SEISMIC ANALYSIS OF TAILINGS DAM

A DISSERTATION

Submitted in partial fulfilment of the
requirements for the award of degree
of

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in
EARTHQUAKE ENGINEERING
(With specialization in Soil Dynamics)
by

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CANDIDATE'S DECLARATION

It is certified that this work has been carried out in Department of Earthquake Engineering at Indian Institute of Technology under the guidance of Dr. S.Mukerjee, visiting faculty of, Department of Earthquake Engineering, Indian Institute of Technology Roorkee, Roorkee, India.

Date: 2-June-2016 (Vipul Rungta)

Place: Roorkee 14525018

CERTIFICATE

This is to certify that the above statement made by the candidate is correct to the best of my knowledge and belief.

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ABSTRACT

Tailings dams are used to impound waste tailings generated by mining activities. Failure of a tailings dam is hazardous for that region because corrosive and radioactive material in the tailing would then contaminate the surrounding environment. The tailings material is a fine grind of sizes similar to clay particles however they behave as cohesionless material and thus are very likely to liquefy under earthquake conditions. Stability of a tailings dam needs to be checked under earthquake loading.

The tailings dam under consideration is a zoned dam raised in six stages using the center line method and having three different types of fill materials. The zones, from upstream to downstream, consisted of (i) impervious material (ii) compacted tailings and (iii) pervious random fill. An inclined chimney drain and a connecting horizontal filter are provided to keep most of the dam section on the downstream side dry.

The main objective of this study is to perform static and pseudo-static analyses by the Finite Element Method (FEM) using the Strength Reduction Technique and comparing the results with those from various Limit Equilibrium Methods (LEM) such as Bishop Simplified, Janbu Simplified, Janbu Corrected, Spencer, Corps of Engineers-1, Corps of Engineers-2, Lowe Karafiath and Morgenstern Price, considering both circular and non-circular failure surfaces. Finite element analyses have been performed using the geotechnical software PHASE-2, while Limit equilibrium analyses has been performed using geotechnical software SLIDE.

The results are presented in the form of normalized plots for the following cases: (i) Stagewise (With Tailings) (ii) Stagewise (Without Tailings) (iii) Considering circular failure surface (iv) Considering non-circular failure surfaces.

Keywords: Slope Stability, Tailings Dam, Limit Equilibrium Method (LEM), Finite Element Analysis, Strength Reduction Method (SRM).

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Chapter 1

INRODUCTION

To obtain metals and minerals for industry and mining operations, large quantities of rocks are mined, crushed and pulverized to obtain the useful metals and other mineral value. A fine grind is often required from sand sized to even micron level. These fine grinds are known as tailings.

Due to vast industrial development, the amount of tailings production has increased manifold. Ore generally contains less than few percent of metal values, the residue becomes tailings. This tailing disposal is a major problem in mining operations. Earlier the tailings were disposed as conveniently but as the local concern occurred such as sedimentation in downstream water course, contamination of underground water due to presence of radio-active waste, there occurred necessity to dispose it safely.

There are several methods of tailings disposal which include disposal of thickened or dry tailings in impoundments, backfilling underground mines or open pits, subaqueous disposals and most importantly the disposals of tailings slurry in impoundments.

Impoundments are favored because they are economically attractive and relatively easy to operate. These impoundments are generally constructed of tailings and other waste materials. The tailings embankments in some projects may reach several hundred feet in height and may cover several square miles.

Under any circumstances, these impoundments must not fail as the mere failure of it can damage the local region due to presence of corrosive and radio-active elements in it. Due to such threat static and dynamic analysis is being done.

1.1 Overview of Tailing Disposal

Impoundment of slurry tailings is the most common method of disposal and is the main focus of this report. Impoundments are favored because, among other things, they are "economically attractive and relatively easy to operate". Tailings impoundments can be and are designed to perform a number of functions, including treatment functions. These include:

- Removal of suspended solids by sedimentation
- Precipitation of heavy metals as hydroxides
- Permanent containment of settled tailings

• Equalization of wastewater quality

• Stabilization of some oxidizable constituents (e.g., thiosalts, cyanides, flotation reagents)

• Storage and stabilization of process recycle water

• Incidental flow balancing of storm water flows.

There are, however, a number of disadvantages to tailings impoundments requiring attention in design,including:

• Difficulty in achieving good flow distribution

• Difficulty in segregating drainage from uncontaminated areas

• Difficulty in reclamation, particularly with acid-generating tailings, because of the largesurfacearea and materials characteristics

• Inconsistent treatment performance due to seasonal variations in bio-oxidation efficiency

• Costly and difficult collection and treatment of seepage through impoundment structures

• Potentially serious wind dispersion of fine materials unless the surface is stabilized by revegetation, chemical binders, or rock cover.

1.2 Objective of Study

Tailings dams are used to impound waste tailings generated by mining activities. Failure of a tailings dam is hazardous for that region because corrosive and radioactive material in the tailing would then contaminate the surrounding environment. The tailings material is a fine grind of sizes similar to clay particles however they behave as cohesionless material and thus are very likely to liquefy under earthquake conditions. Stability of a tailings dam needs to be checked under earthquake loading.

The main objective of this study is to perform static and pseudo-static analyses by the Finite Element Method (FEM) using the Strength Reduction Technique and comparing the results with those from various Limit EquilibriumMethods (LEM) such as Bishop Simplified, Janbu Simplified, Janbu Corrected, Spencer, Corps of Engineers-1, Corps of Engineers-2, Lowe Karafiath and Morgenstern Price, considering both circular and non-circular failure surfaces. Finite element analyses have been performed using the geotechnical software PHASE-2, while Limit equilibrium analyses have been performed usinggeotechnical software SLIDE.

Chapter 2

REVIEW OF LITERATURE

Debarghya Chakraborty and Deepankar Choudhury: Performed numerical analysis to study the static and dynamic seismic behavior of tailings dam by FLAC-3D. The result showed amplification in base level motion with the height of dam and concluded that factor of safety under static condition is higher than in dynamic condition.

Debarghya Chakraborty and Deepankar Choudhury: In his another paper the author objective was to check the stability of dam slope during earthquake events. A 28 m high tailing dam, constructed using downstream method in two phases was selected and analysis was performed using geotechnical software TALREN4. At last it was presented that in proposed tailings dam the factor of safety and yield acceleration value decrease significantly as was expected.

H. Klapperich et al. : The author used seed's method to compute dynamic stresses and strains in time domain by numerical finite element analysis . Classical theory of failure circle was also used for stability analysis wherein material softening was considered.
Depending upon the results and failure model the author recommended some short and long term measures.
SHORT TERM: Installation of seismographs and piezometers at different levels of dam to measure earthquake response and inclinometer to measure lateral movement along the failure plane.
LONG TERM: Drainage of tailings material to increase the effective shear strength and installation of toe resistant dam at downstream. Nailing of failure surface by piles is done to increase the factor of safety.

Jonathan Z Liang and David Elias(2010): The author did the comparative study of two tailings dams built through upstream and downstream construction methods. Seismic performance were investigated using numerical dynamic analysis. The numerical modelling was carried using PLAXIS geotechnical software. Investigation revealed that horizontal and vertical displacement at crest is lower in downstream than in upstream method of construction. The upstream dam is very prone to liquefaction, as a result of which large deformation or failure may occur.

Jorge H. Troncoso (2011): The author studied the deformations caused in the body of tailing deposit by earthquake and its effect on structural and hydraulic stability of dam. The limits of acceptable deformations were suggested by empirical and analytical methods.

Luis Valenzuela (2011): The author represents the summary of the main concepts involved on selection of seismic coefficients over the last four decades. In the present paper, the relationship of the seismic coefficients with critical acceleration is also discussed. At the end the author gives two basic approaches of deformation of seismic coefficients for dams of similar characteristics.

David M Chambers and Bretwood Higman (2011): The author presents the long term risks of tailings dam failure due to catastrophic release of large amount of tailings. The paper mentions that rate of failure of tailings dam is much higher than water supply reservoir dams due to following reasons:

(i) The capability to use erection types for tailings dams that are more vulnerable to failure.

(ii) Tailings dams are built in successive raises over numerous years that makes quality control difficult more difficult compared water retention dams which are raised entirely at once.

The author finally concluded that the policy makers should not only rely on assumptions about specific hazards (as they are probably flawed) but also keep in view the risk from conservative probabilistic perception.

D.V.Griffiths and P.A.Lane (1999): The author claims that numerical finite element analysis of slope stability is much better tool than traditional limit equilibrium method due to its fewer assumptions. Several advantages of finite elements are presented in support of above. The paper describes several examples of finite element analysis with comparison against other methods.

At the end author suggested that the method should now be considered strongly by geotechnical practitioners as an alternative to limit equilibrium methods in computer aided analysis.

T.E. Martin and E.C. McRoberts: The author presents considerations in the stability analysis of upstream tailings dams. This paper describes a comparable study of drained vs undrained static stability off upstream tailings dam. The author supports the view that effective stress analysis for upstream dams constructed of contractant, potentially liquefiable tailings can be fundamentally incorrect and unsafe.

Hendra Jitna: Investigated a typical upstream raised tailings dam for liquefaction and deformation behavior by using different earthquake design ground motions with different response spectra (matched or unmatched)using FLAC2D an FEM software. The modelling of soil was done using USCSAND constitutive soil model. It has been observed that scaling and matching of earthquake record have great effect on response of tailings dam, so must be advisedly done.

Jianping Pan et al. (2015): The author studied the effect of different parameters such as intensity, mean grain size, outside slope gradient etc for liquefaction risk analysis on tailings dam. It was observed that there is occurrence of decrease in seismic liquefaction risk of dam with increase in mean grain size and with decrease of outside slope gradients. Reliability theory in liquefaction evaluation was given priority over other methods.

Gonzalo Castro (2003): The author performed seismic analysis on tailings dam having fine tailings. The result of analysis showed loss in the peak undrained strength under seismic loading. And also this loss is better related to the cyclic strain in comparison to the increment in pore pressure due to earthquake.

LIU Hou-xing etal. (2007): The author performed effective stress analysis on tailings dam in upstream raising method which is 113.5 m high. The result showed that liquefaction resistance and seismic stability are improved bizarrely and the depth of liquefaction area at the top of dam is greatly lowered. The result also showed that liquefaction is key cause of seismic failure of high tailings dam and the outcome of seismic inertia force on its stability is the secondary cause.

Chapter 3

CONSTRUCTION METHODS OF TAILINGS DAM

Tailings dam is mostly constructed by use of tailings. Some of the drawbacks include high susceptibility to frost action and internal piping, the surface being highly erodible, and liquefaction under earthquake shocks. The two ways, during the construction, to improve the above qualities are use of compactionand coarse tailings. The Compaction is usually done by vibratory compactors.

3.1 Construction Techniques

The three methods of construction are upstream, downstream and center line.

3.1.1 Upstream Method

It is the oldest and most economical method of tailings dam construction which starts with downstream toe. It takes the advantage of self-consolidating beach where the coarse particles settle close to the discharge spigot forming a beach while the finer materials flow away making slimes. Some mechanical compaction is usually accompanied before subsequent stage of the dam is erected.

3.1.2 Downstream Method

It is much similar to conventional water storage dams. It also start with a starter dam built with compacted material predominantly clay or any impervious material to reduce seepage. This method is so termed because the successive lift rests on the downstream slope of the previous dike and the centerline of the subsequent dikes shifts downwards as the dam phases are gradually raised.

This method of construction provide greater degree of stability than upstream method due to its compaction and to the fact that foundation strength of dike raises are not structurally dependent on tailings deposit. The major disadvantage lies in its construction cost. Huge amount of construction material is required which elevate its cost especially when tailings mill cannot provide sufficient sand. The other disadvantage is that it necessitates huge areaand can be of crucial concern when the space is restricted.

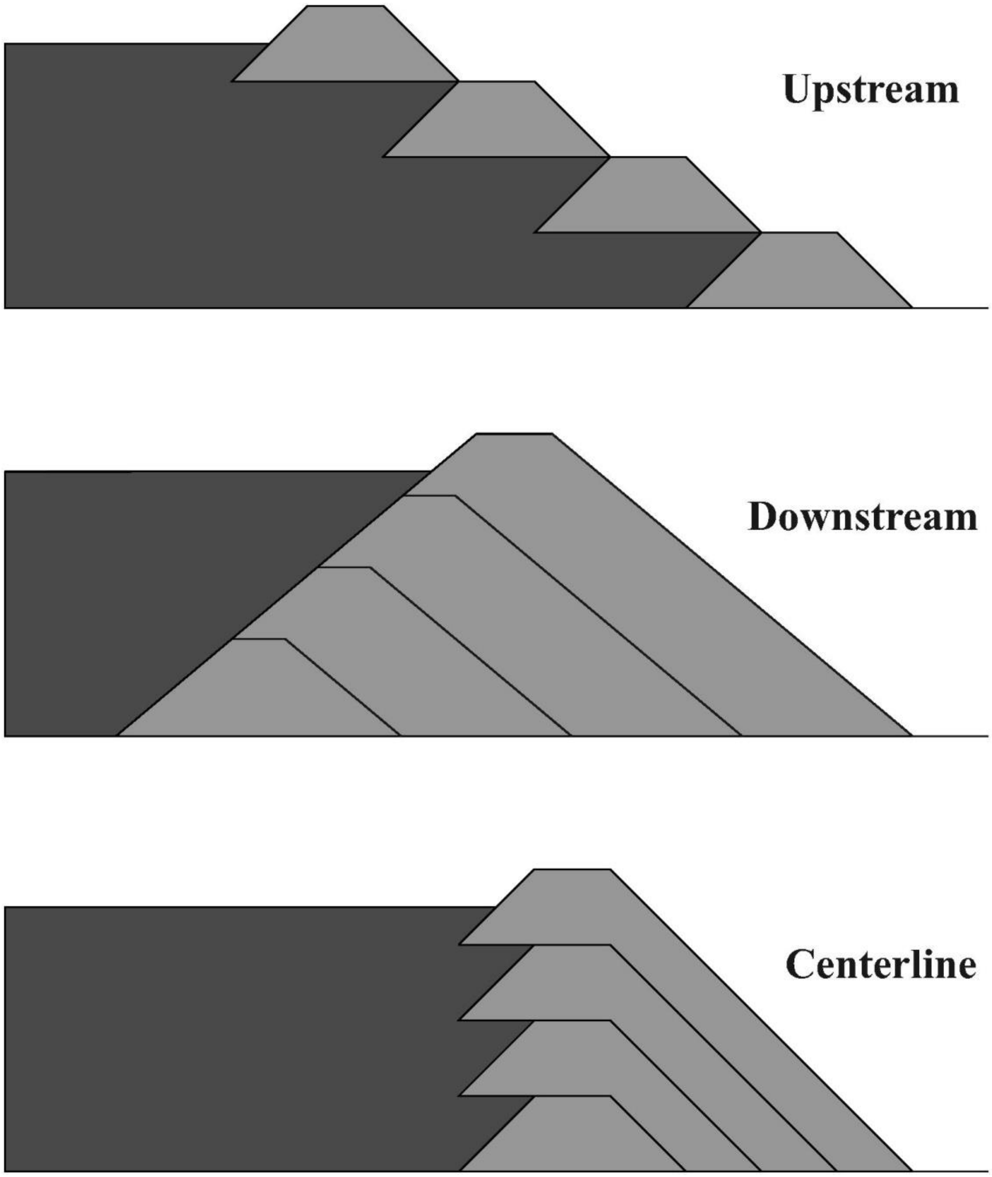


Fig. 1 Type of sequentially raised tailing dams

3.1.3 Centerline Method

Centerline method is similar to both the above methods. It starts with the starter dam and the tailings are spigotted off on both side of crest. It is named so because centerlines of the dikes are maintained same as the subsequent dams are raised. Tailings especially on the downstream sides are compacted properly to prevent any shear failure.

