Detailed Slope Stability Analysis and Assessment of the Original Carsington Earth Embankment Dam Failure in the UK

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ABSTRACT

A 1225 m long, 35 m high zone earth filled embankment was being constructed from 1981 to 1984 from a British Regional Water Authority to regulate flows in the River Derwent in England. The Carsington Dam was planned to be one of the largest earth filled dams in Britain. Its reservoir capacity was 35 million m^3 and the watertight element was Rolled Clay Core with an upstream extension of boot shaped and shoulders of compacted mudstone with horizontal drainage layers of crushed limestone about 4 metres apart and a cut-off grout curtain (Davey and Eccles, 1983).

The downstream slope was 1:2.5 and the upstream slope 1:3. Fill placing began in May 1982 and took three summers, with winter shutdowns. In August 1983 a small berm was placed at the upstream toe to compensate for a faster rate of construction. Earth filling restarted in April 1984 and was one metre below the final crest level on 4 June 1984 when the upstream slope slipped (Skempton, 1985). Observations of pore pressure and settlement were made during construction at four sections and horizontal displacements were observed from August 1983. The Carsington Dam was almost completed on 1984.

However, at the beginning of June 1984, a 400-m length of the upstream shoulder of the embankment dam slipped some 11 m and failed. At the time of the failure, embankment construction was virtually complete with the dam approaching its maximum height of 35 m. Horizontal drainage blankets were incorporated in both the upstream and the downstream shale fill shoulders. Piezometers had been installed and pore pressures were being monitored in the foundation, in the clay core, and in the shoulder fill. The failure surface passed through the boot shaped rolled clay core and a relatively thin layer of surface clay in the foundation of the dam. Investigation of the events at Carsington has made important contributions to the fundamental understanding of the behaviour of large earthworks of this type (Vaughan et al., 1989; Dounias et al., 1996).

The objective of this research is to evaluate a detailed slope stability assessment of the Carsington Earth Embankment Dam in the UK used to retain mine tailings.

By using and applying advanced geotechnical engineering analysis tools and modelling techniques the Carsington Earth Embankment Dam, which is considered a particular geotechnical structure, is analysed.

In the current detailed slope stability analyses the total and effective stress state soil properties / parameters were used, and the most critical slip circle centre according to Fellenius - Jumikis method was initially determined. Subsequently, the Carsington Earth Embankment Dam and its foundation was analysed and examined against failure by slope instability. Considerations of loading conditions which may result to instability for all likely combinations of reservoir and tailwater levels, seepage conditions, both after and during construction were made, and hence three construction and / or loading conditions were examined in particular:

- The right after Construction Condition,
- The Steady Seepage Condition, and
- The Rapid Drawdown Condition in the reservoir

In this context, the slope stability at the three above mentioned discrete loading cases of the Carsington Earth Embankment Dam was analysed and presented, and certain valuable conclusions concerning the overall stability conditions of the Earth Embankment Dam during its original construction are deduced in this research paper.

In addition, for comparison reasons, a Slope Stability Analysis of the Carsington Earth Embankment Dam during loading case (a) using Taylor's curves was performed. Furthermore, the Shear Strength Reduction (S.S.R.) Analysis Method, based on the Finite Element Analysis technique (F.E.A.), was executed, for purposes of verification of the Global Slope Stability Analysis of the whole Carsington Earth Embankment Dam for the Loading case (a), i.e. right after construction condition of the Dam but prior to its filling with water, which proved that the results between the Shear Strength Reduction (S.S.R.) Analysis Method and the Limit Equilibrium Analysis Method (LEM) based on the method of slices are comparable and similar.

Finally, the reasons why the Fellenius - Jumikis method is inaccurate were examined, analysed and explained, as well as the technical lessons learned from this large scale Earth Embankment Dam body failure were pointed out.

KEYWORDS: Slope Stability Analysis; Earth Embankment Dams; Slope Failure; Embankment Loading Conditions; Soil Properties / Parameters; Critical Slip Circle Centre Determination; Steady Seepage Condition; Rapid Drawdown Condition; Shear Strength Reduction Analysis Method; Fellenius - Jumikis Method; Computer Aided Slope Stability Analysis & Design.

INTRODUCTION

The British Regional Water Authority in Derbyshire as part of its water storage system in order to regulate flows in the River Derwent, decided to construct a 35 m high and 1225 m long zone earth filled embankment from 1981 to 1984. The Carsington Dam (Fig. 1) would be one of the largest earth-fill dams in the UK. A 10 km long diversion tunnel would divert the water during the winter and been stored in the reservoir of an estimated capacity of 35 million m^3. When the water level in the river was low, water would be released from the reservoir. The reservoir would also facilitate as a rainwater runoff regulatory hydraulic system (Fig. 2).

The characteristics of the original construction of the Carsington Earth Embankment Dam, are briefly described as follows. A shallow trench that was excavated upstream of the centreline into the weathered gray foundation mudstone was connected to the central clayey core of the embankment. Below the base of the trench a cut-off grout curtain extended. In the downstream and upstream shells the earth fill was classified as Type A and Type B (Skempton and Vaughan, 1993). The Type A fill was described as a yellow brownish spotted clay with fine gravel < 5 mm and pebbles of mudstone and was placed immediately upstream and downstream of the clay core. The Type B soil was intended to be the same general type of material but without pebbles, and was located in the outer portions of the shells. The representative cross sections of the original and reconstructed Carsington Earth Embankment Dam are shown in Fig. 3 (Banyard et al., 1992).

Figure 1: The study area of the Carsington Earth Embankment Dam in Derbyshire region, UK

Figure 2: Close up of the reconstructed Carsington Earth Embankment Dam area.

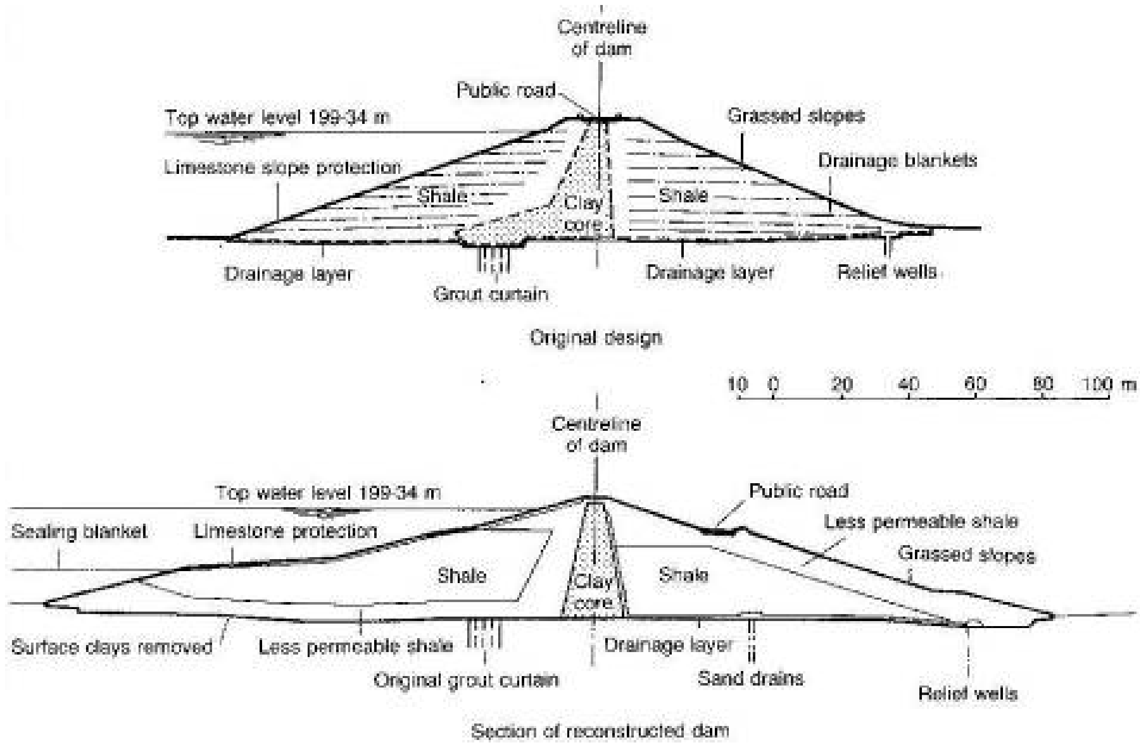


Figure 3: Representative cross sections of the original and reconstructed Carsington Earth Embankment Dam (Banyard et al., 1992).

In 3 successive summer seasons during 1982, 1983, and 1984, the construction of the embankment earth material moving would take place. The fill had reached almost 200 m in the north valley and 197 m in the main section of the dam, at the end of the 1983 season. Earth moving was resumed in April of 1984 and the embankment was completed to full height at the north end and about 1 m short at the main section. Queries about the stability had been raised by the Contractor and therefore, the Kennard Report, comprising a detailed review setting out clearly the parameters used, the slip surfaces considered, the methods of analysis, and the resulting Factors of Safety, was written. Concern was summarized in a statement that "a revised design was essential if the dam was to be completed with confidence and safety". It was fully expected that a revised design would be prepared in the 1983/84 winter period. However, no meeting was held to discuss the request, no change to the design was made.

Surprisingly, though, just before construction completion was reached, tension cracks were first observed on the crest of the embankment on June 4 of 1984, when 1 m of crest remained to be placed, over an approximate length of 65 m. A 400 m section of the upstream slope failed 36 hours later, with a maximum horizontal displacement of almost 15 m.

According to Skempton and Coats, 1985 and Skempton and Vaughan, 1993, the exact incident description is as follows. The failure started in the early hours of 4 June 1984 with a 50 mm crack on the crest over a length of about 120 m. During the night of 5 June a major upstream slip occurred. The slip propagated along the embankment in both directions extending to a length of about 500 m, with an embankment crest slumping of 11 m, as shown in Fig. 4, and the upstream toe moving 13 m horizontally by 6 June 1984. The initial slip sheared through the core which already contained shear surfaces due to rutting and along a layer of yellow clay in the foundation which contained solifluction shears. Both materials were brittle with low residual strengths (Skempton and Coats, 1985; Skempton and Vaughan, 1993).

Figure (4). Close up of the upstream slip failure surface of the Carsington earth embankment dam, showing the slip propagated along the embankment with an embankment crest slumping of 11 m (Environment Agency, 2011).

According to Johnston et al., 1999, exact wording: "…..Investigations indicated that the initial slip resulted from shearing through the core and along a layer of yellow clay beneath the foundation. In addition, there were a number of shear surfaces in the core caused by rutting from construction plant. The yellow clay layer in the foundation was found to contain solifluction shears, and the factor of safety reduced to 1.0 when strength reductions due to pre-existing shear planes, progressive failure and lateral load transfer, were taken into account….." (Johnston, 1995; Johnston et al., 1999).

