and the height of lower slope (HL) is taken as 15m. The angle of slope is taken as αU at above the berm and αL at below the berm in degrees and HU and HL as the vertical height of slope at upper part and lower part respectively. In all the three boreholes indicates the presence of three soil strata at Kethikal. Hence the developed program can be applied to evaluate the stability analysis of the slope. The bore log details are used in arriving at the thickness of each layer. The bottom layer is of thickness HL1 = 0.25*(HU+HL), with soil parameters cohesion as c1, angle of internal friction as Φ1, and density of soil as γ1 etc. Similarly the middle layer and top layer are of thickness HL2 = 0.50*(HU+HL) and HL3 = 1.25*(HU+HL) with corresponding soil parameters are considered. The length of horizontal plane at the top of slope and at the toe of slope is taken as equal to two times the height (HU + HL) of slope.

The failure surface on field indicates that there is a presence of tension crack at the top layer. The portion of mass which is remained there had a vertical profile at top. There were number of abandoned laterite quarries present at the top of hill which help in saturating the soil and also in the development of pore pressure. The unconsolidated undrained triaxial test indicates a pore water ratio of about 0.28 in the top layer of soil. The index properties are selected from the results of laboratory tests. The list of the different input values which are used in the program are shown in Table 5.

Table 5: Input values to the program:

Sl. No.	Properties	Values
1.	Unit weight of bottom soil layer	17.85 kN/m^3
2.	Unit weight of middle soil layer	0.95*17.85 =16.96 kN/m^3
3.	Unit weight of top soil layer	0.90*17.85 = 16.07kN/m^3
4.	Unit cohesion of bottom layer	60.0 kN/m^2
5.	Unit cohesion of bottom layer	0.75*60.0 = 45.0 kN/m^2
6.	Unit cohesion of bottom layer	0.50*60.0 = 30.0 kN/m^2
7.	Angle of internal friction of bottom layer	3.0 degrees
8.	Angle of internal friction of middle layer	2.0*3.0 = 6.0
9.	Angle of internal friction of top layer	3.0*3.0 = 9.0 degrees
10.	Upper slope angle (AU)	65.0 degrees
11.	Lower slope angle (AL)	55.0 degrees
12.	Upper height of slope (HU)	12.0 m.
13.	Lower height of slope (HL)	15.0 m.
14.	Depth of bottom layer (HL1)	0.25*(HU + HL)=6.75m.
15.	Depth of middle layer (HL2)	0.50*(HU + HL)=13.5m.
16.	Depth of top layer (HL3)	1.25*(HU + HL)=27m.

17.	Pore pressure ratio (ru)	0.28
18.	Width of berm (B)	12.0 m.

These input parameters are used as input to the program with presence of berm and with the presence of tension crack and absence of tension crack. The results of stability analysis using the developed program are tabulated in Table 6.

Table 6: Results of stability analysis of Kethikal Landslide.

Sl. No.	Type of slope	Value of factor of safety
1.	Slope with presence of berm and absence of tension crack – Base failure.	1.1216
2.	Slope with presence of berm and absence of tension crack – Toe failure.	1.5947
3.	Slope with presence of berm and absence of tension crack – Slope failure.	1.3473
4.	**Slope with presence of berm and presence of tension crack – Base failure.**	**0.9357**
5.	Slope with presence of berm and presence of tension crack – Toe failure.	1.7555
6.	Slope with presence of berm and presence of tension crack – Slope failure.	2.7378

From the above table it clearly shows that the minimum value of factor of safety is in base failure with the presence of tension crack. From the field observation the type of failure is base failure at Kethikal land slide. The program gave the co-ordinates of failure surface which is as shown in Figure 4.

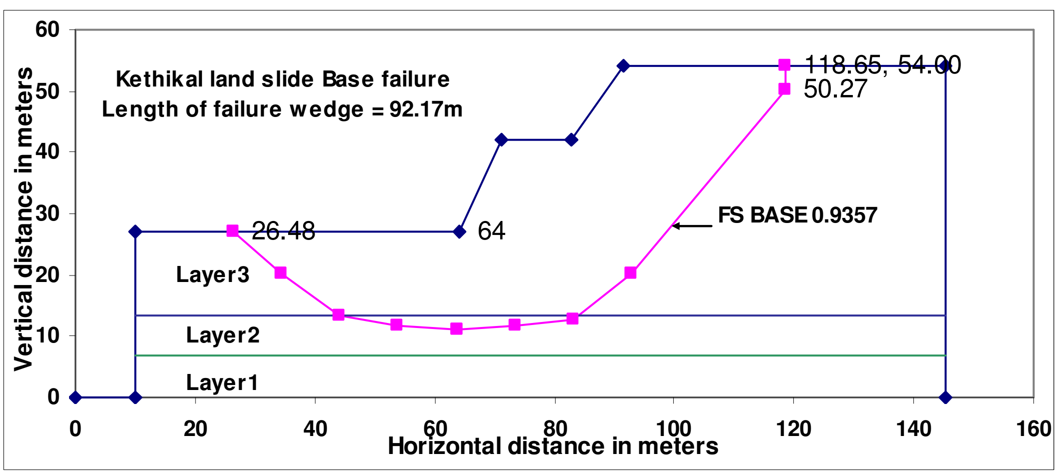

Fig 4: Failure surface in Kethikal land slide - Base failure in slope with presence of berm and tension crack.