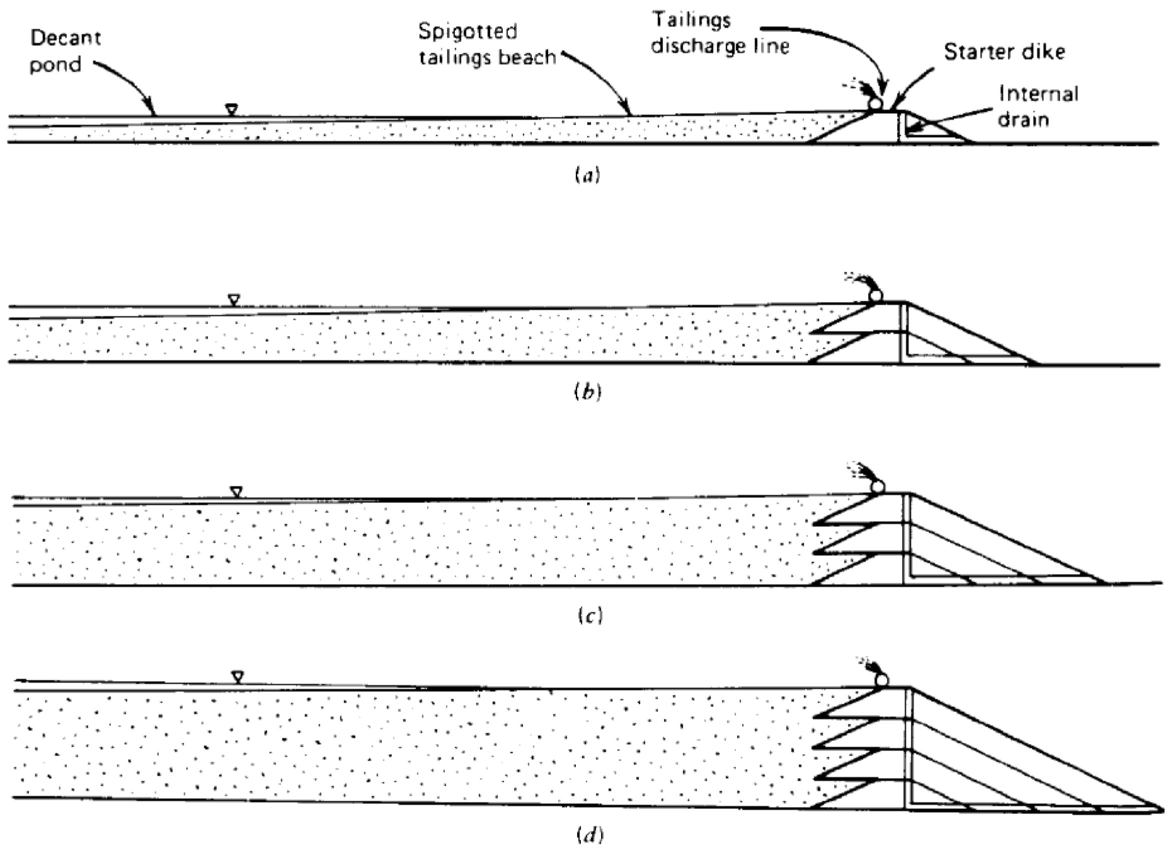


Fig. 2 Centerline Embankment Construction

The degree in stability and economy lies intermediate of upstream and downstream method of construction.

Table 1.Comparison of surface Impoundment Embankment Types

	Water Retention	**Upstream**	**Downstream**	**Centerline**
Mill Tailings Requirements	Suitable for any type of tailings	At least 40-60% sand in whole tailings. Low pulp density desirable to promote grain-size segregation	Suitable for any type of tailings	Sands or low-plasticity slimes
Discharge Requirements	Any discharge procedure suitable	Peripheral discharge and well-controlled beach necessary	Varies according to design details	Peripheral discharge of at least nominal beach necessary

Water Storage Suitability	Good	Not suitable for significant water storage	Good	Not recommended for permanent storage. Temporary flood storage acceptable with proper design
Seismic Resistance	Good	Poor in high seismic areas	Good	Acceptable
Raising Rate Restrictions	Entire embankment constructed initially	Less than 4.5 - 9 m/yr most desirable. Greater than 15 m/yr can be hazardous	None	Height restrictions for individual raises may apply
Embankment Fill Requirements	Natural soil borrow	Natural soil, sand tailings, or mine waste	Sand tailings or mine waste if production rates are sufficient, or natural soil	Sand tailings or mine waste if production rates are sufficient, or natural soil
Relative Embankment Cost	High	Low	High	Moderate

3.2 Failure Modes

There are number of modes by which embankments may fail such as foundation failure, rotational failure sliding, piping, liquefaction, erosion, overtopping, etc. These may result in partial or total failure of dam structure.

This thesis analyses slope failure by rotational sliding but the knowledge of other modes of failure is also important.

a) **Rotational Sliding**

In 2-D analysis, the failure surface can be approximated by a circular arc commonly called circular slip. The failure occurs along most critical slip circle where factor of safety isminimum. In stable slope the resistive shear strength along the potential failure surface exceeds the driving shear stress which tends to induce movements.

The failure is said to be rotational as the failure surface is in form of arc which tends to rotate about instantaneous centre of rotation.

Fig.3 Example of rotational sliding in an embankment

b) Foundation Failure

Foundation failure occurs when there exists weak plane between the interface of foundation and embankment. Movement may occur when induced shear stress exceeds the shear strength of exiting layer. This type of failure is more significant in staged construction (like those of tailings dam) where the foundations of subsequent dykes are the existing dykes itself.

c) Overtopping

Overtopping occurs due to high flood water or mismanagement of tailings entering the ponds. When overtopping occurs, the downstream slope gets eroded which may in turn cause the complete failure. Effective diversion of excess flood water or tailings to some suitable place is key in designing of tailings dam as the entire locality may get contaminated due to presence of corrosive and toxic element into the waste deposits.

Fig.4 Example of an embankment overtopping (EHA, 2008)

d) **Piping**

Piping is sub-surface erosion of tailings embankments which can rapidly cause failure. This happens when critical hydraulic gradient is exceeded mainly in areas of poor compaction. Generally piping starts at the downstream toe and works back towards the reservoir forming pipes or channel under the dam.

Fig.5 Embankment failure by piping along the outlet pipe (USDA, n.d.)

e) **Liquefaction**

Liquefaction occurs in unconsolidated, saturated deposits of similarly sized tailings deposits. Main ingredients of liquefaction is generation of excess pore water pressure under undrained loading since it suddenly decreases the effective stress (which in turn is directly related to shear strength of soil). Earthquake shake often triggers this phenomenon.

f) **Seepage**

Fig.6 Example of seepage through an embankment (USDA, n.d.)

Seepage is movement of water through and around the dam dykes. The seeping tailings materials can contaminate the ground water which may prove hazardous in long run. Seepage can be minimized by use of filter wells, liners, drainage or decanting system.

Fig.7 Construction of a tailings dam with lining in Perth (Cape Crushing, 2013)

3.3 Tailings Dam Failure Incidents

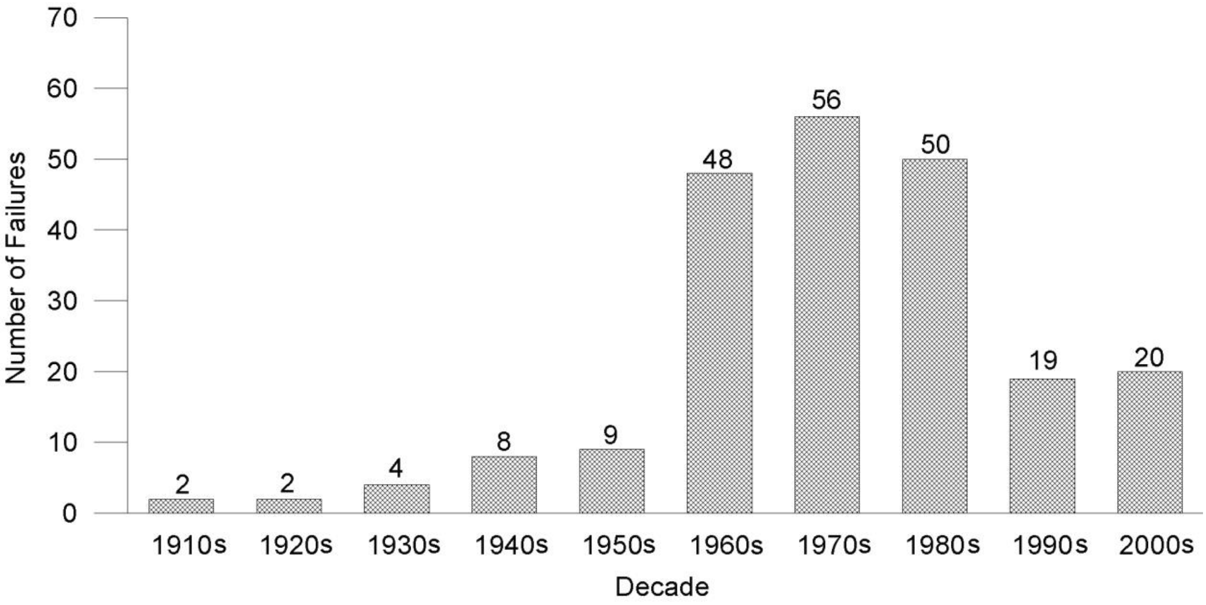
Around 3500 tailings dam are situated all over the world. Worldwide there are around 25420 to 48000 large dams. Tailings dams failure occur more frequently than water retention dam. It happens possibly due to two reasons:

a) The ability to use construction methods that are vulnerable to failure.
b) The fact that tailings dam are often built in sequential raises over numerous years that makes quality control difficult compared to water retention dam that are raised entirely at once.
c) Lack of regulations on design criteria.
d) High maintenance cost

"Satellite imagery has led us to the realization that tailings impoundments are probably the largest man-made structures on earth. Their safety, for the protection of life, the environment and property, is an essential need in today's mining operations. These factors, and the relatively poor safety record revealed by the numbers of failures in tailings dams, have led to an increasing awareness of the need for enhanced safety provisions in the design and operation of tailings dams. The mining industry has a less than perfect record when tailings dam failures are reviewed." (ICOLD, 2001, p. 15)

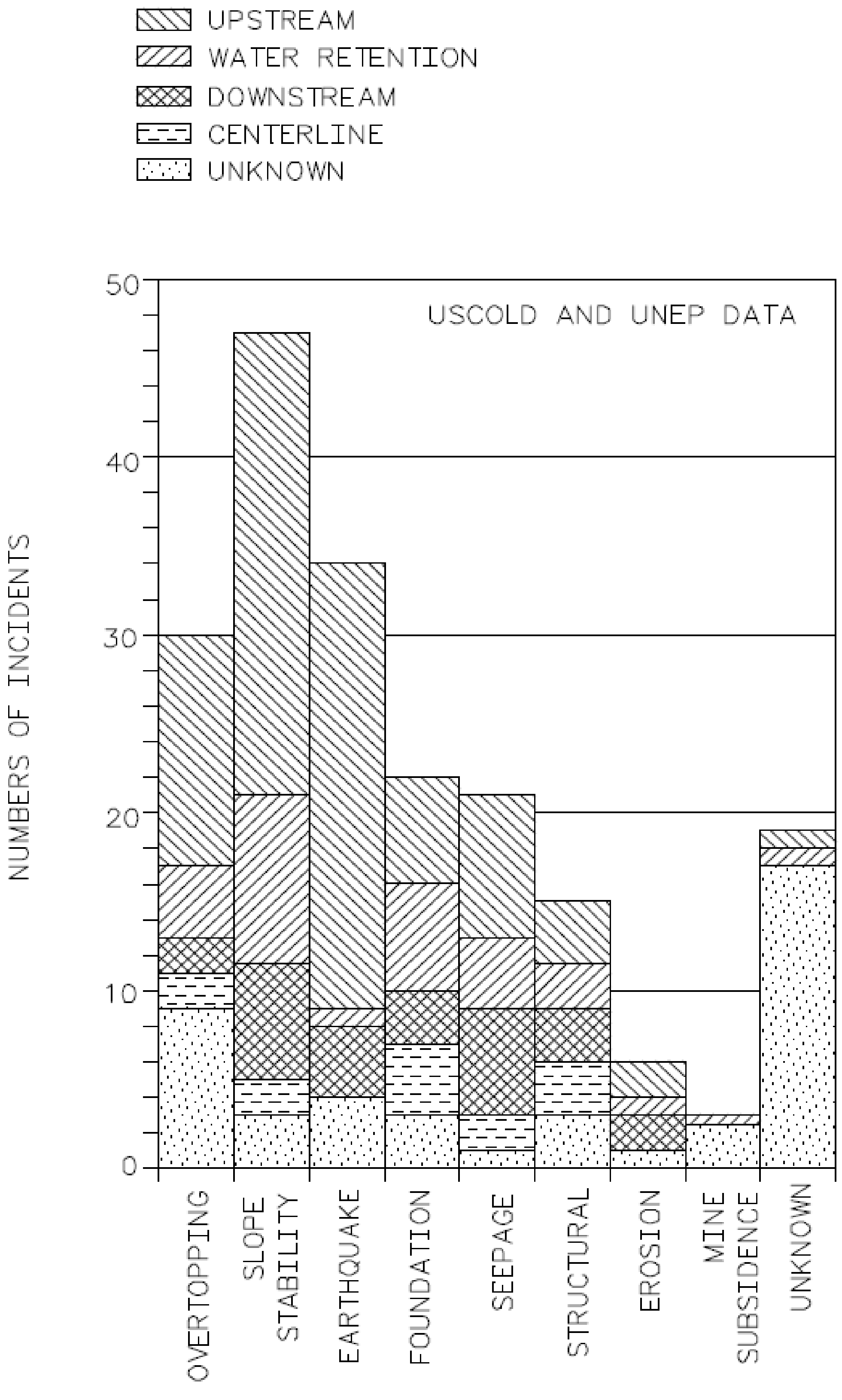
“Unfortunately the number of major incidents continues at an average of more than one a year. During the last 6 years the rate has been two per year.” (ICOLD, 2001, p. 8)

Azam and Li (2010), compiled the failures of tailings dam over last one hundred years. They found that only 8 to 9 tailings dam failed per decade in 1940s and 1950s but rose to about 50 failures per decade in 1960s, 1970s, 1980s and 1990s. The higher rate of failure may be attributed to greater demand for minerals after World War II. Failures significantly reduced to 20 per decade in 1990s and 2000s. This improvement was probably due to improved construction technology, tougher safety criteria and sufficient engineering experience.