Remedial works and reconstruction of the failed dam is described by Banyard et al. (1992), Chalmers et al. (1993), Macdonald et al. (1993) and Vaughan et al. (1991). It commenced in February 1989 and was completed in 1991, seven years after the start of investigations. The main differences in cross-section are shown in Fig. 3. Reconstruction of the new dam involved excavation of two million cubic metres of the original dam to remove all failed material and lay a sound foundation. The downstream view of the reconstructed Earth Embankment of the Carsington Dam as stands nowadays is shown in Fig. 5.

Figure (5). Downstream view of the reconstructed Earth Embankment of the Carsington Dam.

The failure of the Carsington Dam led to additional attention being given to the role of the construction equipment and procedures in the subsequent stability of a structure. In this case, the compaction equipment selected and the rate of fill placement are considered to have been key factors in the observed failure. In addition, the importance of selecting instrumentation, which can provide a precursor to a failure, was reinforced (Rowe, 1991).

Finally, in this research study, a detailed slope stability analysis was performed and presented for three discrete loading cases of the Carsington Earth Embankment Dam construction and post-construction stages, i.e. a) the right after Construction Condition [Loading case (a)], b) the Steady Seepage Condition [Loading case (b)], and the Rapid Drawdown Condition [Loading case (c)]. From these detailed slope stability analysis results, the reasons as to why the Carsington Earth Embankment Dam failed during its original construction became obvious.

EXAMINED ANALYSIS AND LOADING CONDITIONS

The Carsington Earth Embankment Dam and its foundation was analysed and designed against failure by slope instability. Consideration of loading conditions which may result to instability must be made for all likely combinations of reservoir and tailwater levels, seepage conditions, both after and during construction. Three construction and / or loading conditions were examined in particular:

a. <u>The right after Construction Condition</u>

The critical condition to be analysed is at the completion of embankment dam construction but prior to filling with water.

In this case there is no water table present in the reservoir and in the embankment dam. Total or undrained shear strength parameters of soils are used in this loading case.

b. The Steady Seepage Condition

For zoned and homogeneous types of embankment dams and when the reservoir is full of water and some steady seepage into the embankment is established, the conditions to be considered for the steady state seepage analysis should be:

- Steady state seepage pore pressures which are fully developed as a result of the reservoir have being storing water over a long period of time. In this case there is a Phreatic Surface Line under steady seepage state.
- The combination of upstream and downstream water levels that is found to be the most critical. In this research, this case is not examined.

Effective or drained shear strength parameters of soils are used in this loading case.

c. The Rapid Drawdown Condition in the reservoir

Fluctuations in reservoir water level may cause the upstream face stability to become critical mainly due to the removal of the supporting water. When the reservoir is rapidly evacuated and drawn down, pore water pressures in the dam body are reduced in two ways. There is a slower dissipation of pore pressure due to drainage and there is an immediate elastic effect due to the removal of the total or partial water load. The exact mechanism of this phenomenon is as follows: It is assumed that the reservoir has been maintained at a high level for a sufficiently long time so that the fill material of the dam is fully saturated and steady seepage established. If the reservoir is drawn down at this stage, the direction of flow is reversed, causing instability in the upstream slope of the earth dam. The "instantaneous" drawdown is a hypothetical condition that is assumed and pore pressures along the sliding surface are determined by drawing the "instantaneous" flow net. The most critical condition of sudden drawdown means that while the water pressure acting on the upstream slope at "full reservoir" condition is removed, there is no appreciable change in the water content of the saturated soil within the embankment. The saturated weight of the slope produces the shearing stresses while the shearing resistance is decreased considerably because of the development of the pore water pressures which do not dissipate rapidly (Ranjan, Rao, 2005). Therefore, it was considered very important such an analysis to be carried out and included in this research.

Hence, in this Rapid Drawdown Condition, there is no water table present in the reservoir but in the embankment dam body there are still full pore water pressures. Effective or drained shear strength parameters of soils are used in this loading case, too.

In the following Figures 1, 2 and 3, the Cross Section Views of the Carsington Earth Embankment Dam model drawn in CAD, showing the internal water piezometric surface trajectory, as well as the 3-D Model Views (Block Views) based on provided dimensions, are presented on exact scale.

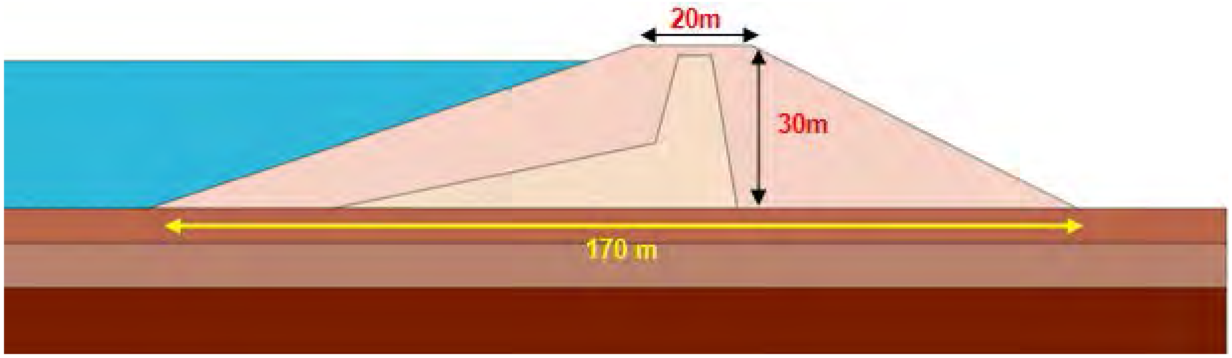


Figure 6: Cross Section View of the Carsington Earth Embankment Dam model, based on provided dimensions (On exact Scale).

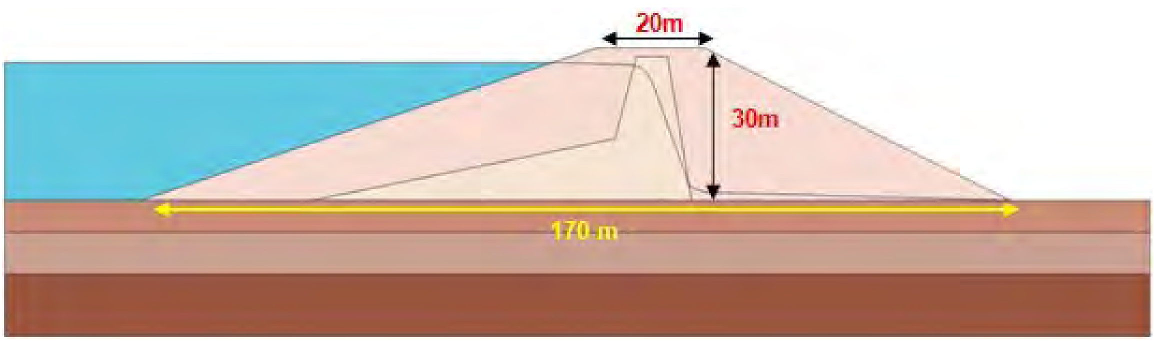


Figure 7: Cross Section View of the Carsington Earth Embankment Dam model, showing the internal water piezometric surface trajectory, based on provided dimensions (On exact Scale).

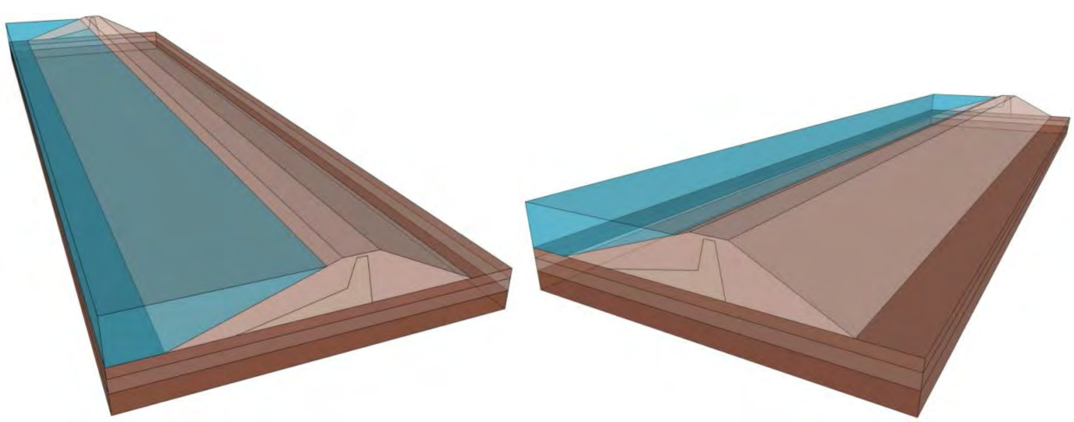


Figure 8: 3-D Model Views (Block Views) of the Carsington Earth Embankment Dam, based on provided dimensions (On exact Scale).

GEOMETRY OF THE CARSINGTON EARTH EMBANKMENT DAM PROFILE FOR SLOPE STABILITY ANALYSIS

The general geometry and the foundation layers of the Carsington Earth Embankment Dam profile is shown in Figure 4. It should be noted that this profile is not to scale.

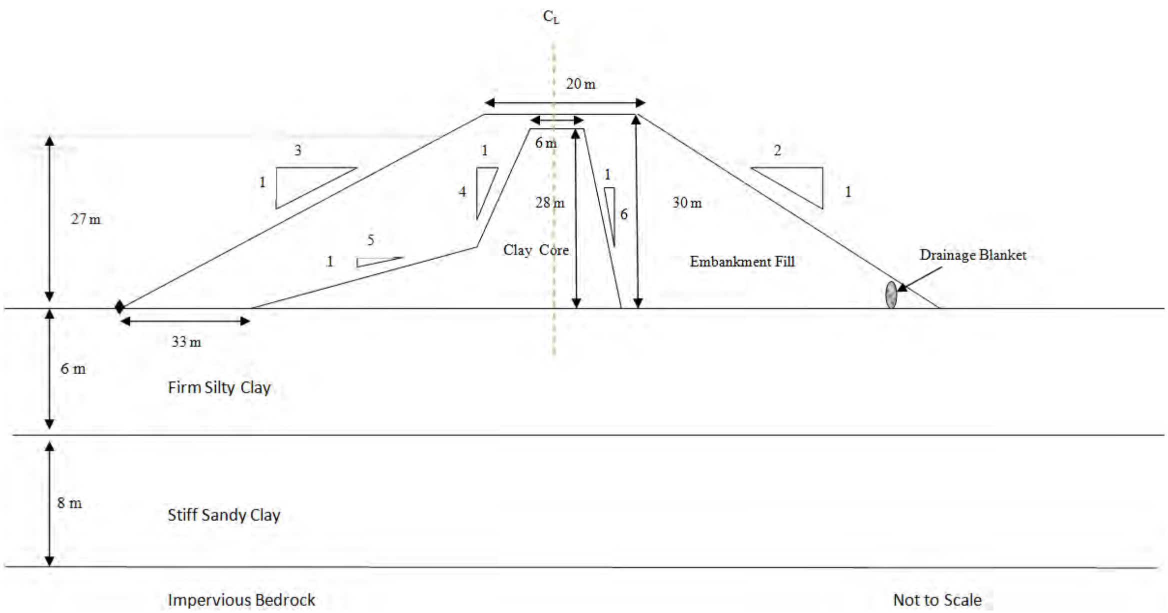


Figure 9: Geometry (not to scale) of the cross section of the Carsington Earth Embankment Dam. (Embankment Dam Profile for Slope Stability Analysis).

