Seismic Stability Analysis Of Unreinforced, Reinforced Embankments and Earth dams Using Rocscience Slide 6 software

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CERTIFICATE

This is to certify that the project titled **Seismic Stability Analysis Of Unreinforced, Reinforced Embankments and Earth dams Using Rocscience Slide 6 software** is carried out by

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Abstract

The analysis of seismic stability of earth dams has received large attention, in this work by applying fellinius method presented by the computer program rocscience slide 6 software to define potential slip surface and calculate the factor of safety of Earth dam (Mandali dam in iraq) under four main conditions steady state seepage downstream, full reservoir conditions, partial drawdown condition 3m drawdown, 6m drawdown, 9m drawdown by considering various seismic coeffecients (0.02, 0.05, 0.1) allowing the user to obtain the minimum factor of safety of an earth dam immediately, based on the height and slope of the dam, as well as properties such as cohesion, angle of internal friction, permeability coeffecient, and unit weight of materials used..

Keywords: Slope stability, Siesmic coefficient, Fellinius method, Factor of safety.

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CHAPTER 1

Introduction

1.1 Earth dam

A dam is a hydraulic structure composed of impermeable material that is built across a river or stream to form a reservoir on the upstream side of the river or stream for water storage for various reasons. Earthen dams are preferred over concrete gravity dams due to their ease of construction and lower cost. Locally available materials and lower-skilled labour reduce construction costs, hence it is still widely used. Dams have played a significant and critical role in human and societal growth. Dams are important for flood control and give considerable benefits in the fields of renewable energy and agriculture.

1.2 Earth Dam Failures

- Structural failures.
- Hydraulic failures.
- Seepage failures.

1.2.1 Structural Failures.

1. Failure due to pore pressure: Impervious compressible soils have an extremely slow drainage capacity. Excessive pore pressures develop in the soil during and immediately after the construction of earth dams made from such soils. The permeability of the soil determines the amount of pore pressure present.The lower the permeability, the higher the pore pressure. If the permeability of the soil used in the dam's construction is very low, there may not be a significant drop in pore pressure in the core zone of the dam by the time the dam's construction is completed.

2. Sudden drawdown on the upstream face: When a reservoir is suddenly emptied, the water pressure on the water face decreases. However, due to the suddenness of the operation, the saturated soil, which had been filled with water before it was emptied, does not have enough time to release water and produce equilibrium conditions.This phenomenon is known as sudden drawdown.Drawdown reduces the hydrostatic pressure caused by reservoir water, leaving no resistance pressure created by water retained in the soil. The upstream face slides due to the unbalanced outward pressure caused by water held in the soil of the upstream face. Although such a slip of the upstream face does not usually result in dam failure, it can clog conduit outlets and cause problems. This feature eliminates the sloughing and sliding process from continuing, increasing the risk of failure.

3. Down stream slope slide: When the reservoir is full and the rate of percolation is at its highest, the risks of downstream slope sliding are maximum.Because of this, the earthen dam is prone to failure. The slips that may occur on the dam's downstream side could be either deep or shallow. Shallow slides only occur on sandy soils and never reach deeper than 1 to 2 in the direction of slope. In clayey dams and clayey foundations, deep slips occur. When a deep slide occurs, pore pressure is not released, and the unstable vertical slide slip is left standing, often forming sloughs or slides.This slide process continues until the dam approaches the point of failure. The dam will now be failed due to a single strong wave of water.

4. Foundation slide: If the dam is built on fine silt or soft soil, the entire structure may collapse. The presence of soft, weak clayey soil, as well as silt and sand, can both lead to foundation failure.Expansion of soil on saturation may cause lifting of the slopes and thus may cause failure of foundation.

5. Failure by spreading: When a dam is built on stratified deposits with layers of soft clay, it is possible for the fill to fail due to spreading.

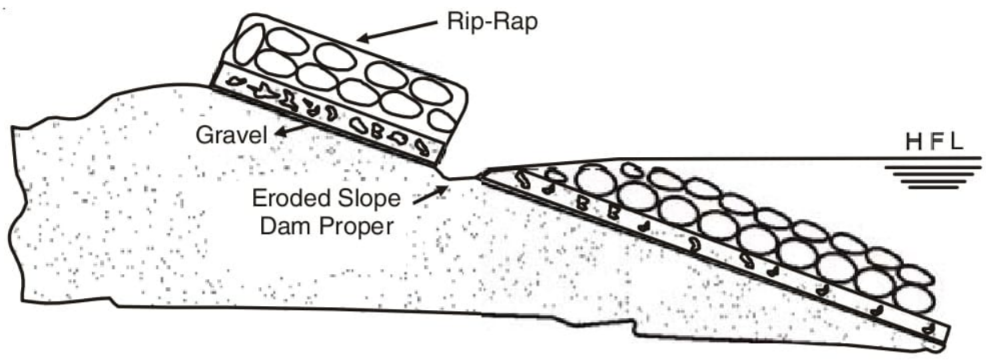


Figure 1.1: Failure of riprap

6. Slope protections failure: Pitching or riprap protects the dam's slopes in most cases. Pitching, also known as riprap, is a layer of huge rocks that acts as a protective barrier. This layer is usually placed on top of a gravel or filter blanket layer. The riprap may be loosened as a result of repeated impacts with water waves at the reservoir's level. This exposes the embankment to erosion. The failure of the rip rap may ultimately lead to failure of the dam.

7. Failure due to holes caused by burrowing animals: Burrowing animals may dig holes in the dam or foundation, causing the dam to fail due to pipe failure. In most cases, such animals will not dig in moist soils below the seepage line. Burrowing animals may honeycomb the dry upper part of the dam if the water level in the dam is very low for a lot of years. When the water level rises, the water may escape through the honeycombed openings, causing the dam to fail.