Graphs 1.Failure events over time

ICOLD Bulletin 121 provided a summary of failure types separated by tailings dam type (upstream, downstream, centerline) for dam failures prior to 2000. The graph shows that upstream dams have failed more often than downstream or 8centerline constructed dams. This could also be due to the number of dams constructed using the upstream method is far greater than the number of downstream or centerline dams.



Graphs 2.Incident cause comparison with dam type (ICOLD Bulletin 121, 2001)

Chapter 4

METHOD OF ANALYSIS

Analysis of slope stability is done to review the design natural or man-made slopes. The main aim is to find the endangered areas, probable failure mechanism, slope sensitivity to various triggering mechanism, designing slopes with respect to desired degree of stability, economics and designing probable corrective measures.

4.1 Limit Equilibrium Method

It is most commonly method used method for slope stability analysis. Limit equilibrium calculate factor of safety using equilibrium of forces or moments or both. It requires minimal input. For force(or moments) equilibrium the factor of safety is calculated by sum of resisting forces(or moments) divided by sum of driving forces(or moments).

The underlined figure shows the embankment with potential circular failure surface spitted into number of slices to be used in method of slices. The following derivation shows the calculation of factor of safety for effective and total stress conditions using methods of slices.

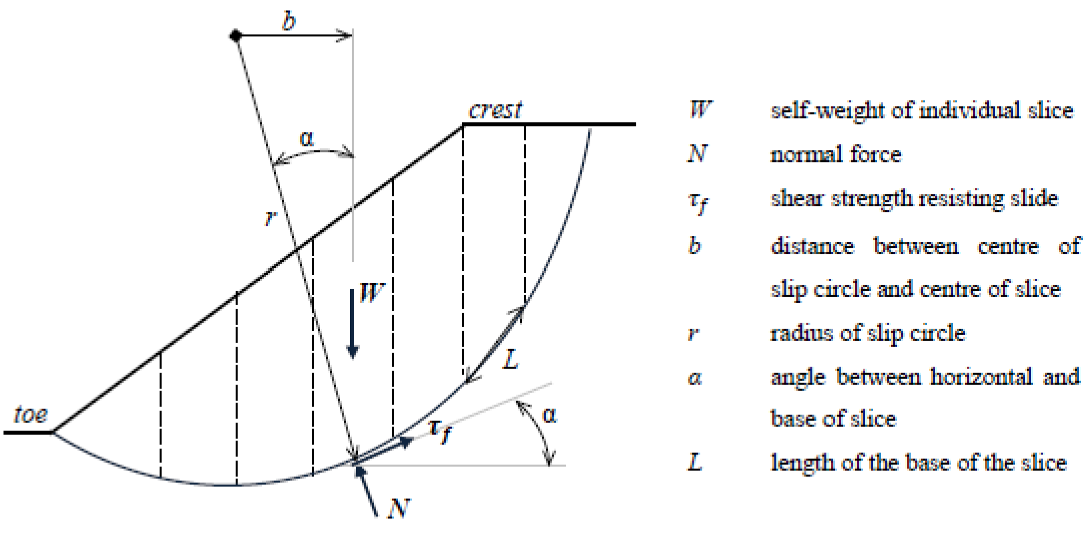


Fig.8 Method of slices

Driving moment (M_b) about centre of slip circle:

$$M_d = \sum Wb \qquad 4.1$$

Where:

$$b = r \sin \alpha \qquad 4.2$$

Substitution the value of b in M_d :

$$M_d = \sum Wr\sin\alpha \tag{4.3}$$

Resisting moment about centre of slip circle:

$$M_r = \sum r\tau_f = \sum \tau_f \tag{4.4}$$

The shear force is equal to mobilized shear stress multiplied by slice area:

$$\tau_f = \tau_{mob}L \tag{4.5}$$

Substituting the above value into equation:

$$M_r = r\sum \tau_{mob}L \tag{4.6}$$

The resisting moment can further be written as below using equation:

$$M_r = r\sum \frac{\tau_f L}{F} \tag{4.7}$$

By rearranging we get:

$$F = \frac{\sum \tau_f L}{\sum W\sin\alpha} \tag{4.8}$$

In terms of total stresses:

The shear stress can be denoted by Mohr-Coulomb's formula in terms of total stress:

$$\tau_f = c + \sigma\tan\varphi \tag{4.9}$$

Using equation we get:

$$F = \frac{\sum (c + \sigma\tan\varphi)L}{\sum W\sin\alpha} \tag{4.10}$$

Since $\sigma = \frac{N}{L}$, we get:

$$F = \frac{\sum cL + N\tan\phi}{\sum W\sin\alpha} \tag{4.11}$$

In terms of effective stresses:

The shear stress can be denoted by Mohr-Coulomb's formula in relation of effective stress:

$$\tau_f = c' + \sigma'\tan\phi' \tag{4.12}$$

Using equation we get:

$$F=\frac{\sum\left(c'+\sigma'\tan\phi'\right)L}{\sum W\sin\alpha} \qquad 4.13$$

Since $\sigma'=\frac{N}{L}-u$, we finally get:

$$F=\frac{\sum\left(c'\mathrm{L}+\left(N-uL\right)\tan\phi'\right)}{\sum W\sin\alpha} \qquad 4.14$$

Table 2. Assumptions in different Methods

Method	**Assumption**
Ordinary method of cells	Interslice forces are neglected
Bishop's simplified/modified	Resultant interslice forces are horizontal. There are no interslice shear forces.
Janbu's simplified	Resultant interslice forces are horizontal. An empirical correction factor is used to account for interslice shear forces.
Janbu's generalized	An assumed line of thrust is used to define the location of the interslice normal force.
Spencer	The resultant interslice forces have constant slope throughout the sliding mass.
Morgenstern-Price	The direction of the resultant interslice forces is defined using an arbitrary function. The fractions of the function value needed for force and moment balance is computed.
Corps of Engineers	The resultant interslice force is either parallel to the ground surface or equal to the average slope from the beginning to the end of the slip surface..
Lowe and Karafiath	The direction of the resultant interslice force is equal to the average of the ground surface and the slope of the base of each slice.
Sarma	The shear strength criterion is applied to the shears on the sides and bottom of each slice. The inclinations of the slice interfaces are varied until a critical criterion is met

4.2 Limit Analysis Method

The limit analysis assumes soil as rigid, perfectly plastic material following associated flow rule. Without carrying elasto-plastic analysis, it can provide solutions to many problems. It is based on the bound theorems of classical plasticity theory. Drucker *et al.,*1951; Drucker and Prager, 1952 states, “The general procedure of limit equilibrium is to assume a kinetically admissible failure mechanism for an upper bound solution or a statically admissible stress field for a lower bound solution , and the objective function will be optimized with respect to control variables”. Later Michalowski, 1995; Donald and Chen,1997 stated, “ Early efforts of limit analysis were merely made on using direct algebraic methods or analytical methods to obtain solutions for slope stability problems with simple geometry and soil profile. Since closed form solutions for most practical problems are not available, later attention has been shifted to employing the slice techniques in traditional limit equilibrium to the upper bound limit analysis”.

Limit analysis is based on two theorems:

1. The lower bound theorem states, “Any statically admissible stress field will provide a lower bound estimate of the true collapse load”.
2. The upper bound theorem states, “When the power dissipated by kinematically admissible velocity field is equated with the power dissipated by the external loads, then the externa; loads are upper bounds on the true collapse load” (Drucker and Prager, 1952).

4.3 Finite Element Analysis

In 1970, Finite Element Method was first applied to slope stability analysis in geotechnical practices. It involves Strength Reduction Method (SRM). Factor of safety(FOS) of a slope is “ratio of actual soil shear strength to minimum shear strength required to prevent failure”. Duncan, 1996 stated FOS as, “factor by which soil shear strength must be reduced to bring a slope to the verge of failure”. In SRM technique, for slope material, elasto-plastic strength is assumed. The shear strength of material are reduced gradually until collapse occurs.

Slope failure occurs when material shear strength is unable to resist the driving shear stresses. Factor of safety is used to assess degree of stability of slopes. Factor of safety greater than 1 means slope is stable while factor of safety less than 1 means slope is unstable. Numerically it is represented as:

$$FOS = \frac{\tau}{\tau_f} \qquad 4.15$$

Where τ is the shear strength of the slope material, which is calculated through Mohr-Coulomb criterion as:

$$\tau = \mathrm{c} + \sigma_n tan\ \phi \qquad 4.16$$

and τ_f is the shear stress on the sliding surface. It can be calculated as:

$$\tau_f = \mathrm{c}_f + \sigma_n tan\ \phi_f \qquad 4.17$$

where the factored shear strength parameters c_f and ϕ_f are:

$$c_f = \mathrm{c}\ /\ SRF \qquad 4.18$$

$$\phi_f = tan^{-1}\left(tan\phi\ /\ SRF \right) \qquad 4.19$$

Where SRF is **strength reduction factor**. This method has been referred to as the '**shear strength reduction method**'. To achieve the correct SRF, it is essential to trace the value of FOS that will just cause the slope to fail.

4.3.1 Basic Algorithm of Strength Reduction Method Used in Phase2

For Mohr-Coulomb materials, the steps for systematically searching for the critical factor of safety value, F, which brings a previously stable slope to the verge of failure, are as follow

Step 1: Develop an FE model of a slope, using the deformation and strength properties established for the slope materials. Compute the model and record the maximum total deformation in the slope.

Step 2: Increase the value of F and calculate factored Mohr-Coulomb material parameters as described above. Enter the new strength properties into the slope model and re-compute. Record the maximum total deformation.

Step 3: Repeat Step 2, using systematic increments of F, until the FE model does not converge to a solution, i.e. continue to reduce material strength until the slope fails. The critical F value just beyond which failure occurs will be the slope factor of safety.

(For a slope that is initially unstable, factor of safety values in steps 2 and 3 must be reduced until the FE model converges to a solution.

4.3.2 Advantages of the finite element method

The advantages of a FE approach to slope stability analysis over traditional limit equilibriummethods can be summarized as follows:

(a) No assumption needs to be made in advance about the shape or location of the failure surface. Failure occurs `naturally' through the zones within the soil mass in which the soil shear strength is unable to sustain the applied shear stresses.(Griffiths, 1999).

(b) Since there is no concept of slices in the FE approach, there is no need for assumptions about slice side forces. The FE method preserves global equilibrium until `failure' is reached.

(c) The method can be applied with complex slope configurations and soil deposits in two or three dimensions to model virtually all types of mechanisms.

(d) General soil material models that include Mohr-Coulomb and numerous others can be employed.

(e) The critical failure mechanism developed can be extremely general and need not be simple circular or logarithmic spiral arcs.

(f) The method can be extended to account for seepage induced failures, brittle soil behaviors, random field soil properties, and engineering interventions such as geo-textiles, soil nailing, drains and retaining walls (Swan et al. 1999).

(g) If realistic soil compressibility data are available, the FE solutions will give information about deformations at working stress levels. (Griffiths, 1999).

(h) The FE method is able to monitor progressive failure up to and including overall shearfailure. (Griffiths, 1999).

By examining the merits of finite element analysis over limit-equilibrium methods for slope design and analysis, the present case study was done using Shear Strength Reduction Method.

Chapter 5

CASE STUDY

For the present study Tailing dam at Jaduguda is selected. The site has a Longitude $86^0$20’ E and Latitude 22^0 37’ N which is situated about 1 km from the sand-slime separation unit.

5.1 Dam Description

The dam is constructed in six phases by centerline method which is intermediate between upstream and downstream construction. It is raised by spreading and compacting successive layers of materials on the crest, on the upstream shoulder, and on the downstream slope.The total height of the dam is 37m which was raised in 6 stages. And the elevation at the top and bottom is 159m and 123m respectively.

The tailings dam under consideration is a zoned dam raised in six stages using the center line method and having three different types of fill materials. The zones, from upstream to downstream, consisted of (i) impervious material (ii) compacted tailings and (iii) pervious random fill. An inclined chimney drain and a connecting horizontal filter are provided to keep most of the dam section on the downstream side dry.

The slurry is discharged into the pond by pumping it from sand slime separation unit to the dam through pipe line. Slurry gets settled into the pond behind the embankment and the clear water is decanted though hume pipe situated at the bottom of the wells.The tailing dam falls in seismic zone II as per IS 1893-1975.

5.2 Material Properties

The properties for the various dam material used in the analysis is given in Table. The tailing material is taken as saturated.

Table 3. Material Properties

	G (kN/m^2)	**ν**	**E** (kN/m^2)	**γ(sat)** (kN/m^3)
Pond Tailing	45640	0.35	123230	19.2
Compacted Tailing	95390	0.35	257550	19.2
Impervious Material	53560	0.40	149970	19.6
Random Material	190250	0.30	494650	21.1
Rock-fill material	190250	0.30	494650	21.1
Foundation Rock	217350	0.20	521640	22.2

5.3 Modelling

The Mohr-Coulomb failure criterion is the most popular failure criterion in soil mechanics and was first presented in 1773 by Charles-Augustin de Coulomb, and was the first criterion to account for the hydrostatic stress and hence this model is used in this analysis.

The following are essential points of mohr's strength theory:

1. Material fails essentially by shear. The critical shear stress causing failure depends upon the properties off the material as well as on normal stress on the failure plane.
2. The ultimate strength of the material is determined by the stress on the potential failure plane.
3. When the material is subjected to three dimensional principle stress the intermediate principle stress does not have any influence on the strength of material.

According to the Mohr-Coulomb criterion the shear strength increases with increasing normal stress,

$$|\tau_f| = c - \sigma' \tan\phi \qquad 5.1$$

where τ is the shear stress on the failure plane, c the material cohesion, σ' the normal effective stress on the failure surface and Φ the angle of internal friction. In Figure the Mohr-Coulomb criterion is illustrated with help of the Mohr circle.

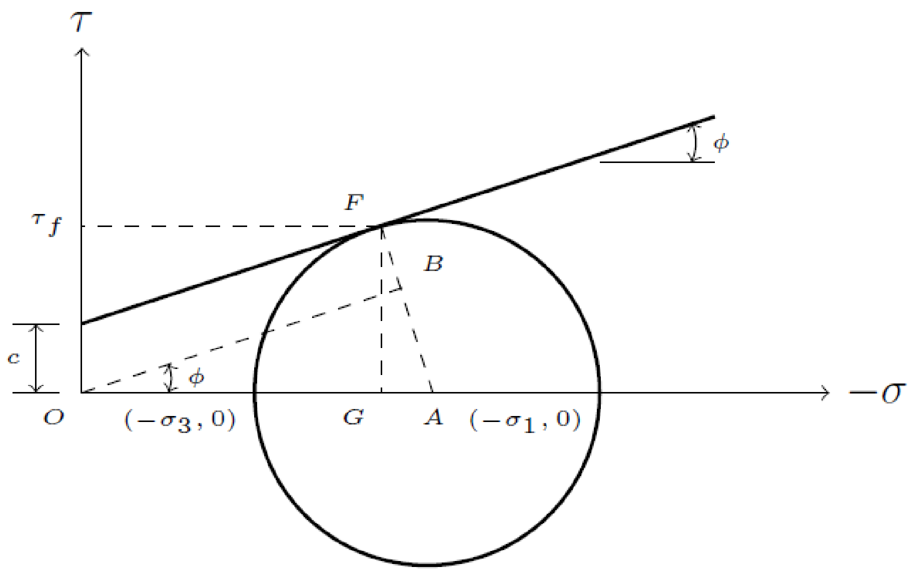


Fig.9 The Mohr-Coulomb criterion and Mohr's circle

With Figure as reference the Mohr-Coulomb criterion can be derived following.

$$-\frac{\sigma_1-\sigma_3}{2}=AB+BF \tag{5.2}$$

That can be rewritten as,

$$-\frac{\sigma_1-\sigma_3}{2}=OAsin\phi+\mathrm{c}\cos\phi \tag{5.3}$$

Inserting that $OA=-\frac{1}{2}(\sigma_1-\sigma_3)$ into above equation we obtained

$$-\frac{\sigma_1-\sigma_3}{2}=-\frac{\sigma_1+\sigma_3}{2}OAsin\phi+\mathrm{c}\cos\phi \tag{5.4}$$

Where σ_1 and σ_3 are the major and minor principal stresses respectively. It also clear from above equation that the Mohr-Coulomb criterion is independent of the effects of the intermediate principal stress (Desai and Siriwardane; 1984). The expression in above can be projected on the deviator- or π plane where it takes the form of an irregular hexagon illustrated in Figure.