DETERMINATION OF THE X, Y COORDINATES OF RELEVANT POINTS OF THE CARSINGTON EARTH EMBANKMENT DAM CROSS SECTION

Based on the cross section geometry of the Carsington Earth Embankment Dam, the determination and calculation of the accurate x, y coordinates of all relevant points of the “on the scale cross-section diagram”, must be carried out. For this purpose, the following procedure was executed. Let’s assume a1, a2, b1 and b2 lengths and point G are located as shown in Figure 5.

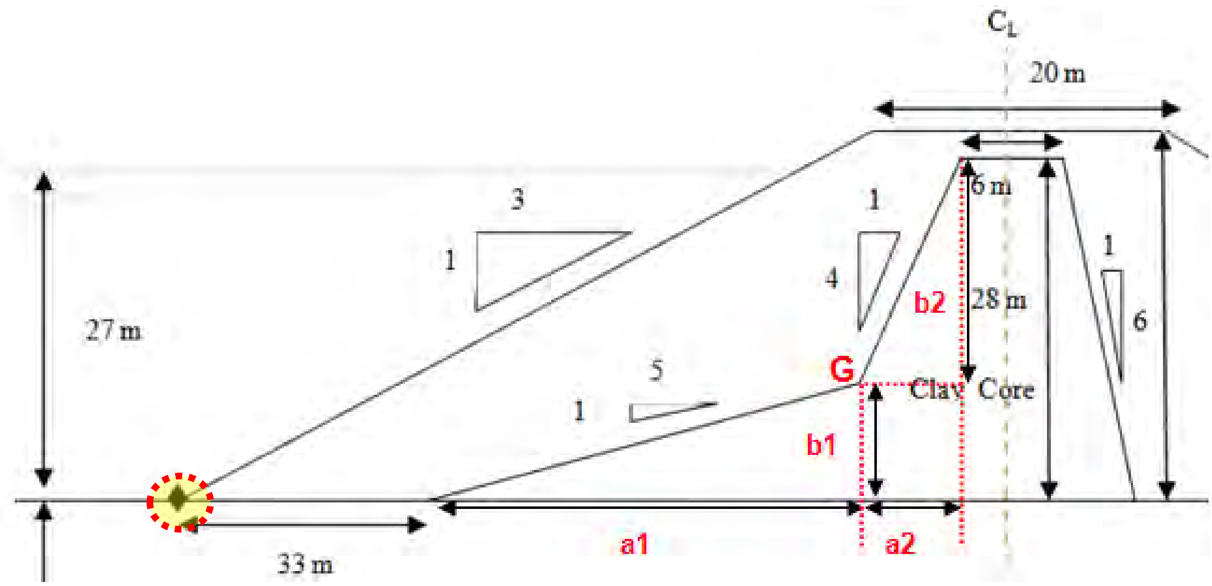


Figure 10: a1, a2, b1 and b2 lengths calculation for the determination of the x, y coordinates of point G.

If trigonometric rules are applied to the "on the scale cross-section diagram", then the following equations can be formulated:

a1 + a2 = [30 / tan (tan-1(1/3))] - 33 + 10 - 3 = 64

and

a1 x tan (tan-1(1/5)) + a2 / tan (tan-1(1/4)) = 28

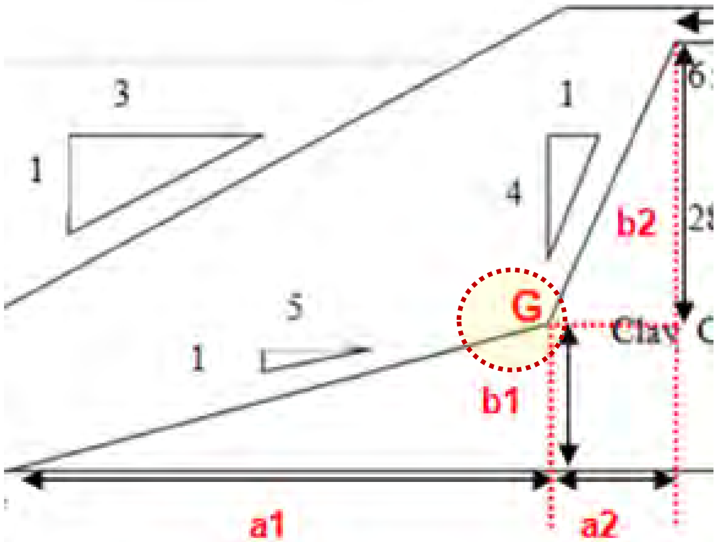
Therefore, solving these two equations of two unknowns, we can easily calculate a1 and a2 lengths, as follows:

a1 = 60 m, therefore b1 = 12 m

a2 = 4 m, therefore b2 = 16 m

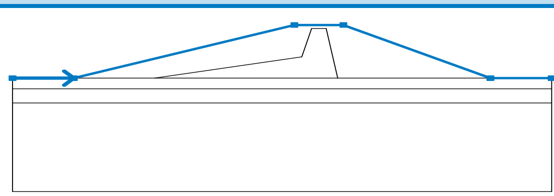
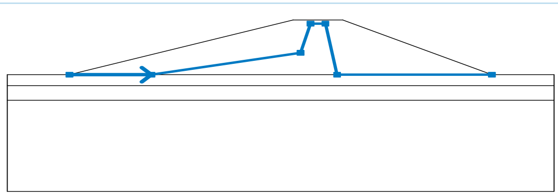
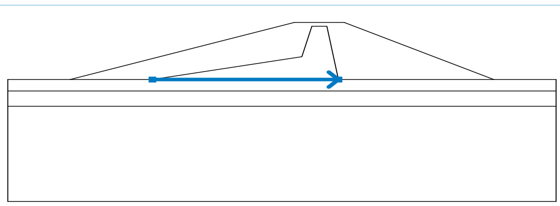
Therefore, point's G x, y coordinates are:

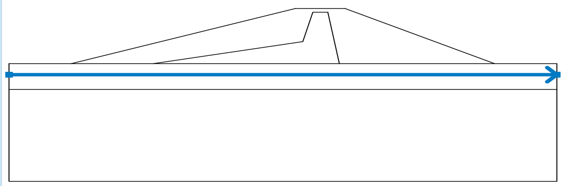
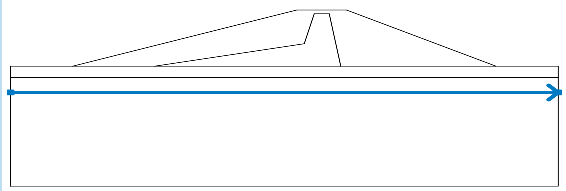
x = 93
y = 12

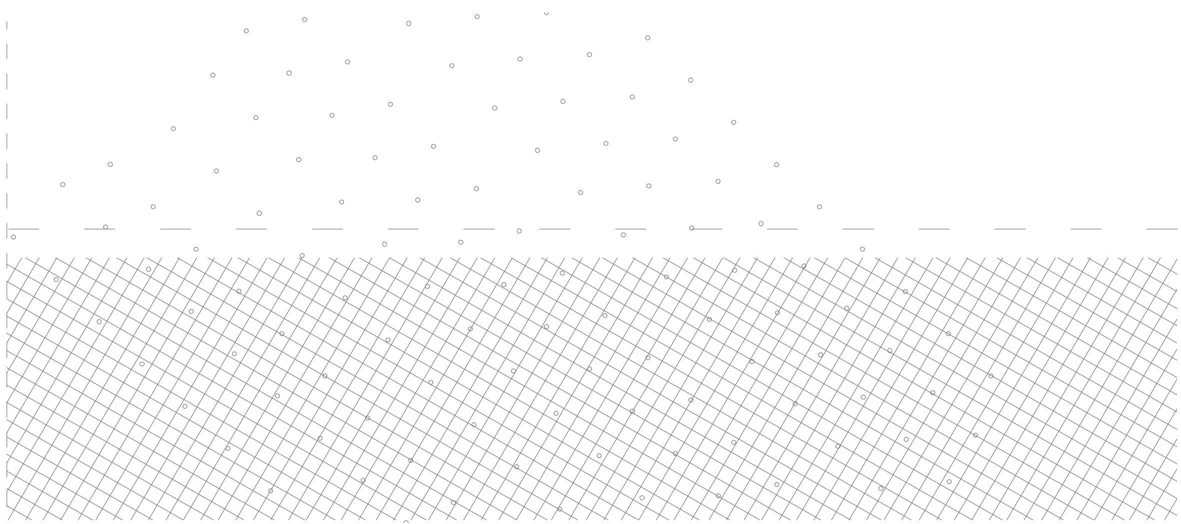


According to the above mentioned coordinates of G point, as well as the geometry of the Carsington Earth Embankment Dam as shown on Figure 4, the coordinates x, y of all other relevant points and the structure interfaces were accurately determined, as shown in Table 1 below, considering x, y coordinates of the point marked with a back diamond in figure 5 as x = 5, y =7.

Table 1: Depicted coordinates of all relevant points and structure interfaces of the Carsington Earth Embankment Dam, considering x, y coordinates of the point marked with a back diamond as x = 5, y =7.

#	Interface location	Coordinates of interface points [m]					
		x	y	x	y	x	y
1		-20.00	7.00	5.00	7.00	95.00	37.00
		115.00	37.00	175.00	7.00	200.00	7.00
2		5.00	7.00	38.00	7.00	98.00	19.00
		102.00	35.00	108.00	35.00	112.70	7.00
		175.00	7.00				
3		38.00	7.00	112.70	7.00		

#	Interface location	Coordinates of interface points [m]					
		x	y	x	y	x	y
4		-20.00	1.00	200.00	1.00		
5		-20.00	-7.00	200.00	-7.00		

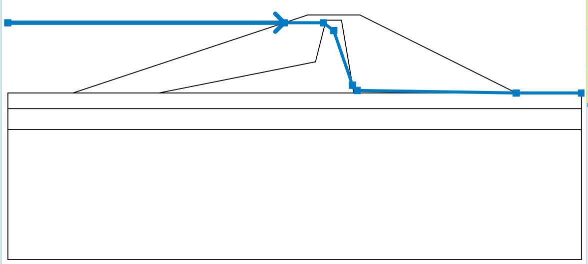


Additionally, the coordinates of the Water Table Profile in the reservoir and inside the body of the Carsington Earth Embankment Dam are also calculated and shown in tables 2 & 3. The water level coordinates shown in table 2 are based on the Black Diamond, as shown in figure 5, having coordinates of (x = 5, y =7).