8. Failure due to earthquake: The following factors may contribute to the failure of an earthen dam as a result of earthquakes. The core may develop cracks, resulting in leakage and eventual failure. Shaking action may cause the dam or its foundation to settle. This could result in the dam being overtopped.
   Hilltops sliding into the reservoir, forcing the reservoir level to rise.This occurrence could result in the dam being overtopped once more. The horizontal component of the earthquake's acceleration force may produce shear slip of a significant amount of the dam's slope, resulting in dam

failure.

9. Failure due to soluble material in dam: If some material which is soluble in water, is present in the dam or in its foundation, it may cause failure of the dam. Such materials are washed away with time and may cause settlement of the dam and thus failure. Such materials if get deposited in the filters, designed for drainage purpose, may cause clogging of the filters and this may ultimately lead to dam failure, as drainage of the dam will not be proper in that case.

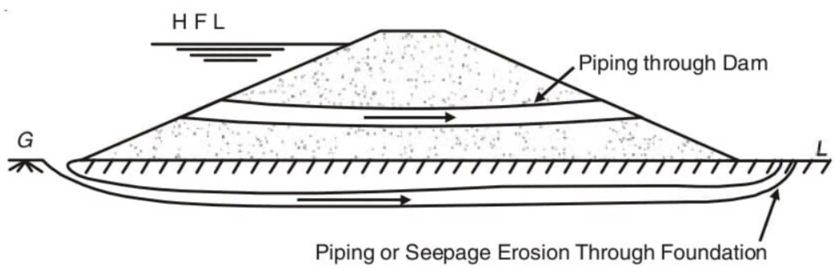


Figure 1.2: piping failure

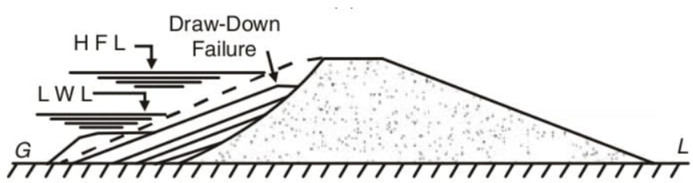


Figure 1.3: drawdown failure

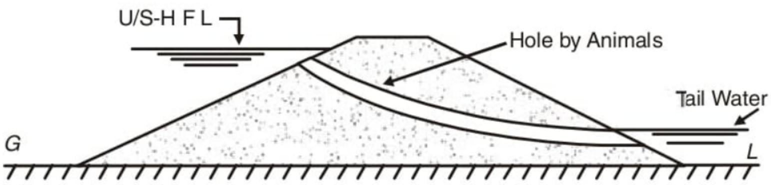


Figure 1.4: Holes by burrowing animals

1.2.2 Hydraulic Failures.

1. Overtopping: If amount of flood has been under estimated, the capacity of the spillways will not be adequate. This may lead to the rise in

water level in the reservoir above the maximum estimated level. This may ultimately lead to the over topping of the dam.

2. Wave erosion: The upstream slope will be eroded if the face is not appropriately protected by riprap. When the wave returns, the roller action in the soil is set, and it is simply scraped out. If the wave erosion action is not stopped in time, it may result in the scooping out of the entire free board soil, causing the dam to fail. Upstream slippage and subsequent failures can also be caused by waves.

3. Toe erosion: If water from the downstream side of the dam comes into touch with the toe of the dam, toe erosion may occur. A thick riprap on the downstream toe of the dam, up to a height slightly above the tail water level, can be used to prevent failure owing to this reason.

4. Gullying. Heavy rain has caused gullying collapse. Gully formation can be avoided by turfing or constructing counter berms on the dam's downstream slope. A good drainage system on the dam's downstream side helps greatly in preventing this failure.

1.2.3 Seepage Failures.

Piping: Piping is nothing but process of progressive erosion of concentrated leaks.The problem with the piping could be in the dam's body or foundation. The water leaking through the dam's body has the following negative consequences. Seeping water creates an erosive force that dislodges soil particles from the soil structure and causes voids between bigger grains to reorganise. The seeping flow, combined with the differential pore pressure it creates, can elevate soil particles, resulting in boiling. Internal soil erosion begins at the place of exit and gradually moves backwards. A conduit or pipe is produced through the earth as the concentration of the leak increases with more and more movement backwards. The process of forming this conduit is known as piping. Sloughing: The phenomena of sloughing is closely related to the piping process. When the reservoir is full, the dam's toe is usually soaked. The toe may deteriorate, resulting in a minor sliding. This slide makes

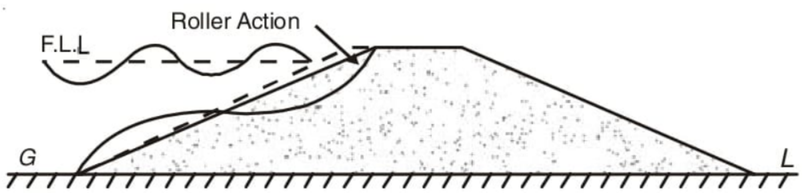


Figure 1.5: Roller action

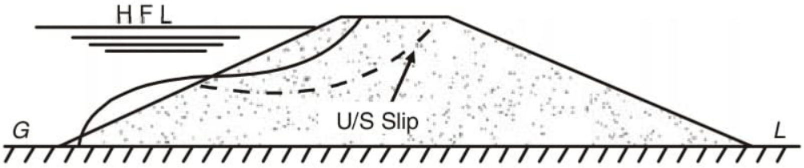


Figure 1.6: Wave action on upstream side

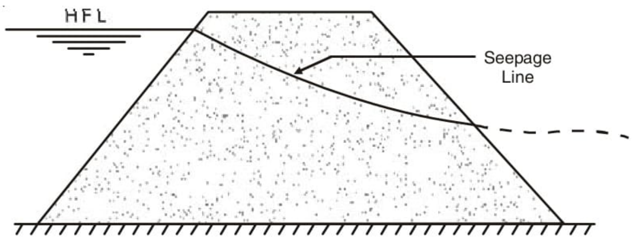


Fig. 12.8. *Seepage line in the dam.*

Figure 1.7: Seepage line in the dam

the downstream face steeper, causing it to become saturated with seepage and slide once more. This leads the downstream face to become even steeper. This seepage and sliding continues until the dam portion becomes too thin to withstand any water pressure, causing the dam to fall. Sloughing, also known as ravelling, is a process in which the toe is repeatedly saturated and then slides.