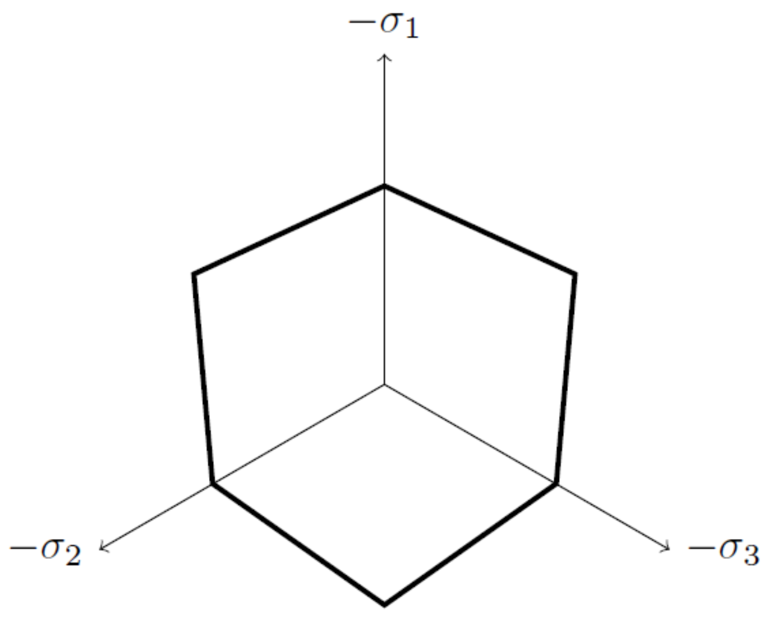


Fig.10 The Mohr-Coulomb criterion on the deviator plane.

The Mohr-Coulomb criterion states that the yield strength in compression is higher than the yield strength in tension. The Mohr-Coulomb criterion is expressed in terms of σ_1 and σ_3, and as mentioned not including σ_2. Therefore it is inconvenient to express the

Mohr-Coulomb criterion with the components of the stress tensor and consequently it becomes difficult to describe the criterion with the stress invariants (I_1; I_2; I_3). The Mohr-Coulomb yield criterion is instead commonly described with (I_1; J_2;θ),

$$I = \sigma_1 + \sigma_2 + \sigma_3 \qquad 5.5$$

$$J_2 = \frac{1}{2}\left(s_1^2 + s_2^2 + s_3^2\right) \qquad 5.6$$

$$\theta = -\frac{1}{3}\sin^{-1}\left(-\frac{3\sqrt{3}}{2}\frac{J_3}{J_2^{3/2}}\right) \qquad 5.7$$

$$\text{where } -\pi/6 \le \theta \le \pi/6 \text{ and } J_3 = \frac{1}{3}\left(s_1^3 + s_2^3 s_3^3\right) \qquad 5.8$$

This leads to the convectional form of Mohr-Coulomb criterion in a three-dimensional stress space as, (Desai and Siriwardane; 1984)

$$f\left(I_1, \mathrm{J}_2, \theta\right) = I_1 \sin\varphi + \sqrt{J_2}\cos\theta - \frac{\sqrt{J_2}}{3}\sin\phi\sin\theta - c\cos\phi = 0 \qquad 5.9$$

One setback to the Mohr-Coulomb criterion is that the shape of the yield surface leads to numerical difficulties when treating the plastic flow at corners of the yield surface.

5.4 Analysis

The safety analysis in phase2 was executed by reducing the strength parameters of the soil. This process is termed as Shear Strength reduction. A safety analysis was performed after each individual calculation phase and thus for each construction stage.

The factor of safety (FOS) of a soil slope is defined as the number by which the original shear strength parameters must be divided in order to bring the slope to the point of failure. The factored shear strength parameters c_f’ and Φ_f’, are therefore given by:

$$c_f^{'} = \frac{c}{FOS} \qquad 5.10$$

$$\phi_f^{'} = \arctan\left(\frac{\tan\phi^{'}}{FOS}\right) \qquad 5.11$$

This method has been referred to as the `shear strength reduction technique' (e.g. Matsui & San, 1992) and allows for the interesting option of applying different factors of safety to the c’ and tan ϕ' terms.

It is done using this method due to previously mentioned advantages.

(a) No assumption needs to be made in advance about the shape or location of the failure surface. Failure occurs `naturally' through the zones within the soil mass in which the soil shear strength is unable to sustain the applied shear stresses. (Griffiths, 1999).

(b) Since there is no concept of slices in the FE approach, there is no need for assumptions about slice side forces. The FE method preserves global equilibrium until `failure' is reached.

(c) The method can be applied with complex slope configurations and soil deposits in two or three dimensions to model virtually all types of mechanisms.

(d) The critical failure mechanism developed can be extremely general and need not be simple circular or logarithmic spiral arcs.

(e) The FE method is able to monitor progressive failure up to and including overall shearfailure. (Griffiths, 1999).

5.5 Geometry

The cross section of the Jaduguda tailing damis as given below along with scale in meter:

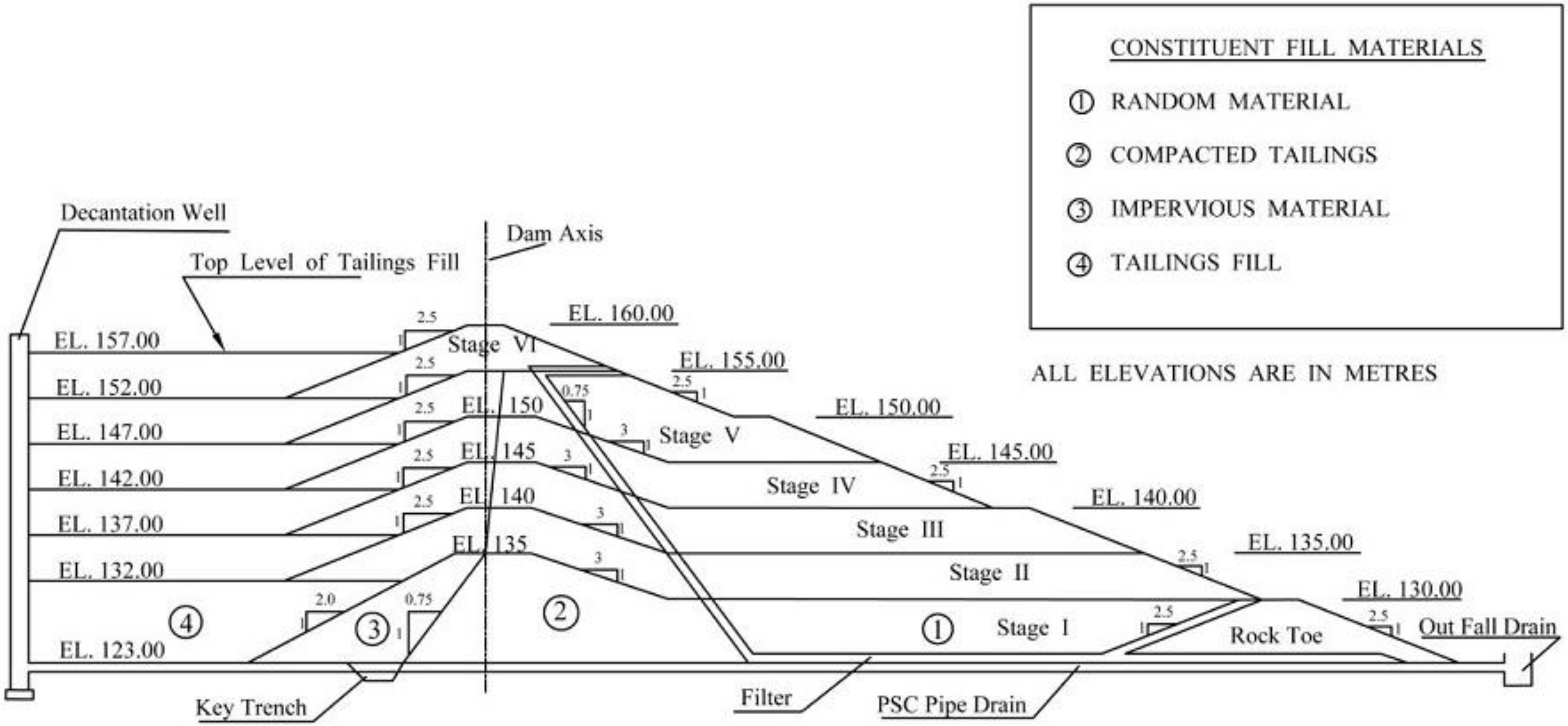


Fig.11 Cross section of the Jaduguda tailing dam

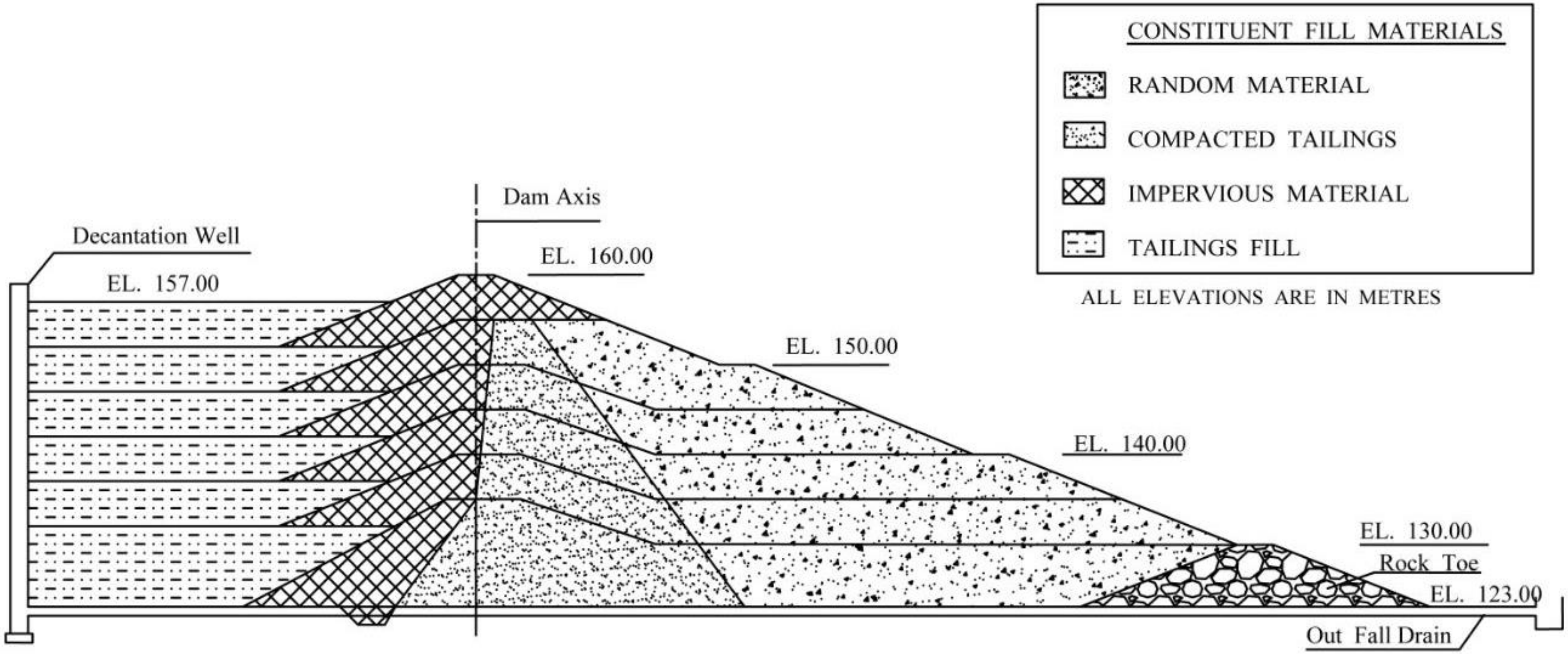


Fig.12 Cross section of the Jaduguda tailing dam showing construction materials

Chapter 6

STATIC ANALYSIS

Analysis of the tailing dam is carried out for static loads due to self-weight, tailings and uplift using Shear Strength Reduction Method as discussed. Static deformations of dam are of interest because excessive deformations can lead to loss of free board and danger of over topping. Excessive spreading may lead to loss of free board and danger of over topping. Excessive spreading may lead to longitudinal cracking and adversely affect stability. Differential settlement between the core and shell can lead to stress reduction in the core and may result in the hydraulic fracture.