Table 2: Coordinates of water level points based on the Black Diamond, as shown in figure 5, having coordinates of (x = 5, y =7).

Water Profile	
X coordinate	**Y coordinate**
5	34
86	34
105	31
112.2	10
175	7

Table 3: Depicted coordinates of water level points based on the Black Diamond, as shown in figure 5, having coordinates of (x = 5, y =7).

#	GWT location	Coordinates of GWT points [m]					
		x	y	x	y	x	y
1		-20.00	34.00	86.00	34.00	101.00	34.00
		105.00	31.00	112.20	10.00	114.00	8.00
		175.00	7.00	200.00	7.00		

All other points shown in Table 1 are then determined around the black diamond point, as well as a list of the x, y coordinates was prepared for:

- ✓ The Water Table Profile
- ✓ The Foundation Ground Soil Layers
- ✓ The Embankment Dam Soil Layers, and
- ✓ The possible slip circle centres (given in a next chapter)

Finally, Figure 11 shows the illustration of x, y coordinates of all relevant points and structure interfaces of the Carsington Earth Embankment Dam, including the water level, considering x, y coordinates of the point marked with a back diamond as 5, 7.

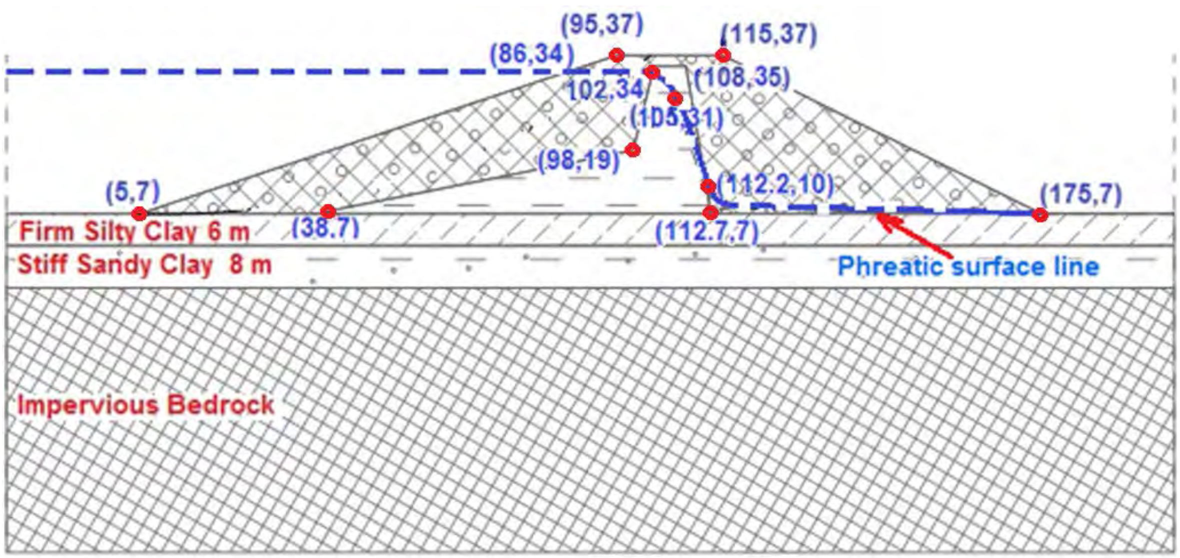


Figure 11: Illustration of x, y coordinates of all relevant points and structure interfaces of the Carsington Earth Embankment Dam, including the water level, considering x, y coordinates of the point marked with a back diamond, as shown in figure 5, as x = 5, y =7.

SOIL PROPERTIES / PARAMETERS IN TOTAL AND EFFECTIVE STRESS STATE

The relevant properties / parameters of both natural (geological) and man-made soils in both total (undrained) and effective (drained) stress state for the Slope Stability Analysis and Design are presented in Table 5. Only the soil mechanics parameters and properties that are of concern and interest for the slope stability analysis of the body of the Carsington Earth Embankment Dam are given.

Table 5: Soil properties / parameters in total and effective stress state and legend.

#	Name	Pattern	φ_{ef} [°]	c_{ef} [kPa]	γ [kN/m^3]
1	Embankment Fill		24.00	20.00	18.00
2	Clay Core		4.00	15.00	19.00
3	Firm Silty Clay		10.00	35.00	17.00
4	Stiff Sandy Clay		30.00	60.00	20.00
5	Impervious Bedrock		35.00	200.00	24.00

Soil parameters - uplift

#	Name	Pattern	γ_{sat} [kN/m^3]	φ_u [°]	c_u [kPa]
1	Embankment Fill		19.00	25	40
2	Clay Core		20.00	0	125
3	Firm Silty Clay		18.00	0	50
4	Stiff Sandy Clay		21.00	15	80
5	Impervious Bedrock		24.00	35	200

Soil properties / parameters in total and effective stress state

Embankment Fill

Unit weight :	γ	=	18.00 kN/m^3
Effective Angle of internal friction :	φ_{ef}	=	24.00 °
Effective Cohesion of soil :	c_{ef}	=	20.00 kPa
Total Angle of internal friction :	φ_u	=	25.00 °
Total Cohesion of soil :	c_u	=	40.00 kPa
Saturated unit weight :	γ_{sat}	=	19.00 kN/m^3

Clay Core

Unit weight :	γ	=	19.00 kN/m^3
Effective Angle of internal friction :	φ_{ef}	=	4.00 °
Effective Cohesion of soil :	c_{ef}	=	15.00 kPa
Total Angle of internal friction :	φ_u	=	0.00 °
Total Cohesion of soil :	c_u	=	125.00 kPa
Saturated unit weight :	γ_{sat}	=	20.00 kN/m^3

Firm Silty Clay

Unit weight :	γ	=	17.00 kN/m^3
Effective Angle of internal friction :	φ_{ef}	=	10.00 °
Effective Cohesion of soil :	c_{ef}	=	35.00 kPa
Total Angle of internal friction :	φ_u	=	0.00 °
Total Cohesion of soil :	c_u	=	50.00 kPa
Saturated unit weight :	γ_{sat}	=	18.00 kN/m^3

Stiff Sandy Clay

Unit weight :	γ	=	20.00 kN/m^3
Effective Angle of internal friction :	φ_{ef}	=	30.00 °
Effective Cohesion of soil :	c_{ef}	=	60.00 kPa
Total Angle of internal friction :	φ_u	=	15.00 °
Total Cohesion of soil :	c_u	=	80.00 kPa
Saturated unit weight :	γ_{sat}	=	21.00 kN/m^3

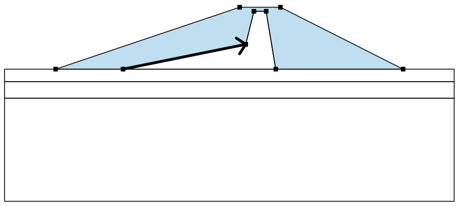
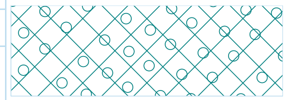
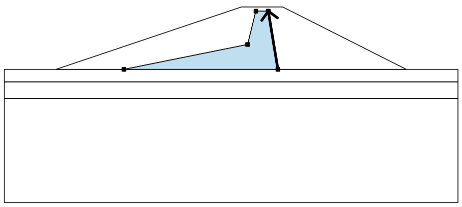
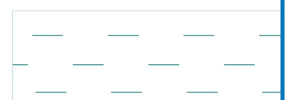
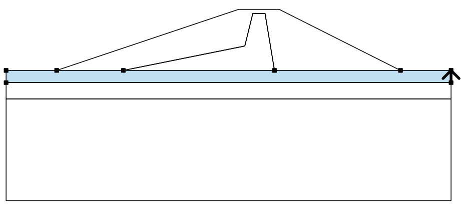
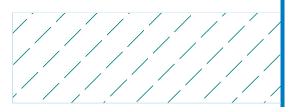
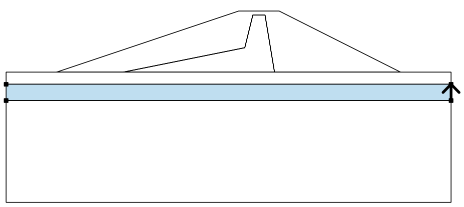
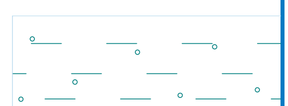
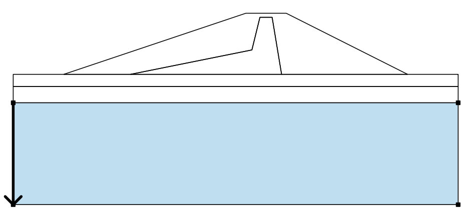
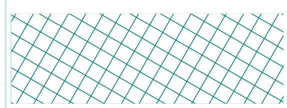
Impervious Bedrock (Conservative Values)

Unit weight :	γ	=	24.00 kN/m^3
Effective Angle of internal friction :	φ_{ef}	=	35.00 °
Effective Cohesion of soil :	c_{ef}	=	200.00 kPa
Total Angle of internal friction :	φ_u	=	35.00 °
Total Cohesion of soil :	c_u	=	200.00 kPa
Saturated unit weight :	γ_{sat}	=	24.00 kN/m^3

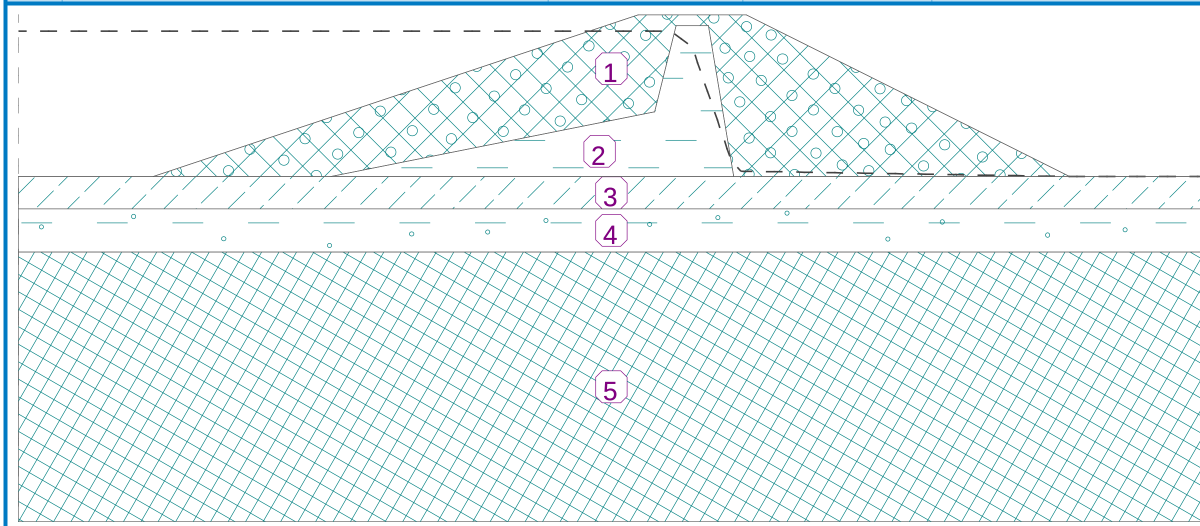
The above mentioned soil mechanics parameters and properties either in total or in effective stress state are assigned to each earth embankment dam surface (region) as follows.