1.3 Prevalent Seismic Hazard In India

According to the United States Geological Survey, roughly 5 lakh earthquakes strike the earth each year, with 1 lakh of them being felt but just a small percentage causing damage. Furthermore, the Indian Subcontinent, particularly the north-eastern and north-western sections, is one of the most earthquake-prone regions on the planet. In 2011, there was an earthquake in

Kashmir. With approximately 1 lakh people killed, the earthquakes in Sikkim are among the worst-affected. The seismic zonation map of India plainly shows that the country is particularly vulnerable to earthquakes. In the previous 100 years, India has had over 650 earthquakes with magnitudes greater than 5.0. Aside from the moderately bustling northern and north-eastern districts, the rest of the country is very quiet. Recent earthquake-related disasters in Peninsular India, such as those in Killari (Maharashtra) and Jabalpur (Madhya Pradesh) in 1993, have prompted concerns, since the incidence of earthquake-related disasters has progressively increased. Subduction earthquakes have been documented in the Himalayan mountain range, the Andaman and Nicobar Islands, the Indo-Gangetic plain, and India's peninsular region, including a few intra-plate earthquakes.

1.4 Importance of seismic study on large dams in India

India has a long history of big earthquakes, making it a seismically active country. Seismic activity is significant in the north-eastern and north-western areas of India because the Indo-Australian plate is subducting beneath the Eurasian plate in this region. The 1967 Koyna earthquake, the 1988 Bihar earthquake, the 1991 Uttarkashi earthquake, the 1993 Killari earthquake, the 1997 Jabalpur earthquake, the 1999 Chamoli earthquake, the 2001 Bhuj earthquake, the 2002 Andaman earthquake, the 2004 Sumatra earthquake, the 2005 Kashmir earthquake, and the 2011 Sikkim earthquake are all recent major earthquakes. During these earthquakes, a vast range of constructions, ranging from small buildings to large dams, failed. The 1967 Koyna earthquake damaged the upstream and downstream sides of the dam, generating several fractures, due to reservoir-induced seismicity. The 2001 Bhuj earthquake caused a large number of earth dams to fail due to liquefaction. However, because the region had been in a drought for the preceding two years, there was no severe flooding. There have been a number of other dam failures in India as a result of earthquakes.

1.5 Mandali dam

The Mandali Dam's stability was investigated using a computer software (SLIDE V.6.0). Minimum water level 172.0, maximum water level 182.5, and seismic coeffecients in this study (0.02, 0.05, 0.1) To define the potential slip surface and calculate the factor of safety of dam slopes using the limit equilibrium method (LEM) according to the fellinius method offered by computer software (SLIDE V.6.0) is used.It is expected that the failure area is separated into number of sections. After evaluating the equilibrium of each section, a factor of safety for the estimated slip surface is calculated, taking into account the equilibrium of the entire mass. Iteratively determine the critical slip surface and factor of safety until a critical slip surface and minimum factor of safety are identified.

CHAPTER 2

Literature Survey

2.1 Analysis methods

Limit equilibrium analysis:

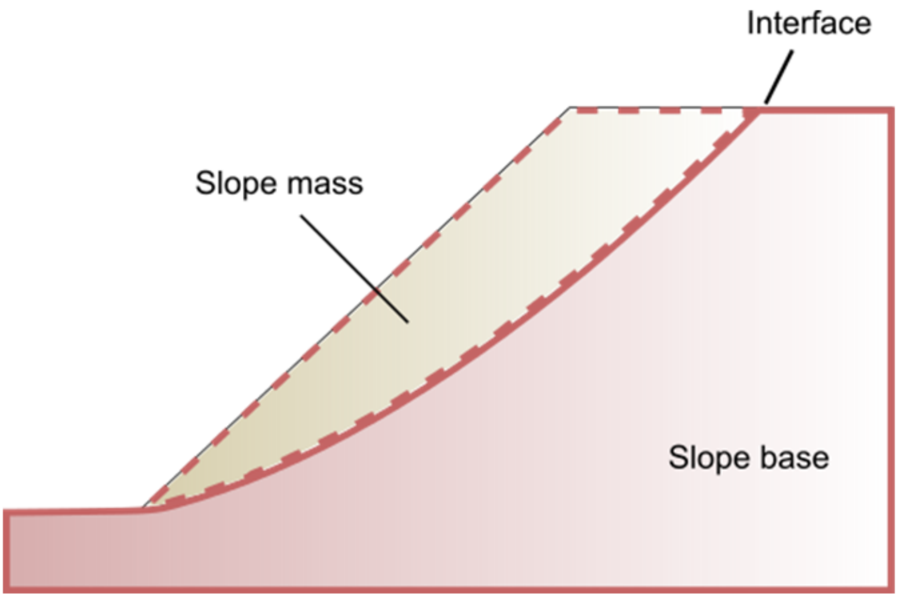


Figure 2.1: A typical cross-section of a slope used in two-dimensional analyses

Kinematic analysis, limit equilibrium analysis, and rock fall simulators are the three types of traditional slope stability methodologies. The limit equilibrium concept for a two- or three-dimensional model is used in most slope stability analysis computer programmes. The analysis of two-dimensional sections is performed under plane strain conditions. Simple mathematical approaches to stability studies of two-dimensional slope geometries can provide important insights into slope design and risk assessment. The shear strengths of the materials along the possible failure surface are assumed to be governed by linear (Mohr-Coulomb) or non-linear relationships between shear strength and the normal stress on the failure surface in all limit equilibrium methods. Terzaghi's theory of shear strength is the most widely used variation, which states that,

$$\tau = \sigma' \tan\phi' + c'$$
$$\sigma' = \sigma - u$$

where τ is the shear strength of the interface, σ' is the elective stress (total stress normal to the interface minus pore water pressure on the interface), φ' is the effective friction angle, and c' is the effective cohesion.