9. INFERENCES FROM THE CASE STUDY OF KETHIKAL LANDSLIDE:

The land slide at kethikal was analyzed using the optimization program using generalized procedure of slices for stratified soil slopes. This slope is taken as a slope with berm which houses a road. The road is formed on the top of lithomargic clay, which has a tendency to lose much of its shear strength on coming in contact with water, and is highly erodible. The program is used to evaluate the factor of safety in three cases of failure and is compared with the field observations. Based on the stability analysis following observations are made.

1. The automated stability analysis optimization program using generalized procedure of slices gave least value of

factor of safety in base failure with tension crack. The obtained result matches with the field observation i.e. there was an upward movement of soil mass at the base of slope.

2. The program gave the length of failure wedge as 92.17m and the actual length of failure wedge is about 105.0m.

3. The stability analysis used the value of depth of tension crack at top layer is 3.73m which can be compared with the actual depth of tension crack as observed at site which is around 4.0m.

4. Kethikal landslide is due to the development of high pore pressure in soil. The development of high pore pressure is assisted by continuous rainfall for 4 to 5 days which resulted in triggering the movement. The presence of abandoned laterite quarries at top of hill which gets filled during the rainy season is also the additional factor in developing the high pore water pressure.

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Fig5: View of Kethikal land slide from the road level.

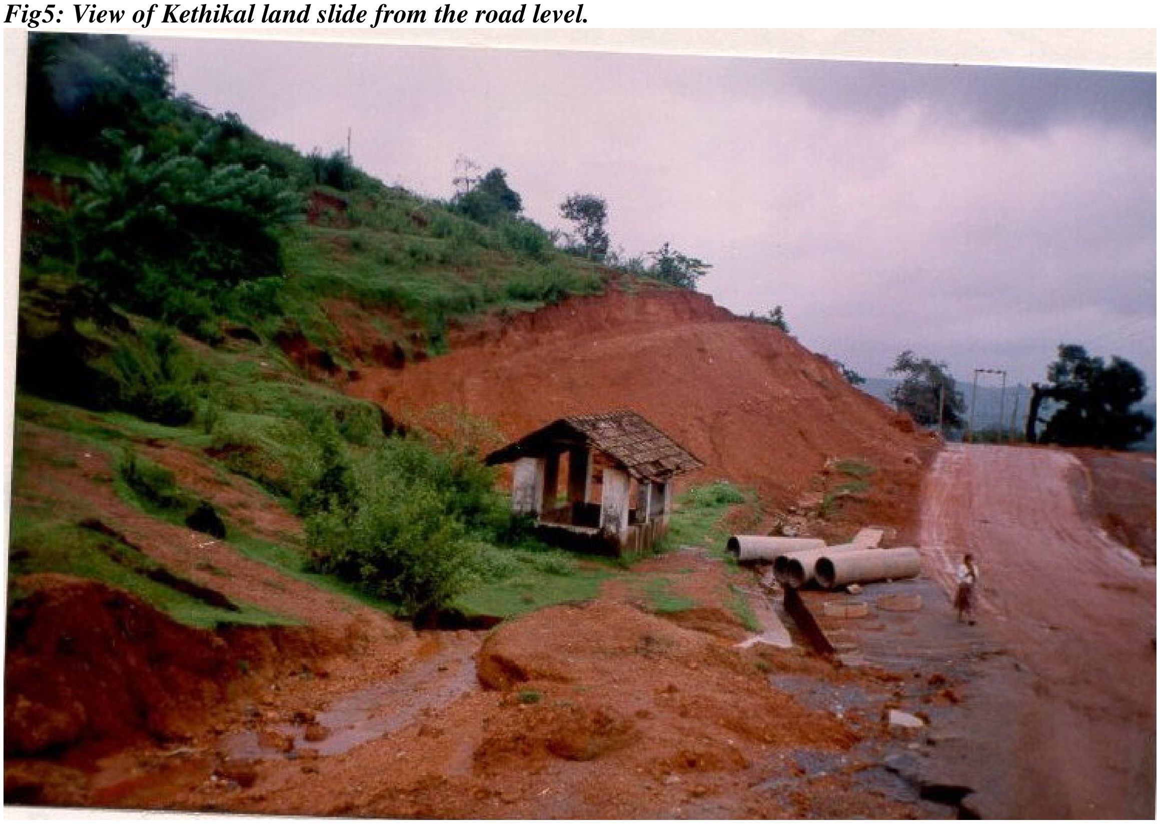

Fig 6: Another view of Kethikal land slide from the top of hill.