Soil Assigning in the Carsington Earth Embankment Dam surfaces (regions)

#	Surface position	Coordinates of surface points [m]				Assigned soil
		x	y	x	y	

#	Surface position	Coordinates of surface points [m] x	y	x	y	Assigned soil
1		38.00	7.00	98.00	19.00	Embankment Fill
		102.00	35.00	108.00	35.00	
		112.70	7.00	175.00	7.00	
		115.00	37.00	95.00	37.00	
		5.00	7.00			
2		112.70	7.00	108.00	35.00	Clay Core
		102.00	35.00	98.00	19.00	
		38.00	7.00			
3		200.00	1.00	200.00	7.00	Firm Silty Clay
		175.00	7.00	112.70	7.00	
		38.00	7.00	5.00	7.00	
		-20.00	7.00	-20.00	1.00	
4		200.00	-7.00	200.00	1.00	Stiff Sandy Clay
		-20.00	1.00	-20.00	-7.00	
5		-20.00	-7.00	-20.00	-57.00	Impervious Bedrock
		200.00	-57.00	200.00	-7.00	

#	Surface position	Coordinates of surface points [m]				Assigned soil
		x	y	x	y	



MOST CRITICAL SLIP CIRCLE CENTRE DETERMINATION BY FELLENIUS - JUMIKIS METHOD

General

Before running any Slope Stability Analysis Computer programme (software), a scale diagram of the Carsington Earth Embankment Dam was drawn and the Fellenius - Jumikis method (Watson, 2012 & Murthy, 2003) was used in order to obtain a very approximate indication of the location of the most critical slip circle centre in the Carsington Earth Embankment Dam.

Procedure for locating the Most Critical Circle

Since the determination of the minimum factor of safety for a slope is very crucial for the design of the Carsington Earth Embankment Dam, it is important to locate the most critical slip circle with as few trials as possible. In a random trial and error approach, the three geometric parameters, namely, the centre of rotation, the radius of slipcircle and the distance of intercept in front of the toe are varied and the minimum factor of safety obtained. This requires a very large number of trials, but computers have made the method feasible. However, it is known that there is a certain pattern in slip circle behaviour and a knowledge of this pattern can be used to advantage and the number of trials reduced.

For instance, it is known that the most critical circle passes through the toe of the slope when (a) the angle of shearing resistance φ is greater than 3°, and (b) the slope angle β exceeds 53°, irrespective of the value of φ. The most critical circle intersects the slope in front of the toe if φ is less than 3° and $\beta < 53°$.

Fellenius (1936) proposed an empirical procedure to find the centre of the most critical circle in a $\varphi_u = 0$ soil. The centre O for the toe failure case can be located at the intersection of the two lines drawn from the ends A and B of the slope at angles α and ψ (Figure 7 (a)). The angles α and ψ vary with the slope β. Table 6 gives these values.

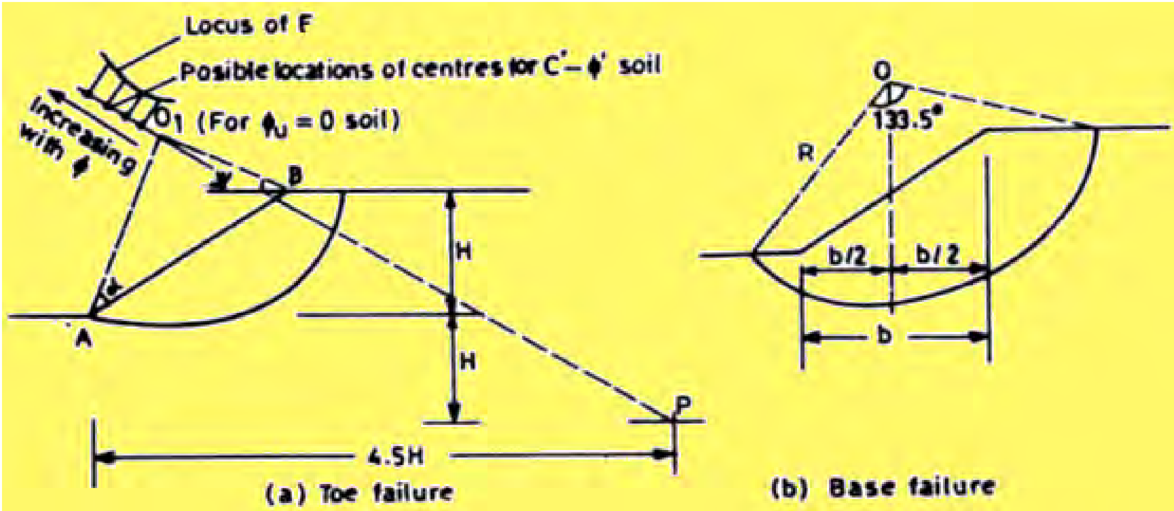


Figure 7: Location of critical circle (Ranjan, Rao, 2005).

Jumikis (1962) further extended this method to the case of a homogeneous, c' - φ' soil. After obtaining the centre O1 for a $\varphi_u = 0$ soil, point P is located at a distance 4.5 H horizontally from the toe of the slope and H below the toe of the slope. The centre of the critical circle is then assumed to lie on the extension of line PO1 and the factors of safety obtained are plotted to obtain a locus from which the minimum factor of safety can be read [Figure 7 (a)].

For flat slopes or when the soil below the toe is softer than the slope material, the critical slip circle will not be a toe circle but will reach much below the toe, resulting in what is called a “base failure”. Fellenius showed that in a homogeneous, purely cohesive soil with slope angles less than 53°, the centre of rotation for the deep seated base failure lies on a vertical drawn through the mid-point of the slope and the central angle of the critical circle is about 133.5° [Figure 7 (b)]. Where a stiffer layer of soil or rock lies beneath the slope, the most critical circle tends to be tangential to this stratum (experience of Dr. C. Sachpazis).

Table 6: Fellenius's Criteria for Locating the of Most Critical Centre Circle of a Slope in a $\varphi_u = 0$ Soil.

Slope Angle β	Slope ratio	Angle α^o	Angle ψ^o
60°	1 / 0.58	29°	40°
45°	1 / 1	28°	37°
33.8°	1 / 1.5	26°	35°
26.6°	1 / 2	25°	35°
18.4°	1 / 3	25°	35°
11.3°	1 / 5	25°	37°

The above guidelines can only serve as pointers and in any stability analysis a sufficiently large number of trials have to be made to locate the correct critical circle. It is found that the factor of safety is more sensitive to lateral shifting of the centre than to vertical movements. Moreover, the guidelines according to Fellenius - Jumikis method, can only serve as a general and approximate way for the determination of the critical slip circle centre.

The reasons of the Fellenius - Jumikis method inaccuracy mainly lie because of:

a) The soils of the Carsington Earth Embankment Dam may not be purely $\varphi_u = 0$ soils, as Fellenius (1936) proposed,
b) The soils of the Carsington Earth Embankment Dam may not be homogeneous c' - φ' soils, (because there might be different soil layers in the sloping embankment body and different soils in the impermeable core) as Jumikis (1962) assumed to further extended Fellenius's method,
c) The existence of water tables and pore water pressures are not taken into account in Fellenius - Jumikis method,
d) Fellenius - Jumikis method becomes less reliable for non-homogeneous conditions, such as irregular slope profile,
e) Surcharging and other type of loading of slopes is not taken into account in Fellenius - Jumikis method, and
f) Seismic loading of slopes is not taken into account in Fellenius - Jumikis method. Seismic loading deteriorates stability conditions and a special treatment is required, (Anastasiadis et al, 2006).

Therefore, according to the above mentioned reasons of inaccuracy the determination of the location of the most critical slip circle centre in an earth embankment dam by using Fellenius - Jumikis method could not be accurate enough.

But nevertheless, the determination of the coordinates (x, y) of the location of the most critical slip circle centre by using even the inaccurate Fellenius - Jumikis method can dramatically help to advantage and reduce the number of computational trials needed.

Finally according to Watson (2012), "…..the methods of Fellenius and Jumikis are often ignored, especially when doing computer analysis, but the ideas are sound and give a good first guess for solving a slope instability problem - they can save a lot of time !".

Location of Most Critical Circle in the Carsington Earth Embankment Dam

By applying the above mention procedure, according to Fellenius - Jumikis method, the following on scale diagram of the Carsington Earth Embankment Dam cross section was drawn (Figure 8) and a very approximate indication of the location of the most critical slip circle centre in the Carsington Earth Embankment Dam cross section was obtained, as shown in Figure 11. For this determination, the following input data were taken into account:

Embankment Fill:

✓ Unit weight	γ	=	18.00	kN/m^3
✓ Effective Angle of internal friction:	φ_{ef}	=	24.00°	
✓ Effective Cohesion of soil	c_{ef}	=	20.00	kPa
✓ Total Angle of internal friction:	φ_u	=	25.00°	
✓ Total Cohesion of soil	c_u	=	40.00	kPa
✓ Slope Angle β		=	18.44°	
✓ Angle α^o		=	25°	
✓ Angle ψ^o		=	35°	

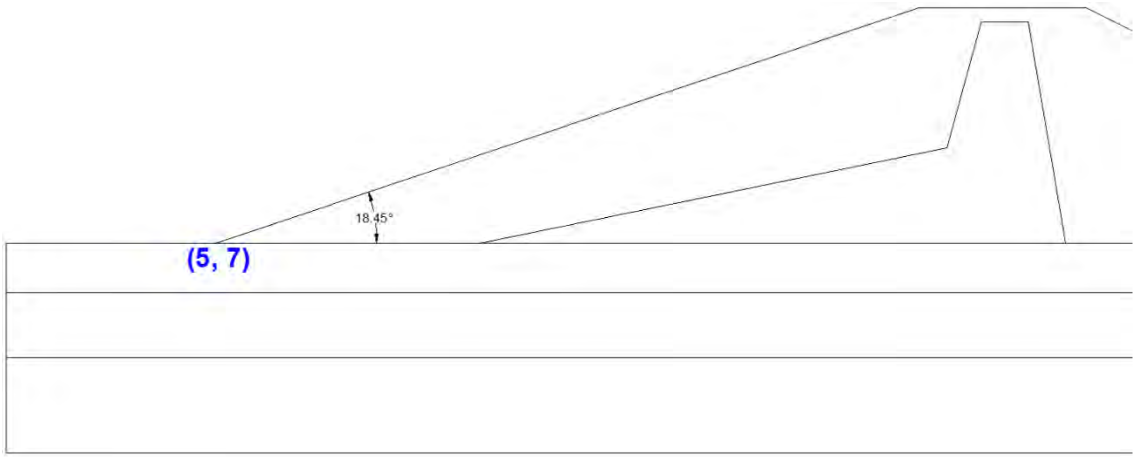


Figure 11: Part of the on scale cross section of the Carsington Earth Embankment Dam for locating of the most critical slip circle centre according to Fellenius method.