Method of slices is often used limit equilibrium technique. The soil mass is discretized into vertical slices in this method. There are several variations of the approach in use. Because of differing assumptions and inter-slice boundary constraints, these variations can produce different results (factor of safety).

2.2 Analytical techniques

Method of slices:

Many slope stability analysis programmes employ a variety of slice methods, including Bishop simplified, Ordinary method of slices (Swedish circle method-/Petterson/Fellenius), Spencer, Sarma, and others. Sarma and Spencer method is a rigorous methods since it meets all three equilibrium conditions: horizontal and vertical force equilibrium, as well as moment equilibrium. Procedures that are more rigorous can produce more accurate results than methods that are less rigorous. Non-rigorous approaches such as Bishop simplified or Fellenius satisfy only some of the equilibrium criteria and make some simplifying assumptions. Some of these approaches are discussed below.

2.3 Swedish Slip Circle Method of Analysis

Swedish geotechnical commission and W. Fellenius analysed slopes made of purely cohesive soil. A trial slip circle, AD of radius r is considered. Weight of wedge ABD acts in downward direction and the downward sliding of soil is resisted by a resisting moment Cu $\widehat{L}$, where Cu is undrained cohesion and $\widehat{L}$ is the length of arc.

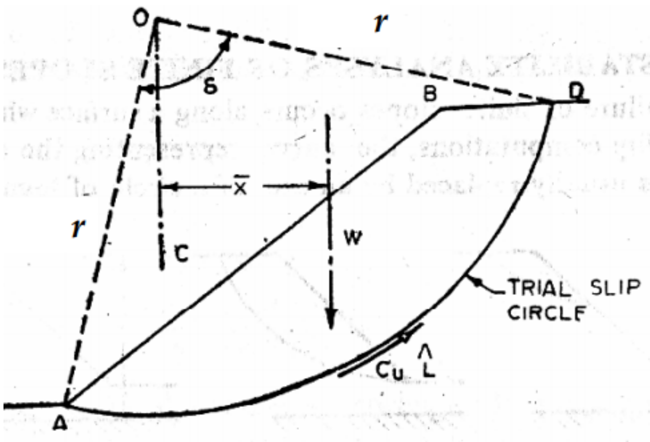


Figure 2.2: slip circle

Driving moment, M_D = W × X

where W is weight of soil wedge ABDA and X is the distance of point of application of W from the centre of slip circle, O. Factor of safety of slope is

$$F = \frac{M_R}{M_D} = \frac{c_u .L.r}{W.\bar{x}}$$

X is obtained by dividing the soil wedge into number of slices. The factor of safety of each circle is determined after considering a number of slip circles. The critical slip circle is the one with the smallest factor of safety. The Swedish slip circle approach uses circular geometry and equations to analyse stress and strength characteristics on a circular failure surface. When the internal driving forces of a slope are compared to the forces resisting slope failure, the factor of safety against sliding is obtained. If the resisting forces are greater than the driving forces, the slope is considered stable.

2.4 Modified Bishop's Method of Analysis

The Revised Bishop's method differs from traditional slice methods in that normal interaction forces between adjacent slices are considered to be collinear, resulting in a zero interslice shear force. Alan W. Bishop of Imperial college came up with this approach. The problem is statically indeterminate due to the limitation imposed by the normal stresses between slices. As a result, to solve for the factor of safety, iterative approaches must be used. It has been demonstrated that the procedure produces factor of safety values that are within a few percent of the correct values.

2.5 Lorimer's method

Lorimer's Method is a technique for assessing slope stability in cohesive soils. It differs from Bishop's Method in that it uses a colthood slip surface instead of a circle. Experiments were conducted to determine the effect of particle cementation on the mode of failure. In the 1930s, Gerhardt Lorimer developed the method.

2.6 Spencer's Method

Spencer's method of analysis necessitates the use of a computer programme with cyclic algorithms, but significantly simplifies slope stability analysis. On each slice, Spencer's method achieves all equilibria (horizontal, vertical, and driving moment). The approach can determine the factor of safety along any slip surface since it allows for unconstrained slip surfaces. The unconstrained slip surface and rigid equilibrium produce more exact safety factors than that given by Bishop's Method or the Ordinary method of slices.

2.7 Sarma method

The Sarma approach, developed at Imperial College is a limit equilibrium methodology for assessing the stability of slopes under seismic conditions. If the horizontal load is set to zero, it can also be used for static conditions. Because it can allow a multi-wedge failure mechanism, the method may analyse a wide range of slope failures and is not limited to planar or circular failure surfaces. It may provide information about the factor of safety or the critical acceleration needed to induce a collapse.

2.8 Rock slope stability analysis

The following modes of failure may be considered in a rock slope stability analysis using limit equilibrium techniques. A rock mass slides on a single surface in a planar failure (special case of general wedge type of failure). The concept of a block sitting on an inclined plane at limit equilibrium can be employed in two-dimensional analysis. The sliding of a natural rock by polygonal shaped surfaces is known as polygonal failure. Calculations are based on assumptions such as sliding over a polygonal surface, rock mass being separated into blocks by internal shear surfaces, blocks being rigid, and no tensile strength being allowed.