6.1 Geometry

The cross section of the Jaduguda tailing dam is as given below along with scale in meter:

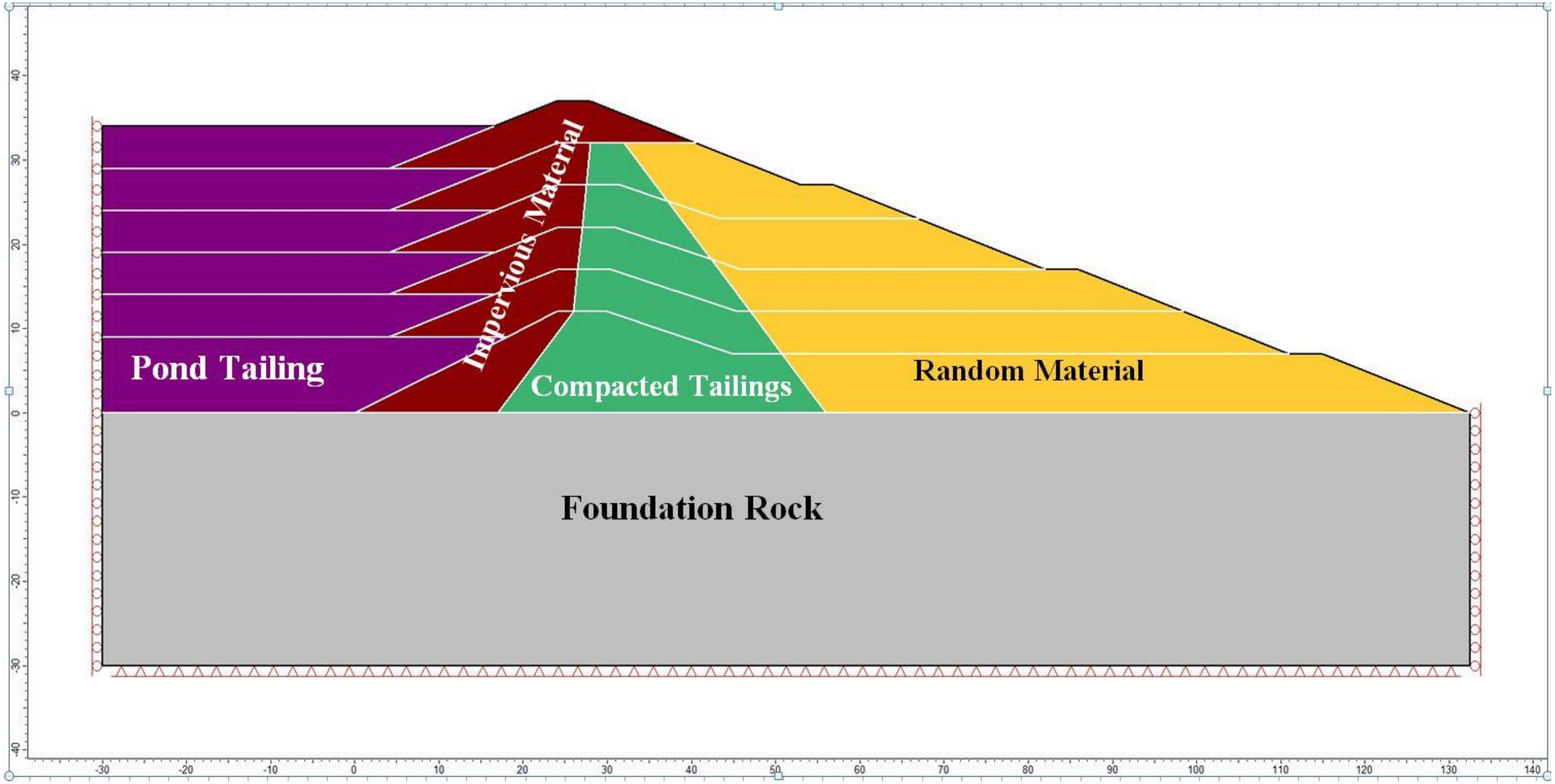


Fig.13Cross section of the Jaduguda tailing dam

6.2 Sequence of Construction and the failure surfaces

The following figures show the six phases(with and without tailings) of construction along with its failure surface analyzed through Strength Reduction Technique (SRM Technique).