Therefore, the coordinates (x, y) of the location of the most critical slip circle centre point (O1) in the Carsington Earth Embankment Dam, as shown in Figure 12, are:

x = 61
y = 60

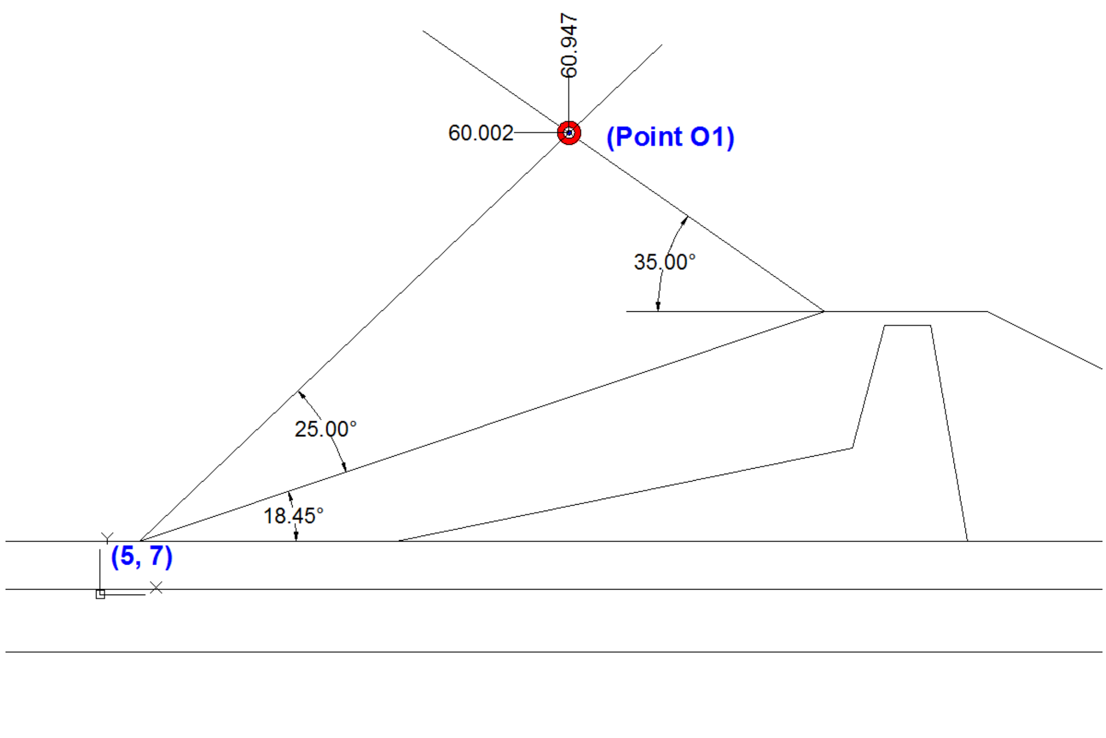


Figure 12: Coordinates (x, y) of the location of the most critical slip circle centre on the cross section of the Carsington Earth Embankment Dam, according to Fellenius method.

However, it should be remembered that the determination of the location of the most critical slip circle centre in the Carsington Earth Embankment Dam could not be accurate enough, and that it can only be used in order to dramatically help to reduce the number of the succeeding computational trials needed.

Therefore, for running any Slope Stability Analysis Computer programme, as it is presented in following chapters, the most probable “window”, where the potential centre of the most critical slip circle might lie, will be arranged according to the above mentioned coordinates (x, y) values of the location of the most critical slip circle centre point (O1) in the Carsington Earth Embankment Dam as determined according to Fellenius - Jumikis method.

These values really did pay off in saving time, and ensured that the correct circle with the lowest value of Factor of Safety (FoS) was found out.

SLOPE STABILITY ANALYSIS OF THE CARSINGTON EARTH EMBANKMENT DAM USING BISHOP METHOD

General

The Carsington Earth Embankment Dam and its foundation was analysed and designed against failure by slope instability.

Considerations of loading conditions which may result to instability for all likely combinations of reservoir and tailwater levels, seepage conditions, both after and during construction were made, and hence three construction and / or loading conditions were examined in particular, as follows:

a. <u>The right after Construction Condition</u>.

The first critical condition to be analysed is at the completion of embankment dam construction but prior to filling with water. [Loading case (a)]. In this case, there is no water table present in the reservoir and in the embankment dam body. Total or undrained shear strength parameters of soils are used in this loading case.

b. <u>The Steady Seepage Condition</u>.

The second critical condition to be analysed is when the reservoir is full of water and some steady state seepage into the Carsington Earth Embankment Dam is established. [Loading case (b)]. In this case, a Phreatic Surface under steady seepage state is present in the embankment dam body. The Water Table Profile in the reservoir and inside the body of the Carsington Earth Embankment Dam is shown in a previous section. Effective or drained shear strength parameters of soils are used in this loading case.

c. <u>The Rapid Drawdown Condition in the reservoir</u>.

Rapid Drawdown in the reservoir water level may cause the upstream face stability to become very critical mainly due to the removal of the supporting water and also due to the development of the adverse seepage forces inside the embankment dam body during pore water pressure dissipation process. In this case, there is no water table present in the reservoir but in the embankment dam body there are still full pore water pressures. Effective or drained shear strength parameters of soils are used in this loading case, too.

Slope Stability Analysis Results for the right after Construction Condition [Loading case (a)]

The analysis was performed using:

- ✓ A model with the simulation of the Carsington Earth Embankment Dam body and its foundation soil layers with all applied loads. The model was developed using exact dimensions in an appropriate scale, as shown in Figure 10.
- ✓ No water table in the reservoir and in the embankment dam body
- ✓ Total or undrained shear strength parameters of soils
- ✓ No seismic actions were taken into account
- ✓ Bishop Method for Slope Stability Analysis

For performing the slope stability analysis for the right after construction condition [Loading case (a)], the specialized Civil & Geotechnical Engineering computer Program Larix / Cubus was used, operating according to E.U. Regulations, European Standard ENV 1997-1. Eurocode 7 (EC-7) - ENV 1997-1.

The overall minimum stability factor of safety for the loading case (a), i.e. exactly after construction of the Carsington Earth Embankment Dam but prior to filling with water, was calculated equal to **0.96**, which means imminent failure conditions. The computed analysis results are illustrated in the following Figures (Fig. 13a and Fig. 13b).

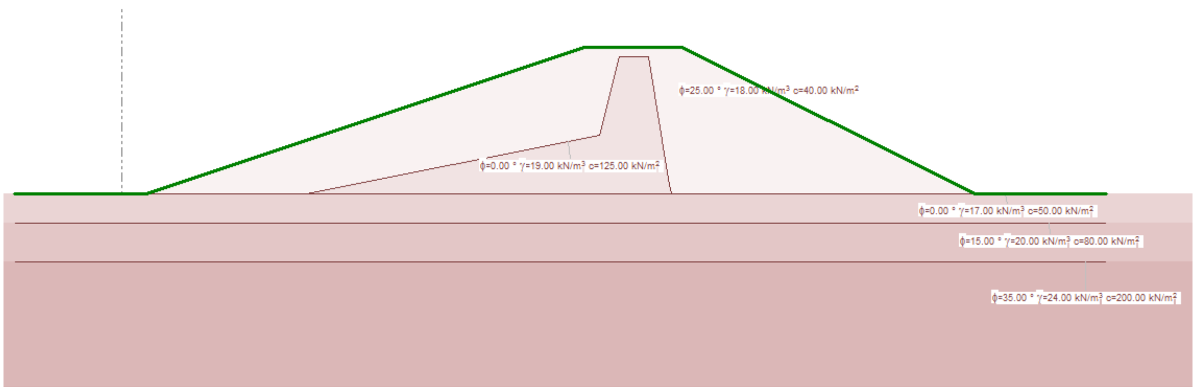


Figure 13. Model setup for Slope Stability Analysis of the Carsington Earth Embankment Dam exactly after construction of the embankment dam but prior to filling with water. (Total shear strength parameters of soils - No water table present in the reservoir and in the embankment dam body).

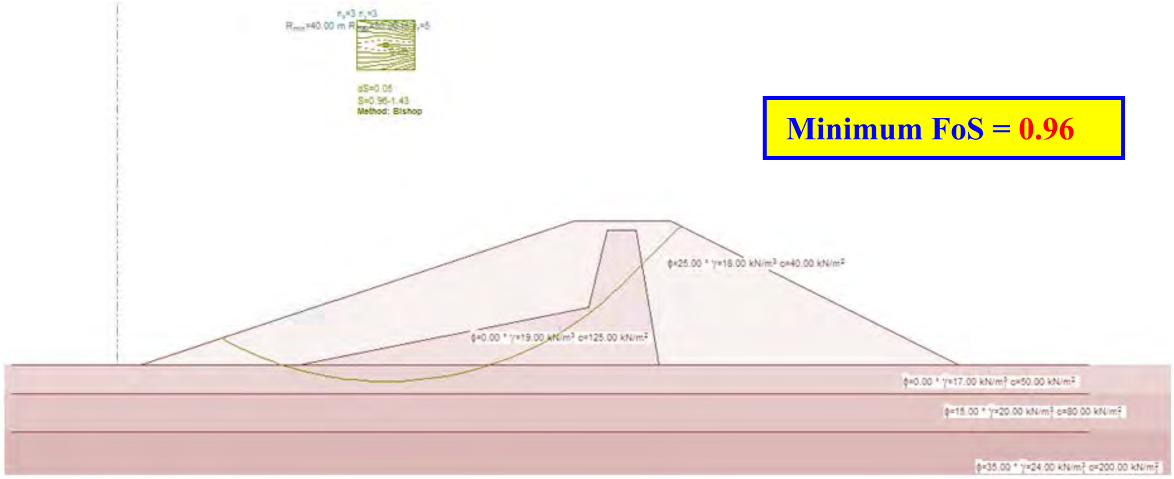
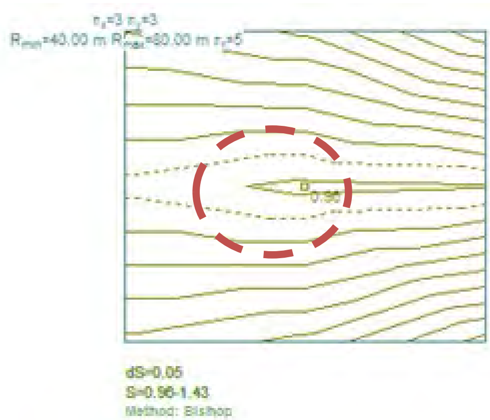


Figure 11a. Overall slope stability Factor of Safety for the loading case (a). Minimum FoS = 0.96, which means imminent failure conditions. (Bishop Method).




```
RESULTS
-------
Design situation: Standard hazard scenario

SLIP CIRCLE WITH MINIMAL SAFETY
Circle    X        Y        R    CmP-No    Anchor    S-Fc    Denomin    L-req
  No.    [m]      [m]      [m]                                 [kN]      [m]
----------------------------------------------------------------------------
24      55.92    73.56    70.00                        0.96    6544.95
```

Figure 11b: Overall slope stability Factor of Safety for the loading case (a). Minimum FoS = 0.96, which means imminent failure conditions. (Bishop Method).