A three-dimensional analysis of wedge failure allows modelling of the wedge sliding on two planes in a direction parallel to the line of intersection. Toppling failure is characterised by a long, thin rock column generated by sharply dipping discontinuities that can rotate around a pivot point in the block's lowest corner. The sum of the moments causing toppling of a block, i.e. the block's horizontal weight component plus the sum of the driving forces from adjacent blocks behind the block under consideration, is compared to the sum of the moments resisting toppling, i.e. the block's vertical weight component plus the sum of the resisting forces from adjacent blocks in front of the block under consideration. Toppling occurs if driving moments exceed resisting moments.

2.9 Stereographic and kinematic analysis

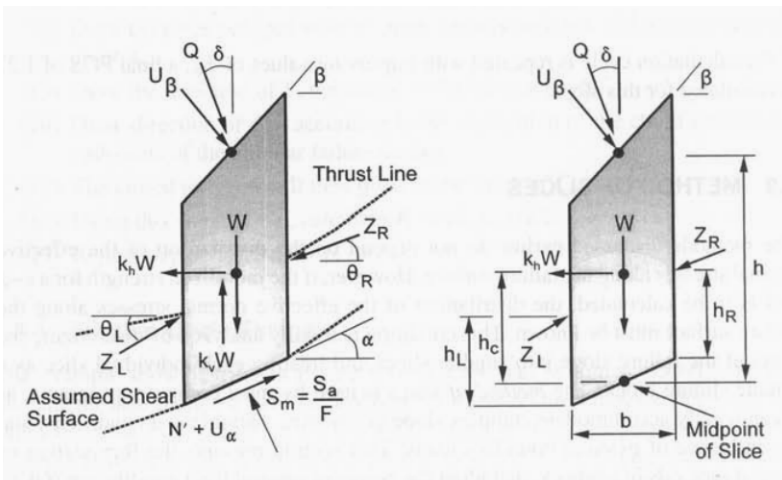
Kinematic analysis looks at the various modes of failure that could occur in a rock mass. The analysis necessitates a thorough examination of the rock mass structure as well as the geometry of any existing discontinuities that contribute to block instability. The planes and lines are shown stereographically (stereo nets). For analysing discontinuous rock slabs, stereo nets are effective. DIPS is a programme that uses stereo nets to visualise structural data.

2.10 Hybrid/coupled modelling

Limit equilibrium analysis mixed with finite element groundwater flow and stress analysis; linked particle flow and finite-deference studies are examples of hybrid programmes that combine several approaches to maximise their major advantages. Hybrid methodologies can be used to look into piping slope failures as well as the impact of high groundwater pressures on the failure of a weak rock slope. Coupled finite-element/distinct-element algorithms can be used to represent both intact rock behaviour and fracture formation and behaviour.

2.11 Manual calculation

The Ordinary method of slices was one of the first analytical approaches for estimating the stability of a slope using the method of slices. The approach assumes that the total of all interslice forces for each slice is inclined at an angle parallel to the slice's base. If the slice forces are resolved in a direction perpendicular to the slice's base (see Figure),



$$\sum F_\alpha = N' + U_\alpha + k_h W \sin\alpha - W(1-k_v)\cos\alpha - U_\beta \cos(\beta-\alpha) - Q\cos(\delta-\alpha) = 0$$

above equation for N' can be rewritten as

$$N' = -U_\alpha - k_h W \sin\alpha + W(1-k_v)\cos\alpha + U_\beta \cos(\beta-\alpha) + Q\cos(\delta-\alpha) = 0$$

The mohr-columb mobilising shear strength

$$s_m = \frac{C + N' \tan\phi}{F}$$

$$\sum M_0 = \sum_{i-1}^{n} [W(1-K_v) + U_\beta \cos\beta + Q\cos\delta] Rsin\alpha \sum_{i=1}^{n} [U_\beta \sin\beta + Q\sin\delta](R\cos\alpha - h)$$

$$-\sum_{i=1}^{n} [s_m] R + \sum_{i=1}^{n} [k_h W (R\cos\alpha - h_c)]$$

$$\mathrm{F} = \frac{\sum_{i=1}^{n} (C + N' \tan\emptyset)}{\sum_{i=1}^{n} A_1 - \sum_{i=1}^{n} A_2 + \sum_{i=1}^{n} A_3} \tag{2.1}$$

$$\mathrm{A}_1 = (W(1-kv) + U_\beta \cos\beta + Q\cos\delta)\sin\alpha \tag{2.2}$$

$$\mathrm{A}_2 = (U_\beta \sin\beta + Q\sin\delta)\left(\cos\alpha - \frac{h}{R}\right) \tag{2.3}$$

W =	weight of slice	h =	height of slice
N' =	effective normal force	c =	cohesive shear strength component of soil
$\mathrm{K_y}$ =	vertical seismic coefficient	φ =	angle of internal friction of soil
$\mathrm{K_h}$ =	horizontal seismic coefficient	α =	inclination of slice base
U_β =	surface water force	β =	inclination of slice top

$$A_3 = k_h W \left(\cos\alpha - \frac{hc}{R}\right) \tag{2.4}$$

CHAPTER 3

Scope and Objective

Seismic analysis of an earthen dam is carried out in this project utilising slide 6 software by considering various seismic coeffecients. Using slide 6 software, the safety factor of earthen dam under the following conditions is determined. The following variables are taken into account.

1. For full reservoir condition
2. various methods which are included in the software, used to predict the factor of safety.
3. For different water columns considering seismic force
4. Different slope angles.
5. For steady state seepage downstream condition.