I – STAGE

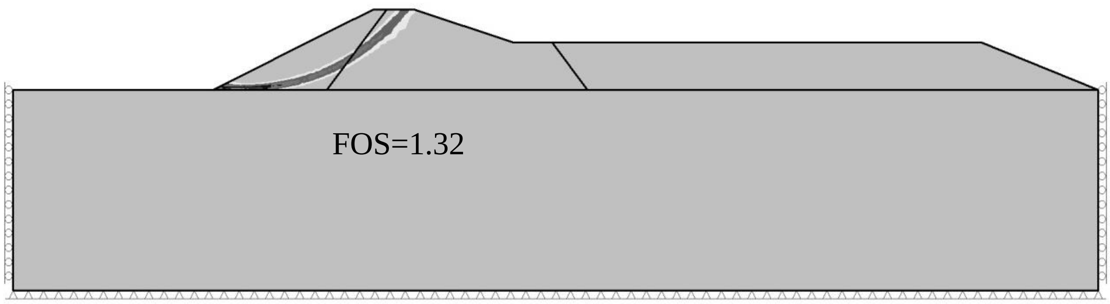


Fig.14 Critical failure surface for I-stage (without tailings) with FOS=1.32

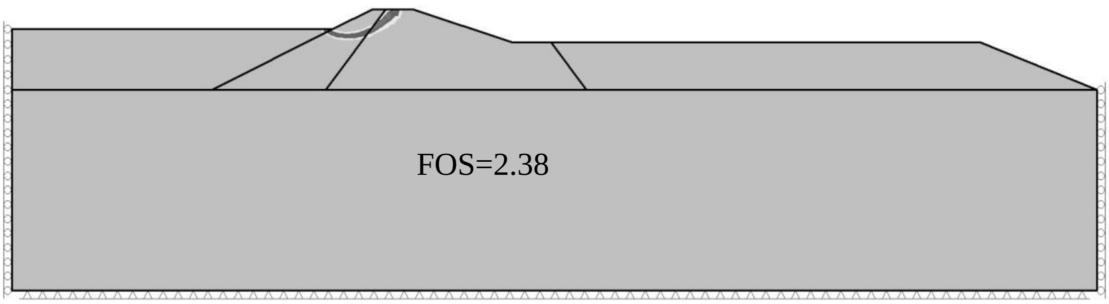


Fig.15 Critical failure surface for I-stage (with tailings) with FOS=2.38

II – STAGE

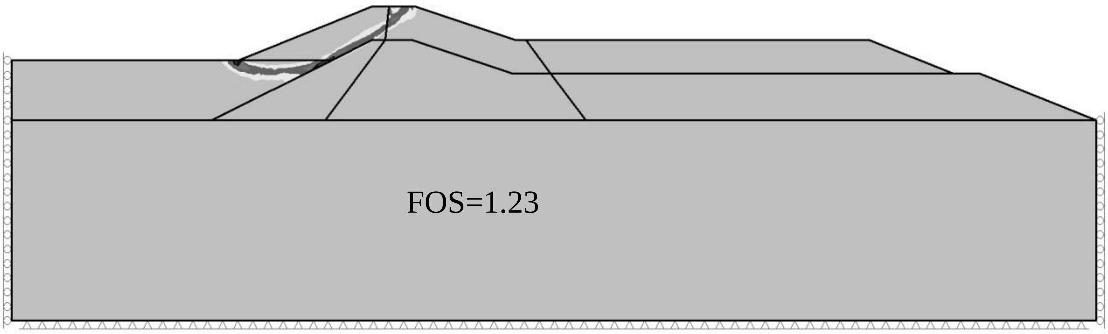


Fig.16Critical failure surface for II - stage (without tailings) with FOS=1.23

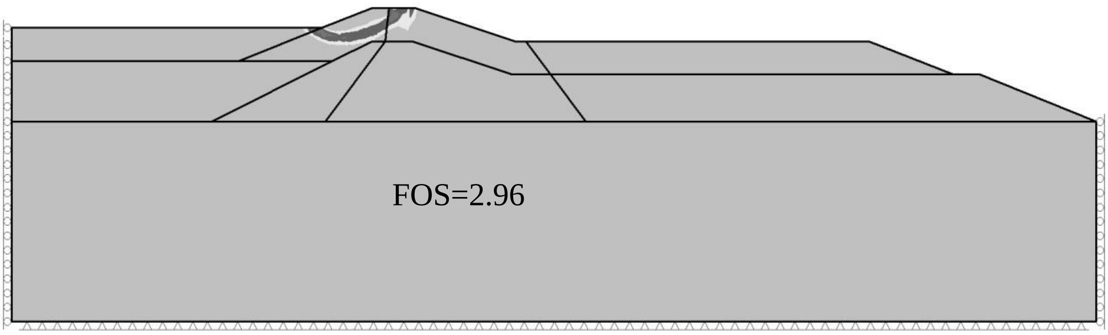


Fig.17Critical failure surface for II - stage (with tailings) with FOS=2.96

III – STAGE

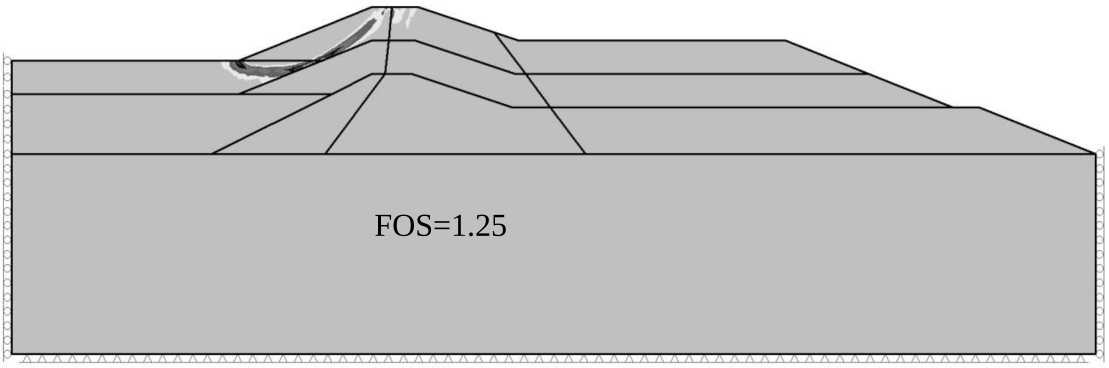


Fig.18Critical failure surface for III-stage (without tailings) with FOS=1.25

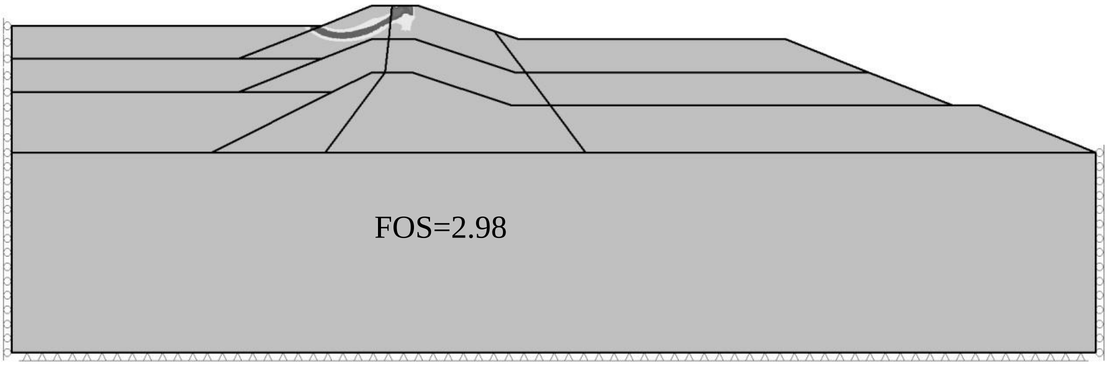


Fig.19Critical failure surface for III – stage (with tailings) with FOS=2.98

IV – STAGE

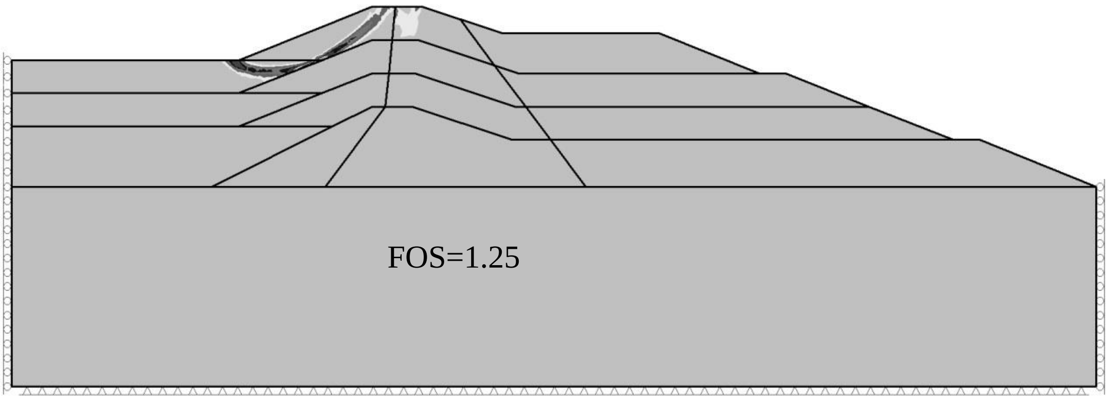


Fig.20Critical failure surface for IV-stage (without tailings) with FOS=1.25

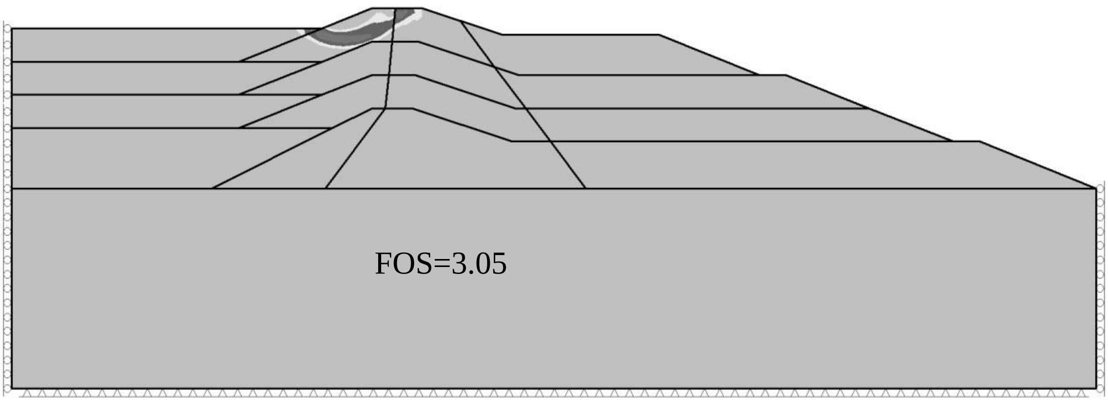


Fig.21Critical failure surface for IV - stage (with tailings) with FOS=3.05

V – STAGE

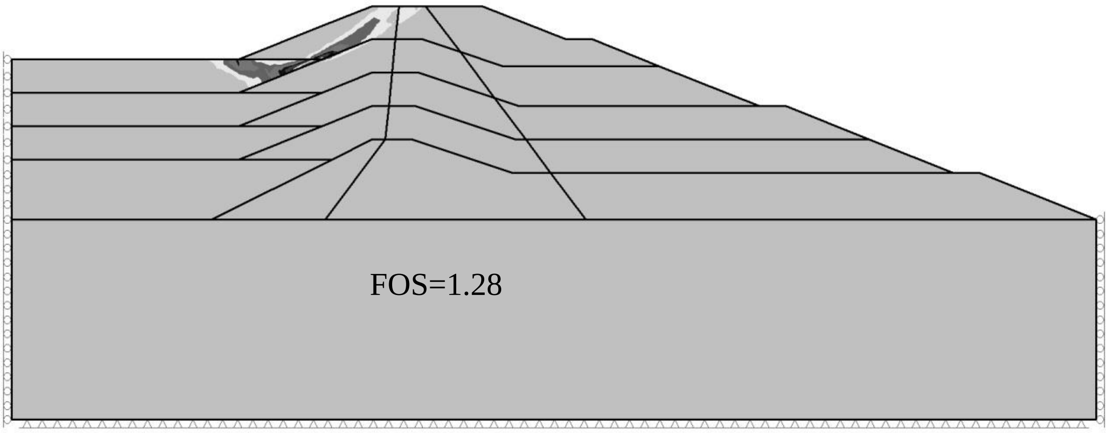


Fig.22Critical failure surface for V - stage (without tailings) with FOS=1.28

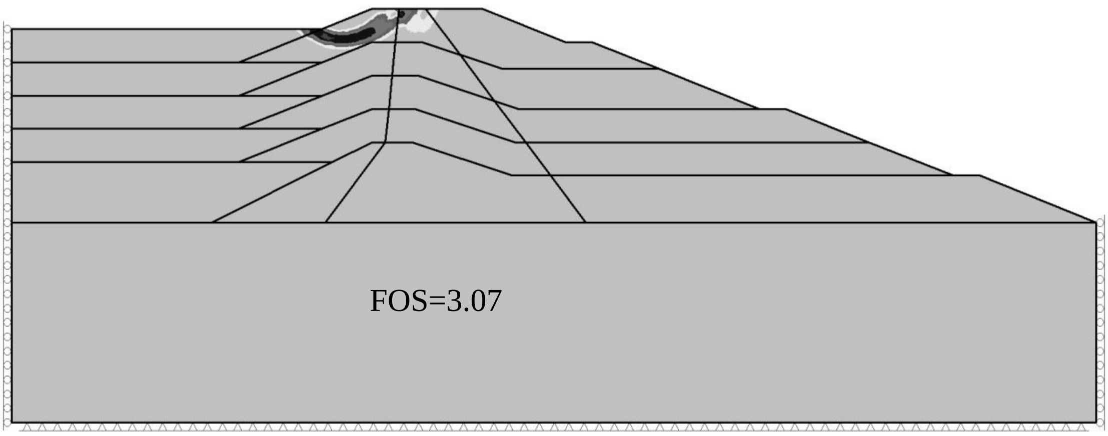


Fig.23Critical failure surface for V - stage (with tailings) with FOS=3.07

VI – STAGE

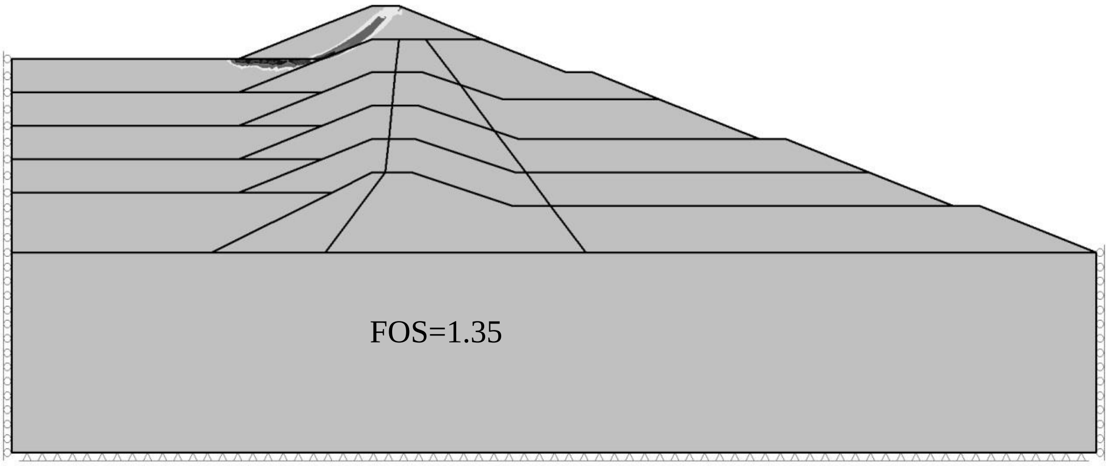


Fig.24Critical failure surface for VI-stage (without tailings) with FOS=1.35

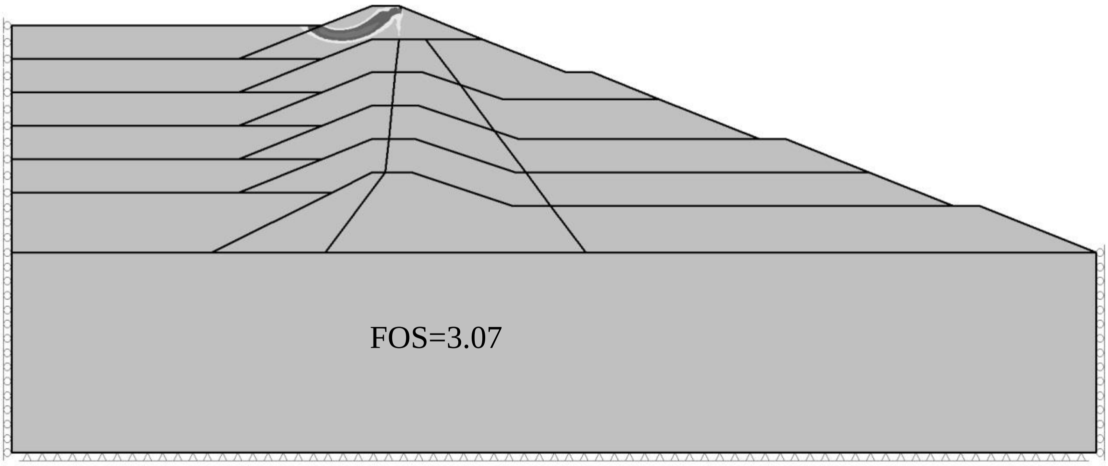


Fig.25Critical failure surface for VI-stage (with tailings) with FOS=3.07

6.3 Results of analysis

The factor of safety of various stages in static condition can be summarized in following table:

Table 4 FOS of various Stages in static condition with SRM

STAGES	WITHOUT TAILINGS	WITH TAILINGS
Stage I	1.32	2.38
Stage II	1.23	2.96
Stage III	1.25	2.98
Stage IV	1.25	3.05
Stage V	1.28	3.07
Stage VI	1.35	3.07

It can be seen from above table that at every stage, in static conditions, the factor of safety is greater than 1. Therefore it is concluded that the dam is safe under Static Conditions.

Chapter 7

Pseudo-Static Analysis

Analysis of the tailing dam was also carried for pseudo-static condition with same boundary conditions. The numerical simulations have been done using geotechnical software PHASE-2. As the dam lies in zone II therefore the acceleration value for triggering was adopted as 0.06g. The factor of safety (FOS) is tabulated in the table below:

Table 5 FOS of various Stages in Pseudo-Static condition with SRM

STAGES	WITHOUT TAILINGS	WITH TAILINGS
Stage I	1.15	2.01
Stage II	1.03	2.41
Stage III	1.05	2.42
Stage IV	1.05	2.46
Stage V	1.07	2.49
Stage VI	1.13	2.54

It can be seen from above table that at every stage, in ‘without tailings’ conditions, the factor of safety is almost equal to 1, hence the dam is on the verse of failure. While at every stage, in ‘with tailings’ conditions, the factor of safety is much greater than 1, therefore the dam is safe.

Chapter 8

Finite element method Vs Limit equilibrium method – A Comparative Study

The main objective of this study is comparing results of static and pseudo-static analyses by the Finite Element Method (FEM) using the Strength Reduction Technique with those from various Limit Equilibrium Methods (LEM) such as Bishop Simplified, Janbu Simplified, Janbu Corrected, Spencer, Corps of Engineers-1, Corps of Engineers-2, Lowe Karafiath and Morgenstern Price, considering both circular and non-circular failure surfaces. The results are presented in the form of normalised plots for the following cases: (i) Stagewise (With Tailings) (ii) Stagewise (Without Tailings) (iii) Considering circular failure surface (iv) Considering non-circular failure surfaces.

8.1 Comparative Study Without Considering Tailings Fill

Table 6 Static without Tailings considering circular failure surface

METHODS	STAGE I	STAGE II	STAGE III	STAGE IV	STAGE V	STAGE VI
SRM	1.32	1.23	1.25	1.25	1.28	1.35
Bishop Simplified	1.259	1.25	1.29	1.309	1.289	1.284
Janbu Simplified	1.165	1.187	1.235	1.25	1.237	1.231
Janbu Corrected	1.233	1.263	1.323	1.348	1.325	1.319
Spencer	1.261	1.246	1.287	1.306	1.286	1.282
Corps Of Engineers-1	1.271	1.246	1.312	1.351	1.313	1.308
Corps Of Engineers-2	1.283	1.268	1.327	1.366	1.329	1.324
Lowe Karafiath	1.262	1.249	1.299	1.357	1.302	1.295
Morgenstern Price	1.269	1.246	1.285	1.306	1.285	1.28

Table 7 Static without Tailings considering non-circular failure surface

METHODS	STAGE I	STAGE II	STAGE III	STAGE IV	STAGE V	STAGE VI
SRM	**1.32**	**1.23**	**1.25**	**1.25**	**1.28**	**1.35**
Bishop Simplified	1.23	1.186	1.233	1.233	1.228	1.229
Janbu Simplified	1.151	1.141	1.203	1.202	1.197	1.197
Janbu Corrected	1.223	1.223	1.289	1.289	1.283	1.283
Spencer	1.249	1.222	1.268	1.268	1.269	1.268
Corps Of Engineers-1	1.252	1.228	1.296	1.294	1.275	1.296
Corps Of Engineers-2	1.252	1.238	1.293	1.305	1.294	1.294
Lowe Karafiath	1.245	1.193	1.242	1.257	1.241	1.24
Morgenstern Price	1.242	1.213	1.256	1.259	1.259	1.26

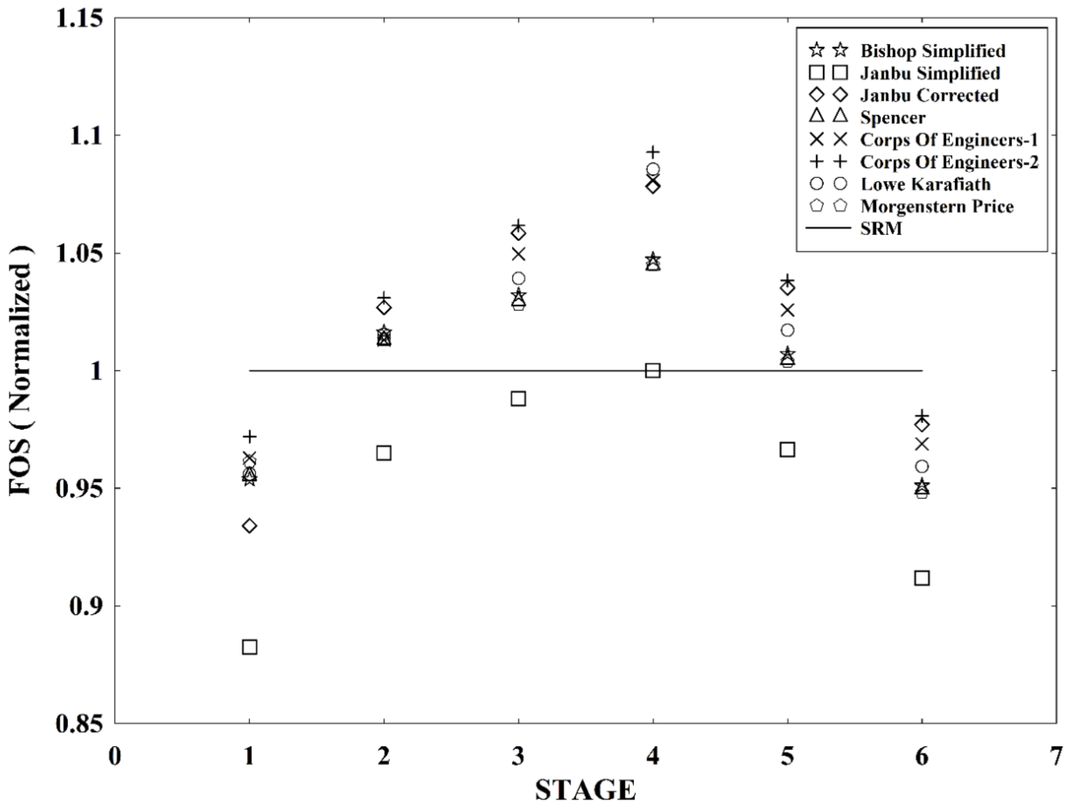
Table 8 Pseudo static without tailings considering circular failure surface

METHODS	STAGE I	STAGE II	STAGE III	STAGE IV	STAGE V	STAGE VI
SRM	**1.15**	**1.03**	**1.05**	**1.05**	**1.07**	**1.13**
Bishop Simplified	1.101	1.05	1.095	1.124	1.092	1.09
Janbu Simplified	1.012	0.991	1.044	1.069	1.043	1.041
Janbu Corrected	1.072	1.055	1.118	1.147	1.118	1.115
Spencer	1.107	1.045	1.092	1.12	1.089	1.087
Corps Of Engineers-1	1.1	1.03	1.093	1.109	1.095	1.091
Corps Of Engineers-2	1.109	1.045	1.104	1.12	1.105	1.101
Lowe Karafiath	1.087	1.031	1.081	1.113	1.085	1.078
Morgenstern Price	1.109	1.046	1.092	1.121	1.089	1.087

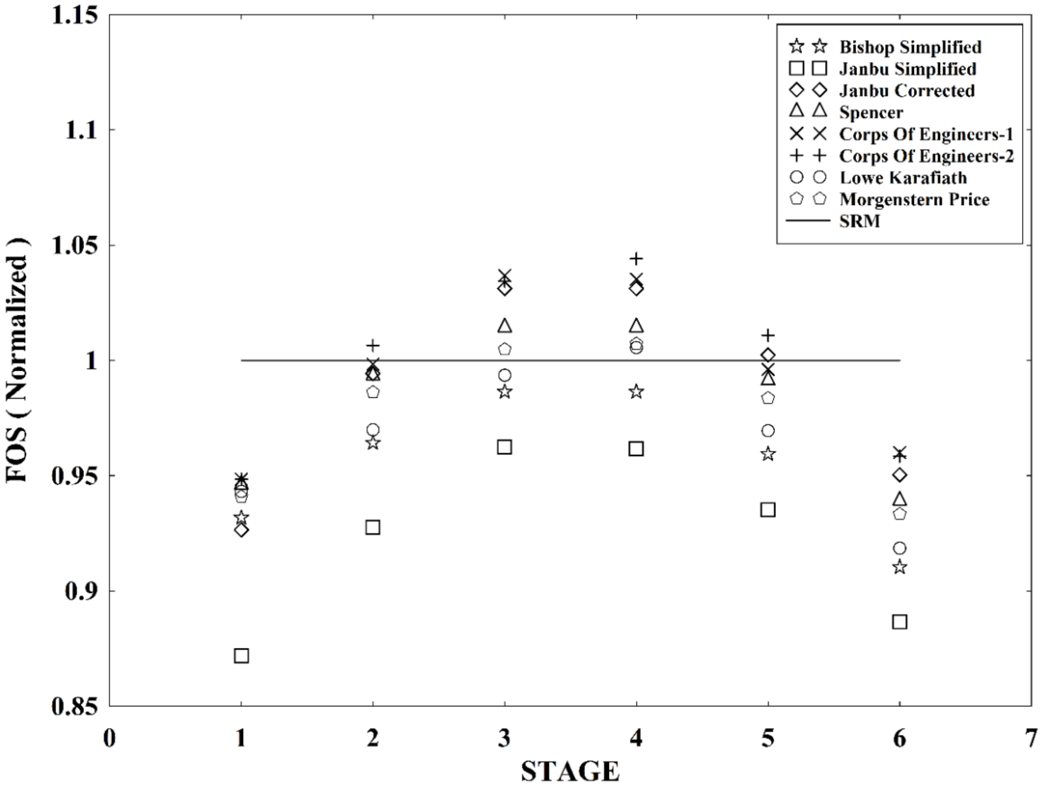
Table 9 Pseudo-static without tailings considering non-circular failure surface