Slope Stability Analysis Results for the Steady Seepage Condition [Loading case (b)]

The loading case (b) to be analysed is when the reservoir is full of water and some steady state seepage into the Carsington Earth Embankment Dam is established.

The analysis was performed using:

- ✓ A model with the simulation of the Carsington Earth Embankment Dam body and its foundation soil layers with all applied loads. The model is shown in Figure 12
- ✓ The exact Water Table Profile in the reservoir and inside the body of the Carsington Earth Embankment Dam is shown in Table 4a & 4b
- ✓ Effective or drained shear strength parameters of soils
- ✓ No seismic actions were taken into account
- ✓ Bishop Method for Slope Stability Analysis

For performing the slope stability analysis for the Steady Seepage Condition [Loading case (b)], the same specialized Civil & Geotechnical Engineering computer Program Larix / Cubus was used.

The overall minimum stability factor of safety for the loading case (b), i.e. when the reservoir is full of water and some steady state seepage into the Carsington Earth Embankment Dam is established, was calculated equal to **0.81**, which means failure conditions. The computed analysis results are illustrated in the following Figures (Fig. 13a and Fig. 13b).

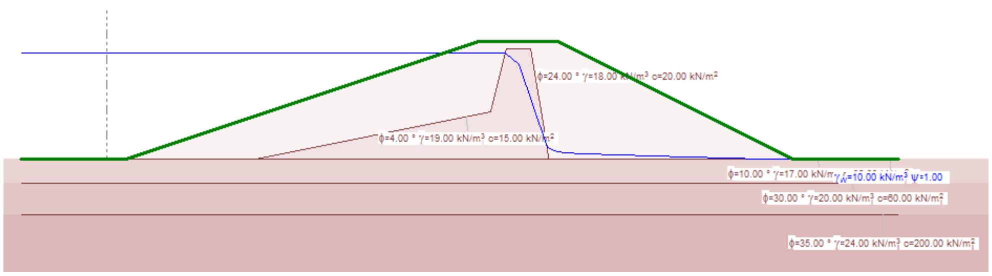


Figure 12: Model setup for Slope Stability Analysis of the Carsington Earth Embankment Dam when the reservoir is full of water and some steady seepage into the embankment is established. (Effective shear strength parameters of soils - Phreatic Surface is present in the embankment dam body and Water Table in the reservoir).

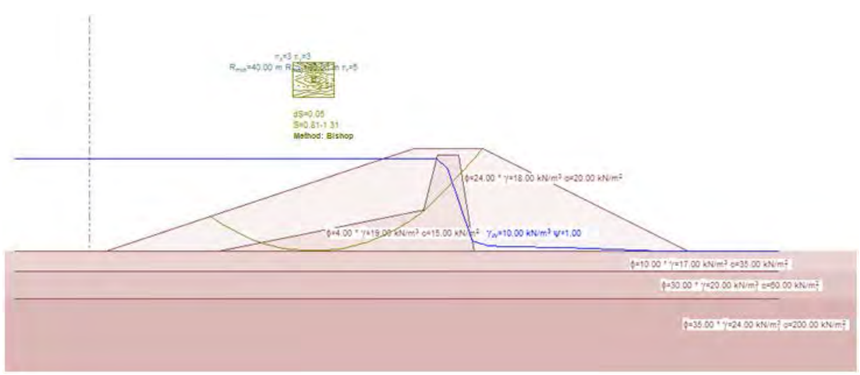
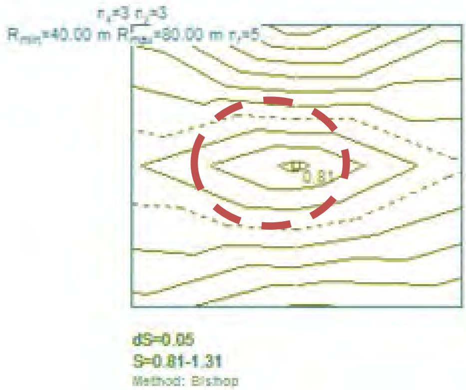


Figure 13a: Overall slope stability Factor of Safety for the loading case (b). Minimum FoS = 0.81, which means failure conditions. (Bishop Method)




```
RESULTS
-------
Design situation: Standard hazard scenario

SLIP CIRCLE WITH MINIMAL SAFETY
Circle    X        Y        R    CmP-No    Anchor     S-Fc    Denomin    L-req
  No.    [m]      [m]      [m]                                 [kN]      [m]
----------------------------------------------------------------------
22      65.63    57.10    50.00                       0.81    2693.64
```

Figure 13b: Overall slope stability Factor of Safety for the loading case (b). Minimum FoS = 0.81, which means failure conditions. (Bishop Method).

Slope Stability Analysis Results for the Rapid Drawdown Condition [Loading case (c)]

The loading case (c) to be analysed is when there is a Rapid Drawdown in the reservoir water level which may cause the upstream face stability to become very critical.

The analysis was performed using:

- ✓ A model with the simulation of the Carsington Earth Embankment Dam body and its foundation soil layers with all applied loads. The model is shown in Figure 14
- ✓ No water table in the reservoir but in the embankment dam body there are still full pore water pressures
- ✓ Effective or drained shear strength parameters of soils
- ✓ No seismic actions were taken into account
- ✓ Bishop Method for Slope Stability Analysis

For performing the slope stability analysis in this loading case (c), the same software was used, (Larix / Cubus).

The overall minimum stability factor of safety for the loading case (c), i.e. when there is a Rapid Drawdown in the reservoir water level, was calculated equal to **0.50**, which means absolute & immediate failure conditions. The computed analysis results are illustrated in the following Figures (Fig. 15a and Fig. 15b).

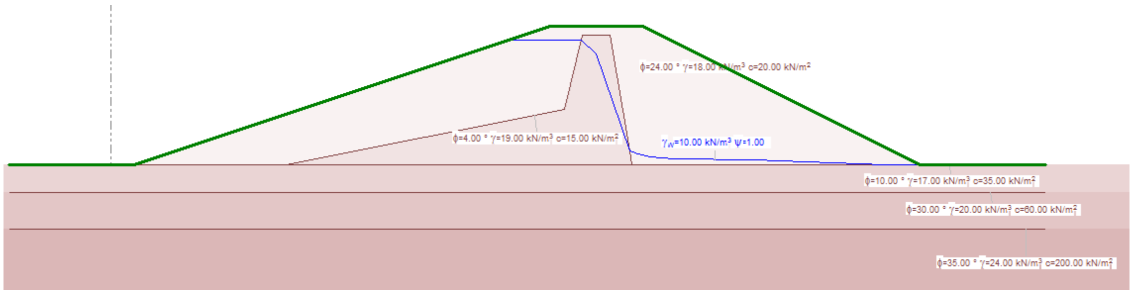


Figure 14: Model setup for Slope Stability Analysis of the Carsington Earth Embankment Dam in Rapid Drawdown Condition in the reservoir. (Effective shear strength parameters of soils - Phreatic Surface is present in the embankment dam body but no Water Table is present in the reservoir).

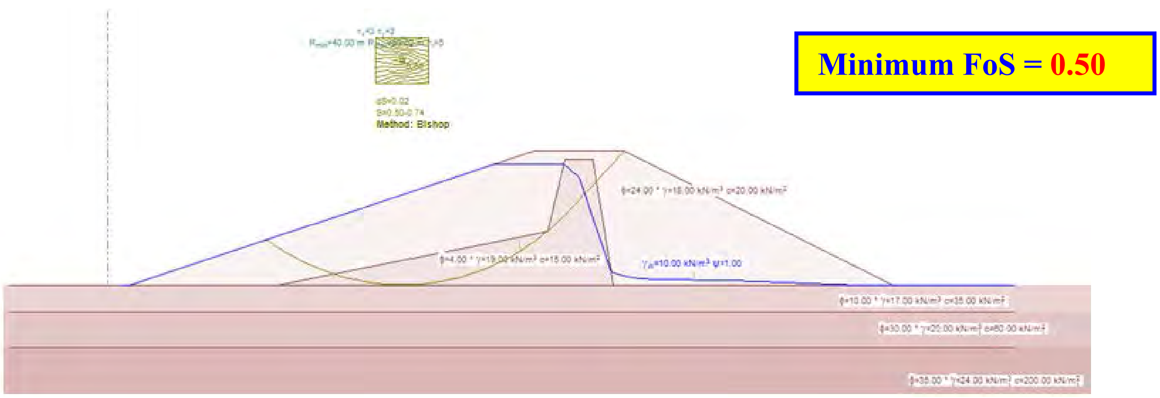
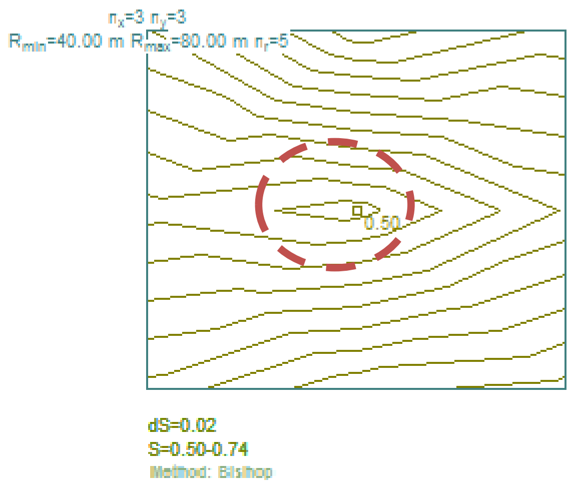


Figure 15a: Overall slope stability Factor of Safety for the loading case (c). Minimum FoS = 0.50, which means absolute & immediate failure conditions. (Bishop Method).




```
RESULTS
-------
Design situation: Standard hazard scenario

SLIP CIRCLE WITH MINIMAL SAFETY
Circle    X        Y        R    CmP-No    Anchor     S-Fc    Denomin    L-req
  No.    [m]      [m]      [m]                                  [kN]      [m]
----------------------------------------------------------------------------------
22      65.63    57.10    50.00                       0.50    4800.86
```

Figure 15b: Overall slope stability Factor of Safety for the loading case (c). Minimum FoS = 0.50, which means absolute & Immediate failure conditions. (Bishop Method).