CHAPTER 4

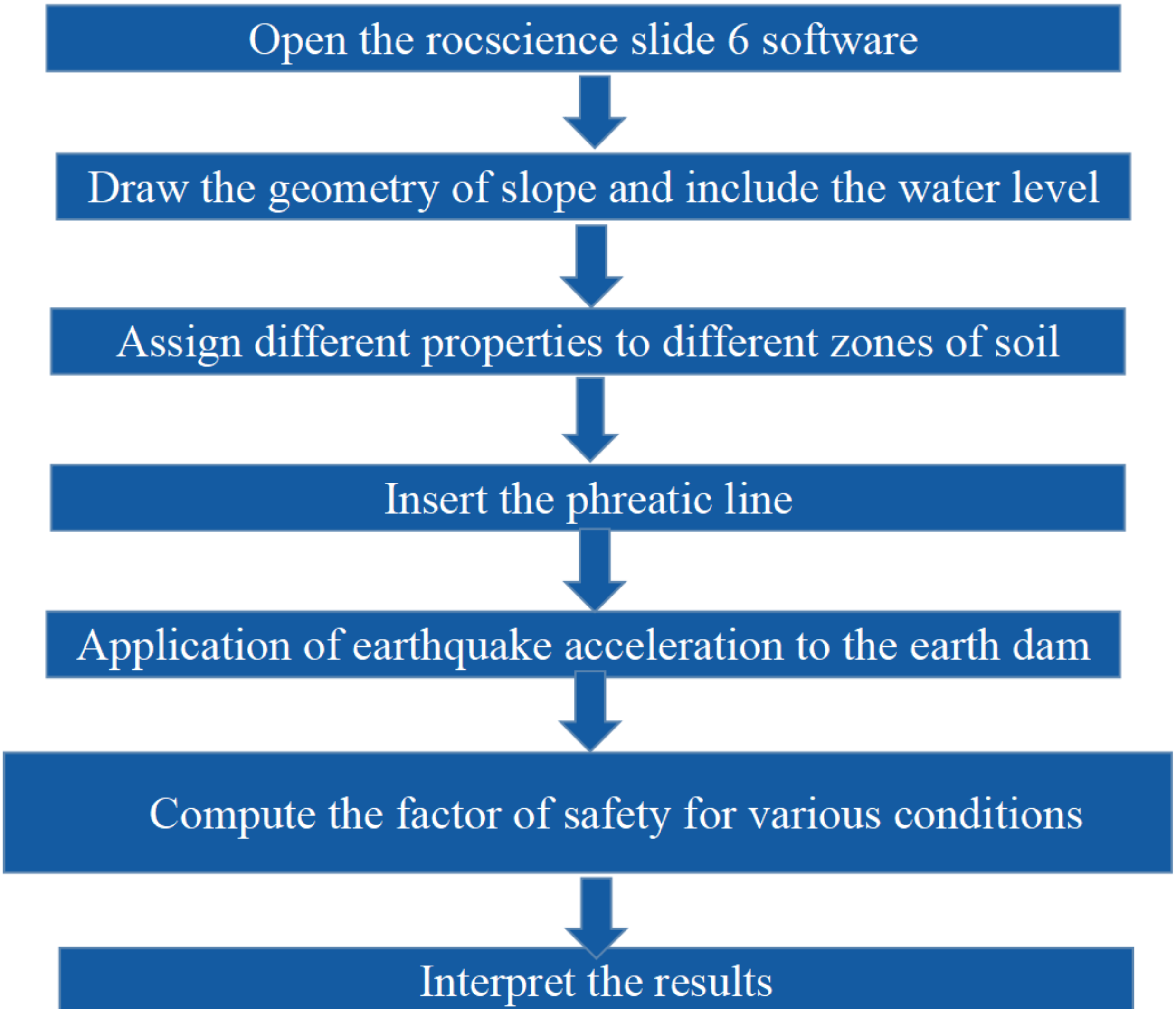
Methodology

4.1 Properties of earthen dam

Following parameters are considered

1. Shell
   - Cohesion (C) = 0 Kpa
   - Angle of internal friction = 40°
   - Coeffecient of permeability K (cm/s) = 1.37*10-5
   - Unit weight of soil, γ (kN/m3) = 21
2. Core
   - Cohesion (C) = 40 Kpa
   - Angle of internal friction = 15°
   - Coeffecient of permeability K (cm/s) = 9.88*10-9
   - Unit weight of soil, γ (kN/m3) = 19.2
3. Foundation
   - Cohesion (C) = 5 Kpa
   - Angle of internal friction = 30°
   - Coeffecient of permeability K (cm/s) = 10-8
   - Unit weight of soil, γ (kN/m3) = 22
4. Filter
   - Cohesion (C) = 0 Kpa
   - Angle of internal friction = 37°
   - Coeffecient of permeability K (cm/s) = 10-2
   - Unit weight of soil, γ (kN/m3) = 21

4.2 Procedure



CHAPTER 5

RESULTS AND DISCUSSION

- factor of safety obtained for full reservoir condition in fellinius method is 1.337 by considering seismic coeffecient 0.05