METHODS	STAGE I	STAGE II	STAGE III	STAGE IV	STAGE V	STAGE VI
SRM	**1.15**	**1.03**	**1.05**	**1.05**	**1.07**	**1.13**
Bishop Simplified	1.074	1.002	1.043	1.049	1.043	1.047
Janbu Simplified	1.003	0.961	1.014	1.02	1.014	1.019
Janbu Corrected	1.063	1.027	1.087	1.092	1.086	1.092
Spencer	1.091	1.032	1.077	1.089	1.08	1.08
Corps Of Engineers-1	1.081	1.019	1.065	1.076	1.082	1.08
Corps Of Engineers-2	1.085	1.026	1.083	1.08	1.084	1.081
Lowe Karafiath	1.072	0.996	1.058	1.05	1.046	1.052
Morgenstern Price	1.085	1.027	1.074	1.079	1.073	1.073
SRM	1.15	1.03	1.05	1.05	1.07	1.13

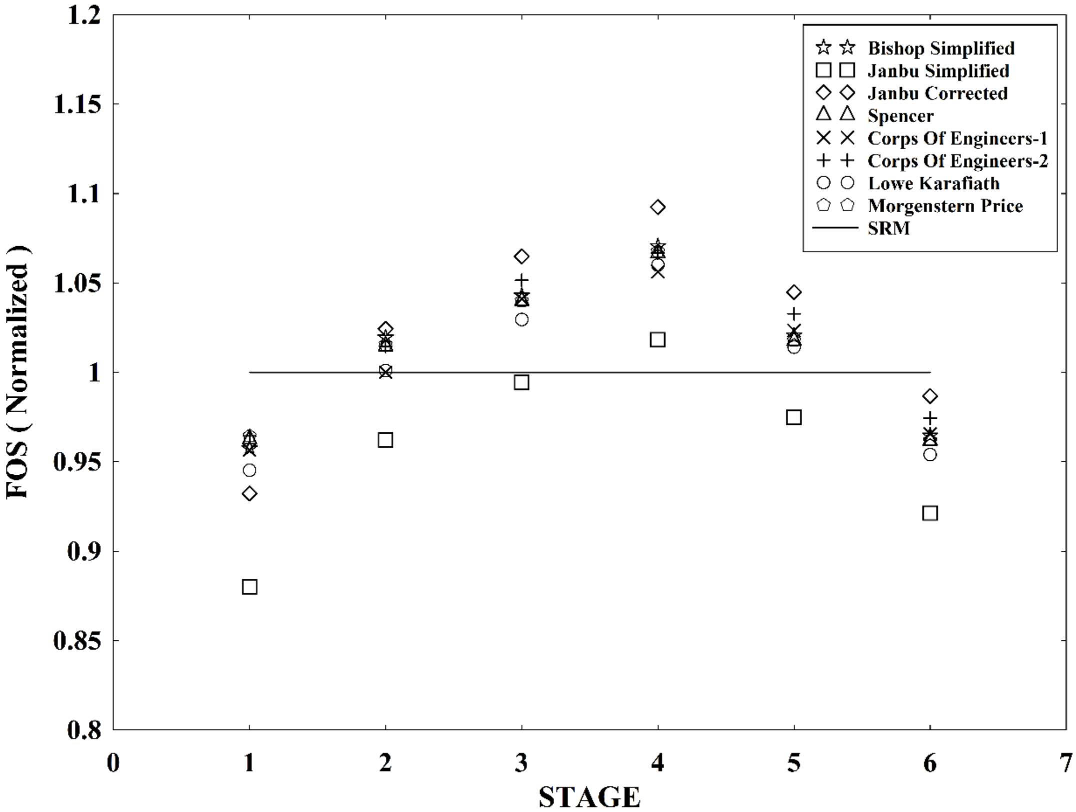
Graphs 3 Static without Tailings considering circular failure surfaces



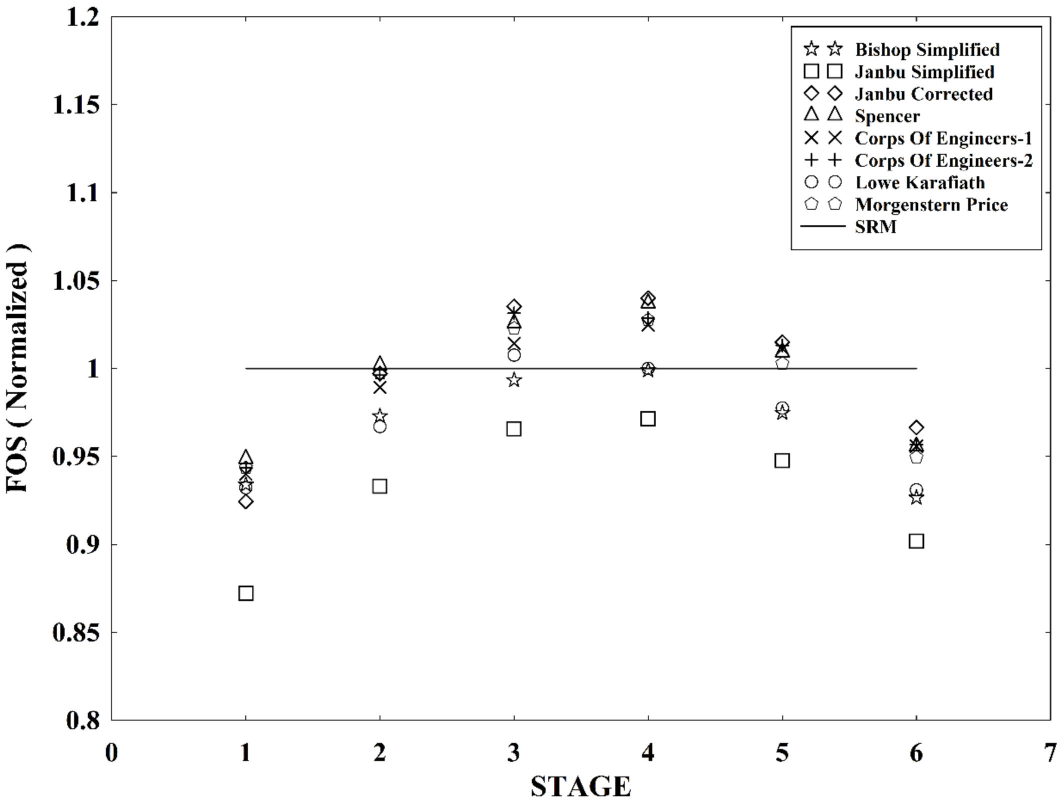
Graphs 4 Static without tailings considering non-circular failure surfaces



Graphs 5 Pseudo-static without tailings considering circular failure surfaces



Graphs 6 Pseudo-static without tailings considering non-circular failure surfaces



8.1.1 Observations

1) Janbu Simplified Method gives lowest Factor of Safety (FOS) among all other considered Limit Equilibrium Methods (LEM).
2) Factor of Safety considering non-circular failure is less than corresponding to Factor of Safety obtained considering circular failure surface among Limit Equilibrium Methods.
3) In stages 1 and 6, Factor of Safety of all Limit Equilibrium are less than those of Strength Reduction Method while in other stages (i.e. 2,3,4 and 5), Factor of Safety obtained considering Limit Equilibrium Methods are closely scattered around Strength Reduction Method.
4) The Factor of Safety for the tailings dam considering tailings fill in the upstream is higher than the Factor of Safety obtained without tailings fill in the upstream.
5) The Variations in Factor of Safety for all Limit Equilibrium Methods with respect to Factor of Safety considering Strength Reduction Method are ± 15.
6) The Factor of Safety by Pseudo-static analysis is less than corresponding static Factor of safety by Static Analysis.

8.2 Comparative Study Without Considering Tailings Fill

Table 10 Static with tailings considering circular failure surfaces

METHODS	STAGE I	STAGE II	STAGE III	STAGE IV	STAGE V	STAGE VI
SRM	**2.38**	**2.96**	**2.98**	**3.05**	**3.07**	**3.07**
Bishop Simplified	2.369	2.886	2.943	2.998	2.995	2.995
Janbu Simplified	2.091	2.548	2.611	2.674	2.715	2.719
Janbu Corrected	2.254	2.757	2.826	2.895	2.935	2.939
Spencer	2.341	2.879	2.936	2.992	2.993	2.993
Corps Of Engineers-1	2.352	2.888	2.94	2.996	3.057	3.061
Corps Of Engineers-2	2.441	2.999	3.056	3.116	3.181	3.045
Lowe Karafiath	2.372	2.93	2.984	3.041	3.072	3.075
Morgenstern Price	2.341	2.882	2.939	2.995	2.993	2.993

Table 11 Static with tailings considering non-circular failure surface

METHODS	STAGE I	STAGE II	STAGE III	STAGE IV	STAGE V	STAGE VI
SRM	**2.38**	**2.96**	**2.98**	**3.05**	**3.07**	**3.07**
Bishop Simplified	2.111	2.657	2.697	2.737	2.777	2.774
Janbu Simplified	1.949	2.494	2.538	2.58	2.632	2.63
Janbu Corrected	2.116	2.695	2.742	2.788	2.853	2.851
Spencer	2.304	2.834	2.877	2.918	2.972	2.971
Corps Of Engineers-1	2.296	2.837	2.879	2.862	2.964	2.998
Corps Of Engineers-2	2.218	2.805	2.854	2.879	2.955	3.096
Lowe Karafiath	2.198	2.678	2.733	2.789	2.912	2.834
Morgenstern Price	2.294	2.826	2.87	2.906	2.961	2.959

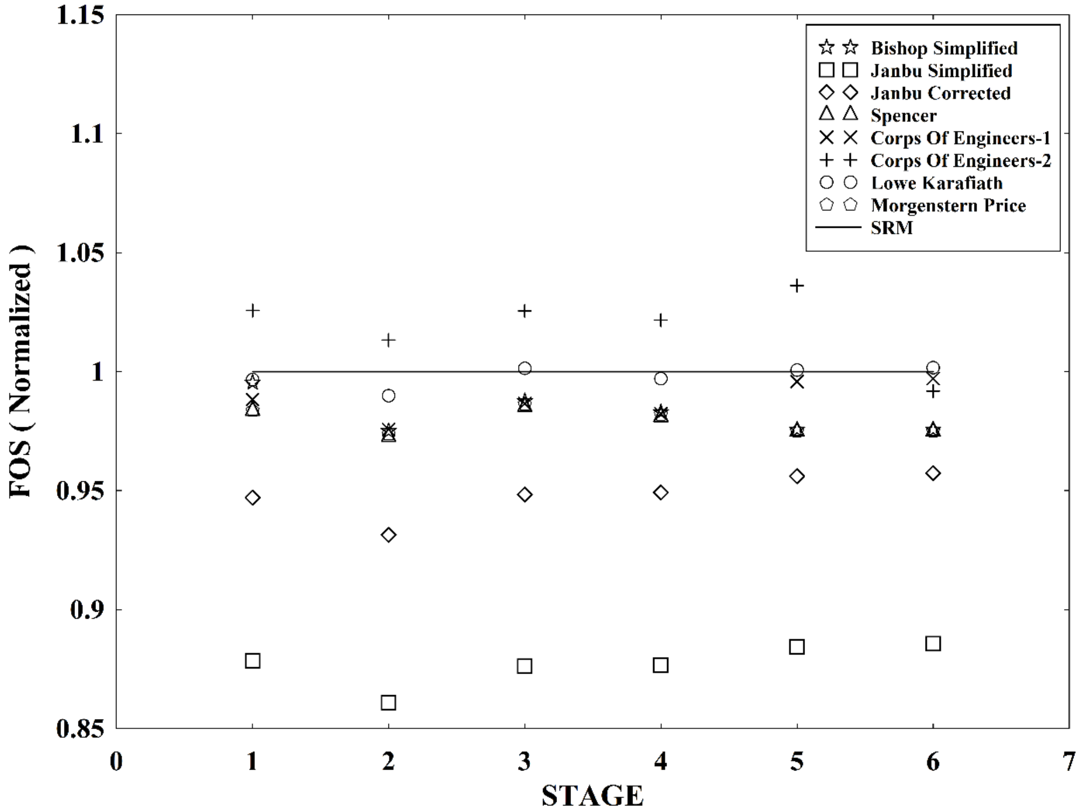
Table 12 Static with tailings considering circular failure surfaces

METHODS	STAGE I	STAGE II	STAGE III	STAGE IV	STAGE V	STAGE VI
SRM	**2.01**	**2.41**	**2.42**	**2.46**	**2.49**	**2.54**
Bishop Simplified	2.011	2.358	2.387	2.416	2.446	2.508
Janbu Simplified	1.751	2.078	2.114	2.149	2.186	2.263
Janbu Corrected	1.893	2.249	2.284	2.319	2.353	2.448
Spencer	1.983	2.349	2.379	2.41	2.441	2.509
Corps Of Engineers-1	1.938	2.293	2.321	2.35	2.379	2.491
Corps Of Engineers-2	2.013	2.379	2.41	2.44	2.47	2.161
Lowe Karafiath	1.955	2.321	2.35	2.379	2.409	2.502
Morgenstern Price	1.982	2.352	2.383	2.413	2.444	2.507

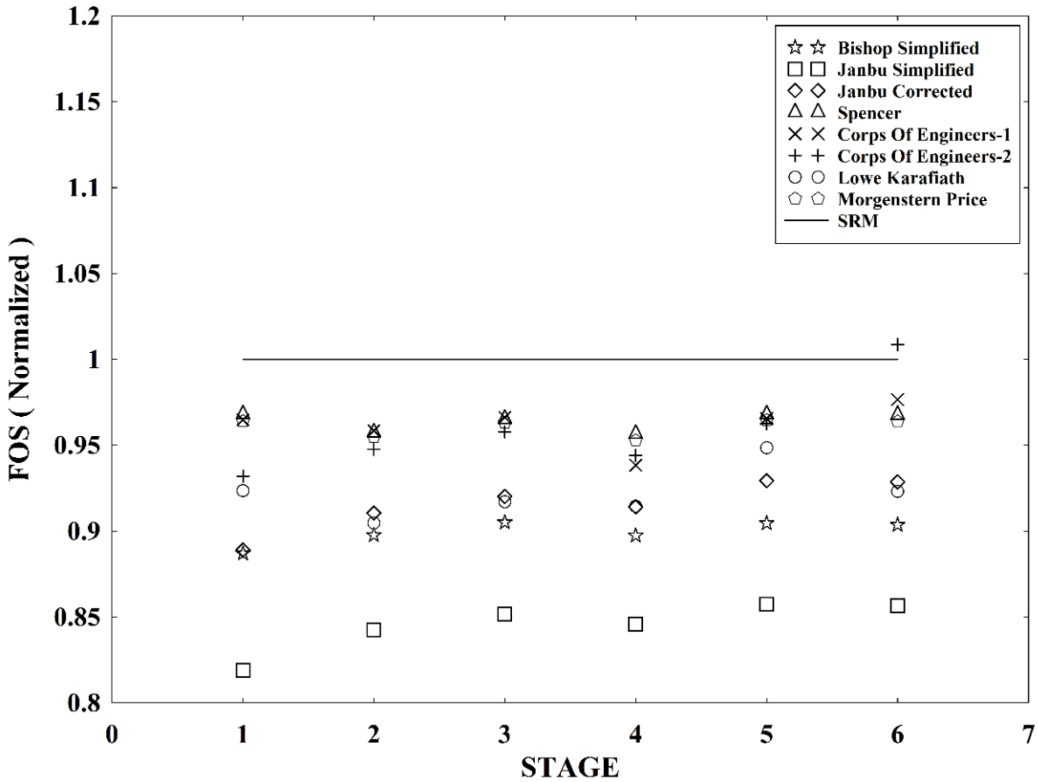
Table 13 Pseudo-static with tailings considering non-circular failure surfaces