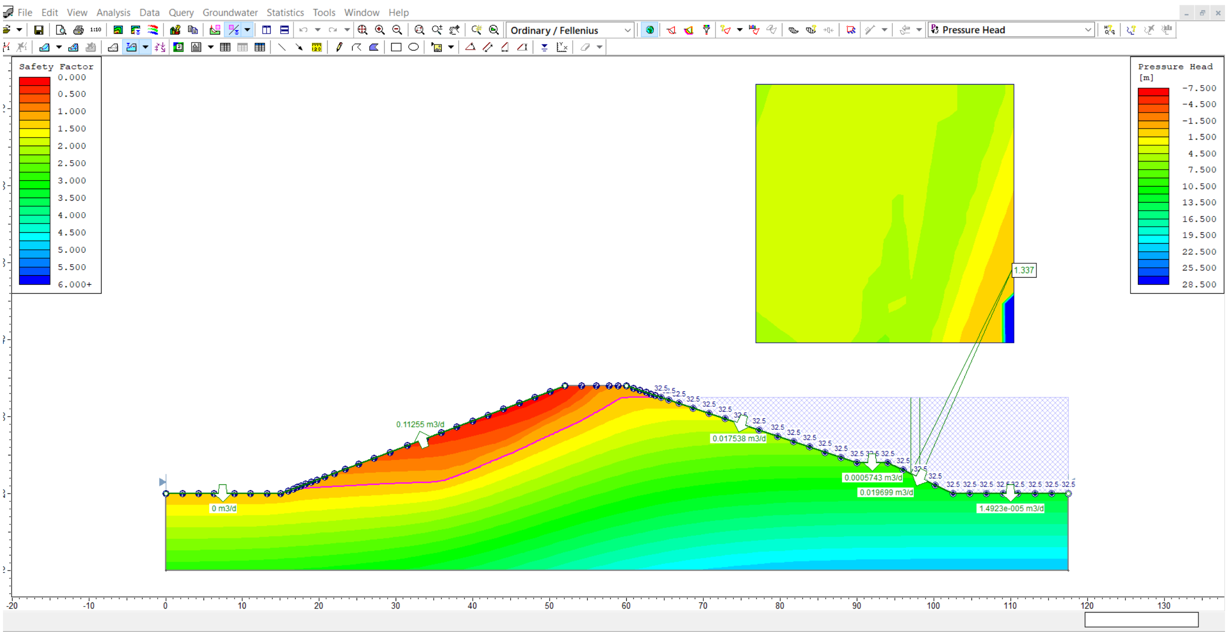


Figure 5.1: F.O.S for mandali dam in fellinius method

- factor of safety obtained for steady seepage downstream condition in fellinius method is 1.507 by considering seismic coeffecient 0.05

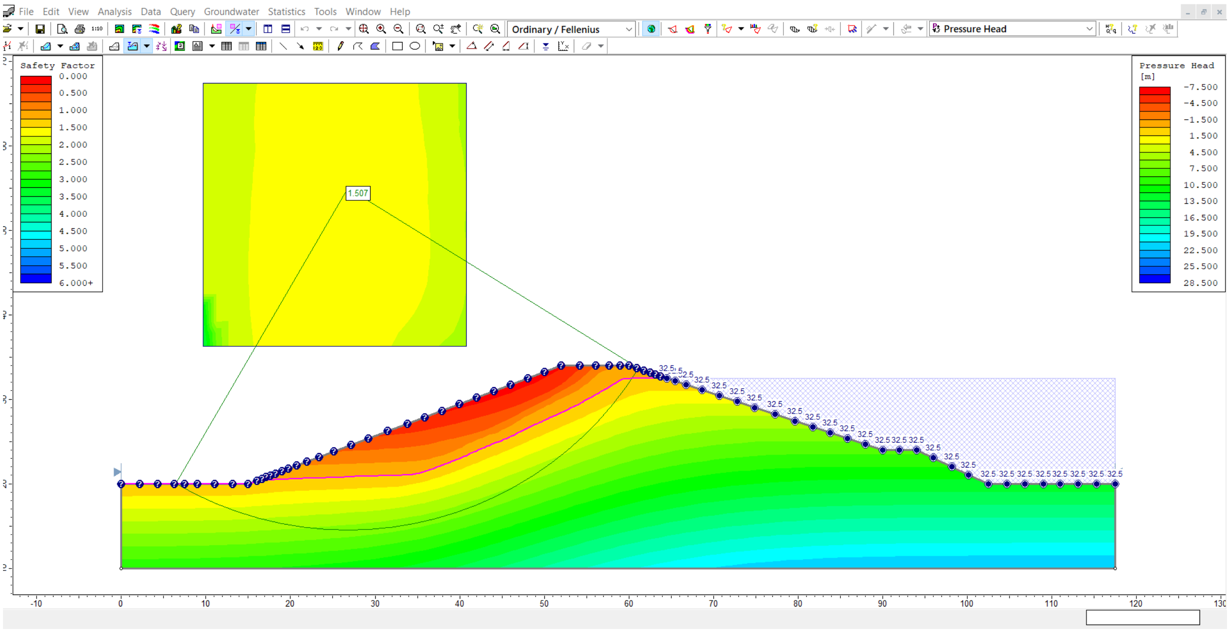


Figure 5.2: F.O.S of mandali dam in fellinius method

- factor of safety obtained for Partial drawdown(3m) condition in fellinius method is 1.339 by considering seismic coeffecient 0.05

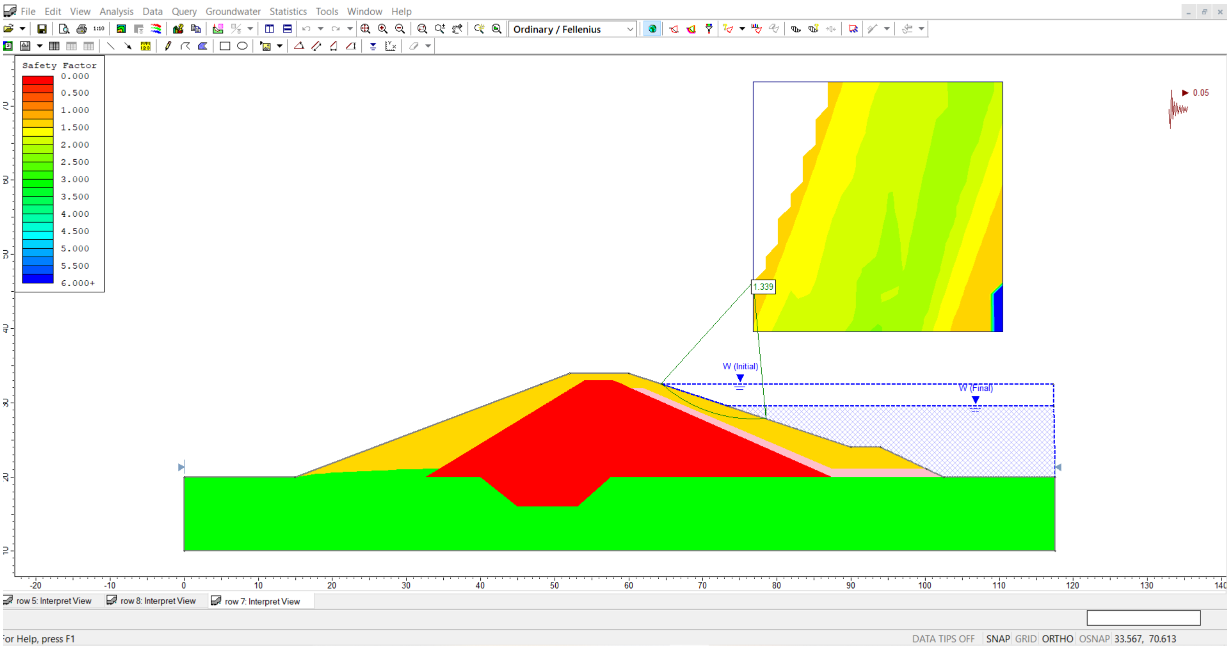


Figure 5.3: F.O.S for mandali dam in fellinius method

s.no	seismic co-effecient	Fellinious	Bishop simplified	Janbu simplified	Janbu corrected	spencer	Corps of engineering 1	Corps of engineering 2	Lowe-Karafiath	GLE/ Morgenstern price method
Full reservoir										
1	0.02	1.528	1.509	1.532	1.545	1.533	2.273	2.532	2.181	1.533
2	0.05	1.337	1.318	1.342	1.353	1.344	2.972	1.342	1.877	1.344
3	0.1	1.090	1.074	1.097	1.106	1.100	1.586	1.097	1.507	1.100
Steady state downstream										
1	0.02	1.654	1.716	1.574	1.689	1.748	1.740	1.805	1.767	1.752
2	0.05	1.654	1.565	1.431	1.534	1.597	1.573	1.631	1.595	1.602
3	0.1	1.307	1.357	1.239	1.328	1.395	1.344	1.400	1.367	1.399

Table 5.1: FOS values for mandali dam in various methods

condition								
water level (3m drawdown)			water level (6m drawdown)			water level (9m drawdown)		
0.02 (sc)	0.05 (sc)	0.1 (sc)	0.02 (sc)	0.05 (sc)	0.1 (sc)	0.02 (sc)	0.05 (sc)	0.1 (sc)
1.497	1.339	1.009	1.258	1.138	0.974	0.986	0.901	0.781

Table 5.2: fos values for different water levels in fellinius method

slice no	C	N	()	A1	A2	A3
1	0	14.7784	37	6.294	2.13*10-3	0.562
2	0	14.8245	37	6.26	2.6*10-3	0.568
3	0	14.8689	37	6.232	3.86*10-3	0.574
4	0	14.9115	37	6.199	1.539*10-3	0.580
5	0	14.9523	37	6.163	2.13*10-3	0.583
6	0	14.9914	37	6.126	2.13*10-3	0.591
7	0	13.747	37	5.569	2.13*10-3	0.545
8	0	13.7769	37	5.551	2.13*10-3	0.551
9	0	13.8056	37	5.498	2.13*10-3	0.554
10	0	14.7784	37	5.461	2.13*10-3	0.558
11	0	14.8245	37	5.422	2.13*10-3	0.563
12	0	14.8689	37	5.382	2.13*10-3	0.566
13	0	14.9115	37	5.34	2.13*10-3	0.570
14	0	14.9523	37	5.298	2.13*10-3	0.574
15	0	14.9914	37	5.254	2.13*10-3	0.578
16	0	13.747	37	5.208	2.13*10-3	0.582
17	0	13.7769	37	5.162	2.13*10-3	0.585
18	0	13.8056	37	5.114	2.13*10-3	0.589
19	0	14.7784	37	5.066	2.13*10-3	0.592
20	0	14.8245	37	5.016	2.13*10-3	0.596
21	0	14.8689	37	4.964	2.13*10-3	0.599
22	0	14.9115	37	4.913	2.13*10-3	0.602
23	0	14.9523	37	5.129	2.13*10-3	0.599
24	0	14.9914	37	5.539	2.13*10-3	0.608
25	0	13.747	37	4.756	2.13*10-3	0.6119

Table 5.3: Manual safety factor calculation for full reservoir condition

CHAPTER 6

CONCLUSION

The following conclusions are made from the work

1. The critical safety factor for complete reservoir condition decreased by 14.28% when seismic coeffecient increased from 0.02 to 0.05.

2. The minimum factor of safety for steady seepage condition decreased by 26.54% when seismic coeffecient increased from 0.02 to 0.1 in fellinius method.

3. The minimum factor of safety for partial drawdown condition decreased by 48.61% when drawdown level increased from 3m to 9m by considering seismic coeffecient 0.05 in fellinius method.

4. The factor of safety decreased with increase in seismic load.

5. The factor of safety obtained from manual calculation for full reservoir condition is 1.51

CHAPTER 7

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