METHODS	STAGE I	STAGE II	STAGE III	STAGE IV	STAGE V	STAGE VI
SRM	**2.01**	**2.41**	**2.42**	**2.46**	**2.49**	**2.54**
Bishop Simplified	1.815	2.187	2.204	2.233	2.255	2.323
Janbu Simplified	1.675	2.047	2.074	2.101	2.126	2.204
Janbu Corrected	1.814	2.207	2.237	2.265	2.291	2.387
Spencer	1.956	2.322	2.349	2.375	2.4	2.492
Corps Of Engineers-1	1.869	2.236	2.258	2.285	2.326	2.456
Corps Of Engineers-2	1.878	2.242	2.248	2.301	2.319	2.508
Lowe Karafiath	1.865	2.184	2.185	2.216	2.232	2.373
Morgenstern Price	1.955	2.321	2.352	2.377	2.406	2.496

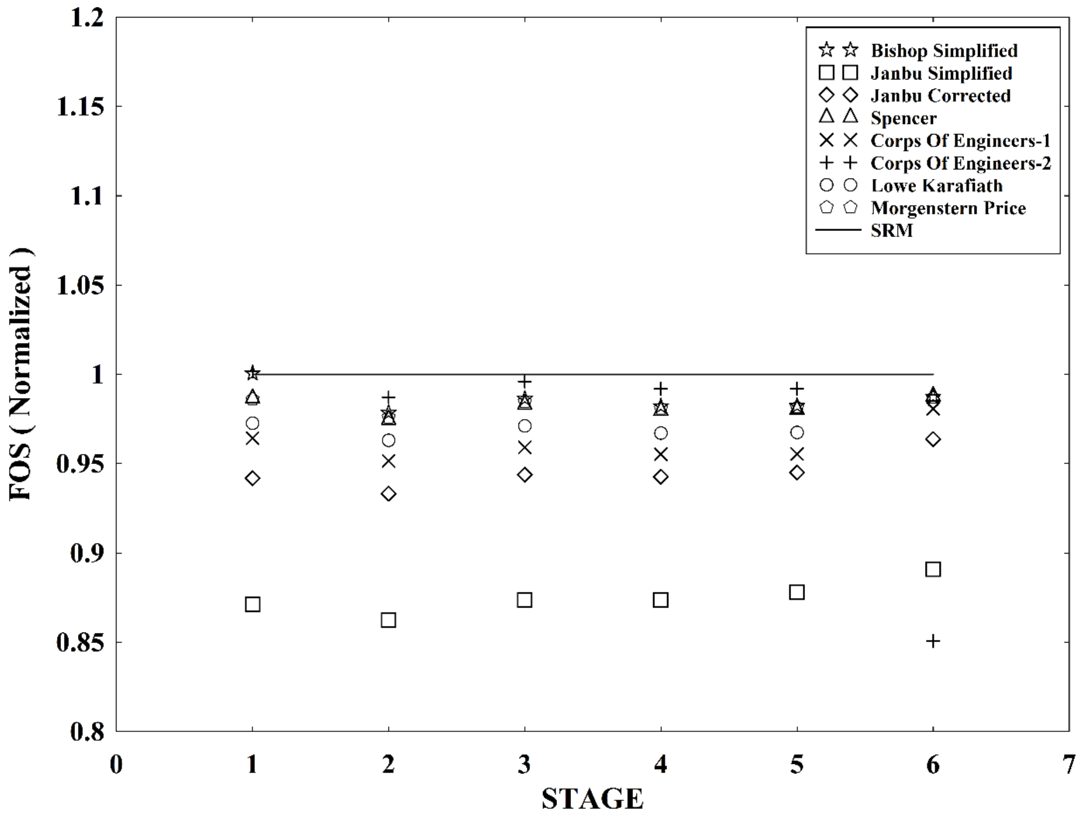
Graphs 7 Static with tailings considering circular failure surfaces



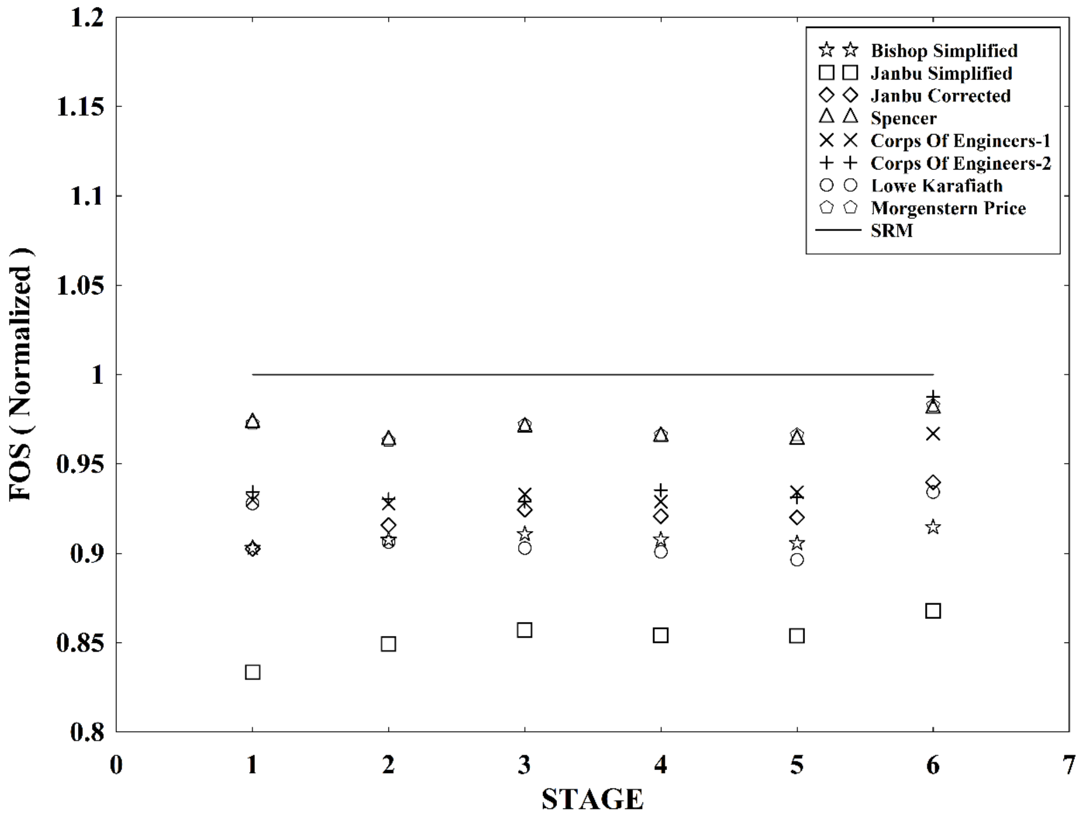
Graphs 8 Static with tailings considering non-circular failure surface



Graphs 9 Pseudo-static with tailings considering circular failure surface



Graphs 10 Pseudo-static with tailings considering non-circular failure surface



8.2.1 Observations

1) Janbu Simplified Method gives lowest Factor of Safety (FOS) among all other considered Limit Equilibrium Methods (LEM).
2) In almost all the cases Factor of Safety obtained through Limit Equilibrium Methods is less than those obtained considering Strength Reduction Method.
3) While considering circular failure surface, Corps of Engineer-2 method gives the maximum Factor of Safety in comparison to other LEM.
4) The Factor of Safety obtained from both Lowe Karafiath method and Strength Reduction Method in circular failure surface in static case are found to be in good agreement.
5) The Variations in Factor of Safety for all Limit Equilibrium Methods with respect to Factor of Safety considering Strength Reduction Method are ± 15.
6) Among Limit Equilibrium Method, Factor of Safety considering Non circular failure surface is less than Factor of Safety considering circular failure surface.
7) The Factor of Safety by Pseudo-static analysis is less than corresponding static Factor of Safety by Static Analysis.

Chapter 9

CONCLUSION

1. At every stage, in static conditions, the factor of safety is greater than 1. Therefore it is concluded that the dam is safe under Static Conditions.
2. In Pseudo-static condition , at every stage, in ‘without tailings’ conditions, the factor of safety is almost equal to 1, hence the dam is on the verse of failure. While at every stage, in ‘with tailings’ conditions, the factor of safety is much greater than 1, therefore the dam is safe.
3. **Comparative study without considering tailings fill**
   1) Janbu Simplified Method gives lowest Factor of Safety (FOS) among all other considered Limit Equilibrium Methods (LEM).
   2) Factor of Safety considering non-circular failure is less than corresponding to Factor of Safety obtained considering circular failure surface among Limit Equilibrium Methods.
   3) In stages 1 and 6, Factor of Safety of all Limit Equilibrium are less than those of Strength Reduction Method while in other stages (i.e. 2,3,4 and 5), Factor of Safety obtained considering Limit Equilibrium Methods are closely scattered around Strength Reduction Method.
   4) The Factor of Safety for the tailings dam considering tailings fill in the upstream is higher than the Factor of Safety obtained without tailings fill in the upstream.
   5) The Variations in Factor of Safety for all Limit Equilibrium Methods with respect to Factor of Safety considering Strength Reduction Method are ± 15.
   6) The Factor of Safety by Pseudo-static analysis is less than corresponding static Factor of safety by Static Analysis.
4. **Comparative study with considering tailngs fill**
   1) Janbu Simplified Method gives lowest Factor of Safety (FOS) among all other considered Limit Equilibrium Methods (LEM).
   2) In almost all the cases Factor of Safety obtained through Limit Equilibrium Methods is less than those obtained considering Strength Reduction Method.
   3) While considering circular failure surface, Corps of Engineer-2 method gives the maximum Factor of Safety in comparison to other LEM.

4) The Factor of Safety obtained from both Lowe Karafiath method and Strength Reduction Method in circular failure surface in static case are found to be in good agreement.
5) The Variations in Factor of Safety for all Limit Equilibrium Methods with respect to Factor of Safety considering Strength Reduction Method are ± 15.
6) Among Limit Equilibrium Method, Factor of Safety considering Non circular failure surface is less than Factor of Safety considering circular failure surface.
7) The Factor of Safety by Pseudo-static analysis is less than corresponding static Factor of Safety by Static Analysis.

REFERENCES

1. Ali Fawaz1, Elias Farah, Fadi Hagechehade(2014), "Slope stability analysis using numerical modelling", American Journal of Civil Engineering 2014; 2(3): 60-67.
2. Anton D. Tzenkov(2008), "Stability Analysis Of A Tailings Dam: Existing State And Planned Heightening", sixth international conference on case histories in Geotechnical Engiineering.
3. Bo He, Hongcai Zhang(2012), "Stability Analysis of Slope Based on Finite Element Method", MECS, 3,70-74.
4. Bishop, A.W. (1955), "The Use of the Slip Circle in the Stability Analysis of Slopes", Geotechnique, 5(1), pp.7-17.
5. Castro G, "Evaluation of seismic stability of tailings dams". *12th Panamerican Conference on Soil Mechanics and Geotechnical Engineering*. ; 2003.
6. Chakraborty D, Choudhury D (2009), " Investigation of the behavior of tailings earthen dam under seismic conditions", American J Eng Appl Sci 2(3):559–564.
7. Chakraborty D, Choudhury D (2011), "Seismic behavior of tailings dam using FLAC3D", ASCE Geotechnical Special Publication No. 211:3138–3147.
8. Chakraborty D, Choudhury D (2013), "Pseudo-Static and Pseudo-Dynamic Stability Analysis of Tailings Dam Under Seismic Conditions", Proc. Natl. Acad. Sci., India, Sect. A Phys. Sci.83(1):63–71.
9. Chambers DM, Higman B, "Long Term Risks Of Tailings Dam Failure". 2011;(907).
10. D.V.Griffoths and P.A.Lane.(1999), "Slope Stability Analysis by Finite Elements", Geotechnique 49, No. 3, , pp. 387-403.
11. Duncan JM. State of the art.(1996), "limit equilibrium and finite-element analysis of slopes", J Geotech Geoenviron Eng ASCE;122(7):577–96.
12. Hou-xiang L, Ning L, Xue L, Cong-shi W, Xu-dong P, "Effective stress analysis method of seismic response for high tailings dam". *Springer*. 2007;14(1):129-134. doi:10.1007/s11771.
13. Jitno H, Pacific BA, "Effects of earthquake design ground motions on the seismic response of upstream-raised tailings dams". *IPENZ Proceedings of Technical Groups 39 (LD)*. Perth, Australia; 2012:1-12.

14. Jonathan Z. Liang, David Elias(2010), "Seismic Evaluation of Tailing Storage Facility", Australian Earthquake Engineering Society 2010 Conference, Perth, Western Australia.
15. Klapperich H, Gmbh W, Richter T, Savidis S, "Seismic Stability of Tailing Dams for the Uranium Industry".
16. LIU Hou-xiang, LI Ning, LIAO Xue, WU Cong-shi, PAN Xu-dong(2007), "Effective stress analysis method of seismic response for high tailings dam", J. Cent. South Univ. Technol.01−0129−06.
17. Martin TE, McRoberts EC, "Some considerations in the stability analysis of upstream tailings dams". *Tailings Mine Waste '99*. 1999;(6):287-302.
18. Matsui T, San KC.(1992), "Finite element slope stability analysis by shear strength reduction technique", Soils Foundations;32(1):59–70.
19. Naylor, D.J. (1981), "Finite elements and slope stability. In: Numerical methods in Geomechanics", Proceedings of the NATO advanced study institute, Lisbon, Portugal, p. 229–44.
20. Pan J, Song Y, Zeng Q, Zhu H, "Influence Factors Analysis of Seismic Liquefaction Risk for Tailings Dam". *Electron J Geotech Eng*. 2015;20:4167-4180.
21. Qiangui Zhang, Guangzhi Yin, ZuoanWei, Xiangyu Fan, WensongWang, Wen Nie(2015), "An experimental study of the mechanical features of layered structures in dam tailings from macroscopic and microscopic points of view.", Elsevier.
22. Troncoso JH, "Permanent Seismic Settlements and Performance of Tailings Dams. In", *5th International Conferencec on Earthquake Geotechnical Engineering*. ; 2011:10-13.
23. Valenzuela L,"Considerations on the Rationality in the Selection and use of pseudo static coefficients in dam design", *5th International Conferencec on Earthquake Geotechnical Engineering*. Santiago; 2011:10-13.
24. W.B. Wei, Y.M. Cheng, L. Li.(January-March 2009), "Three-dimensional slope failure analysis by the strength reduction and limit equilibrium methods Original Research Article." ,Computers and Geotechnics, Vol. 1-2, no.6 , Pages 70-80.

25. Y.M. Cheng, T. Lansivaara, W.B. Wei.(May 2007), "*Two-dimensional slope stability analysis by limit equilibrium and strength reduction methods.*" Computers and Geotechnics, Vol.34, no.3, pp.137-150,.

26. Zheng H, Liu DF, Li CG. Slope stability analysis based on elasto-plastic finite element method. Int J Numer Meth Eng 2005;64:1871–88.