GLOBAL SLOPE STABILITY ANALYSIS OF THE CARSINGTON EARTH EMBANKMENT DAM USING THE SHEAR STRENGTH REDUCTION ANALYSIS METHOD (F.E.A.)

For purposes of verification of the Global Slope Stability Analysis of the whole Carsington Earth Embankment Dam, for the Loading case (a), i.e. right after construction condition of the Carsington Earth Embankment Dam but prior to filling with water, the Shear Strength Reduction (S.S.R.) Analysis Method, based on the Finite Element Analysis technique (F.E.A.), was executed.

For performing the Shear Strength Reduction (S.S.R.) Analysis the specialized Civil & Geotechnical Engineering computer program Phase2 8 / Rocscience Inc was used.

The analysis results showed that the critical slope side of the Carsington Earth Embankment Dam right after its construction is the downstream slope side.

As proved, the global Factor of Safety (FoS) or the Strength Reduction Factor (S.R.F.) according to the Shear Strength Reduction (S.S.R.) Analysis Method for the whole Carsington Earth Embankment Dam structure is: **0.85**, which means failure conditions for the dam.

In the next Figures (Fig. 16 to 19) the following information of the Shear Strength Reduction (S.S.R.) Analysis Model is concisely illustrated:

- ✓ All Input Data
- ✓ The Mesh and Discretization
- ✓ The Mesh and boundary conditions,
- ✓ The Material Properties Definition,
- ✓ The Material Properties Assignment to the Earth Embankment Dam model,
- ✓ The final Analysis Results of the Strength Reduction Factor (S.R.F.)

General Settings

- Single stage model
- Analysis Type: Plane Strain
- Solver Type: Gaussian Elimination
- Units: Metric, stress as kPa

Analysis Options

- Maximum Number of Iterations: 500
- Tolerance: 0.001
- Number of Load Steps: Automatic
- Convergence Type: Absolute Energy
- Tensile Failure: Reduces Shear Strength
- Joint tension reduces joint stiffness by a factor of 0.01

Strength Reduction Settings

- Initial Estimate of SRF: 1
- Step Size: Automatic
- Tolerance (SRF): 0.01
- Limit SSR Search Area: No
- Apply SSR to Mohr-Coulomb Tensile Strength: Yes
- Convergence Parameters: Automatic

Groundwater Analysis

- Method: Piezometric Lines
- Pore Fluid Unit Weight: 9.81 kN/m3
- Probability: None

Field Stress

- Field stress: gravity
- Using actual ground surface
- Total stress ratio (horizontal/vertical in-plane): 1
- Total stress ratio (horizontal/vertical out-of-plane): 1
- Locked-in horizontal stress (in-plane): 0
- Locked-in horizontal stress (out-of-plane): 0

Mesh

- Mesh type: uniform
- Element type: 6 nodded triangles
- Number of elements: 2773

- Number of nodes: 5764

Mesh Quality

- All elements are of good quality

Poor quality elements defined as:

- Side length ratio (maximum / minimum) > 30.00
- Minimum interior angle < 2.0 degrees
- Maximum interior angle > 175.0 degrees

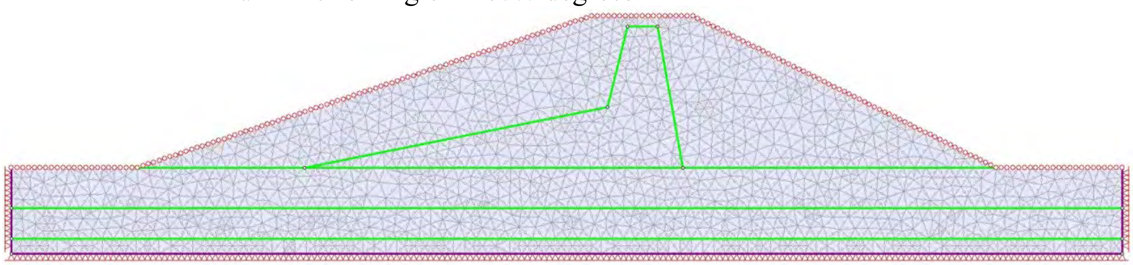


Figure 16: Mesh and Discretization Mesh and default boundary conditions.

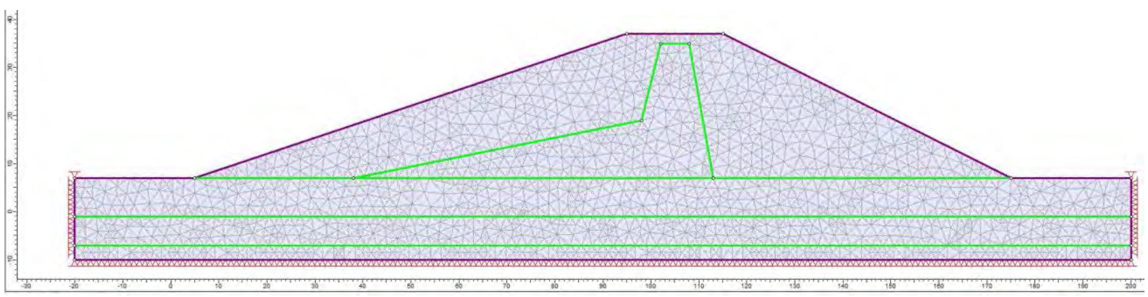
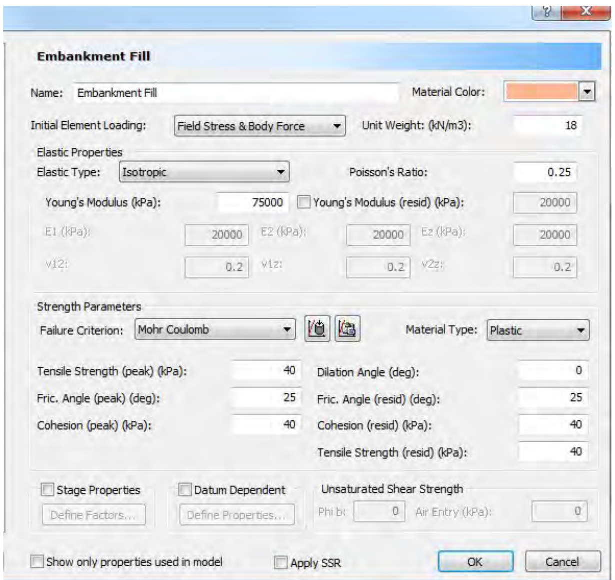
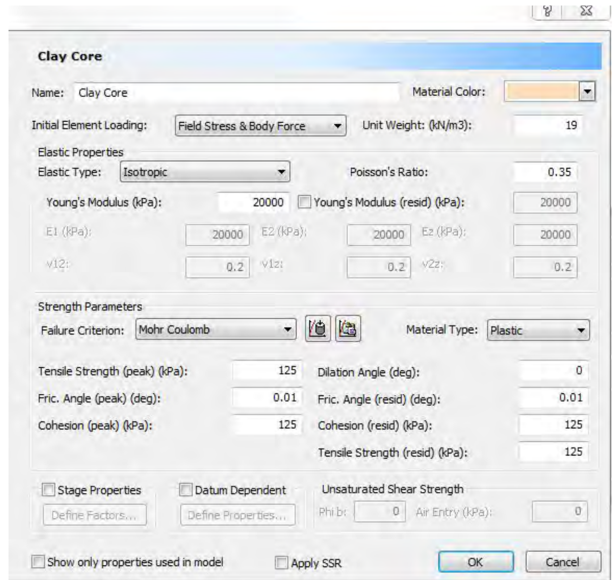
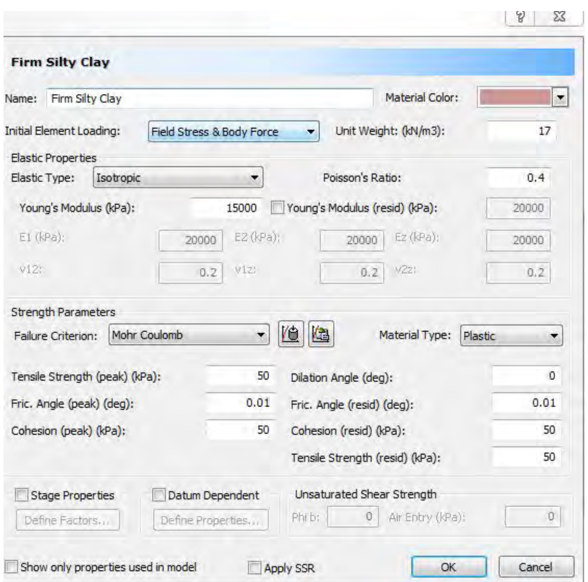


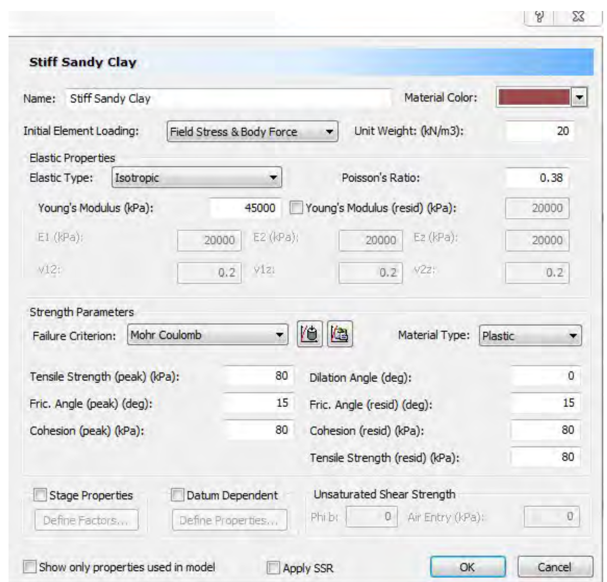
Figure 17: Mesh and Discretization Free boundary condition applied to ground surface.

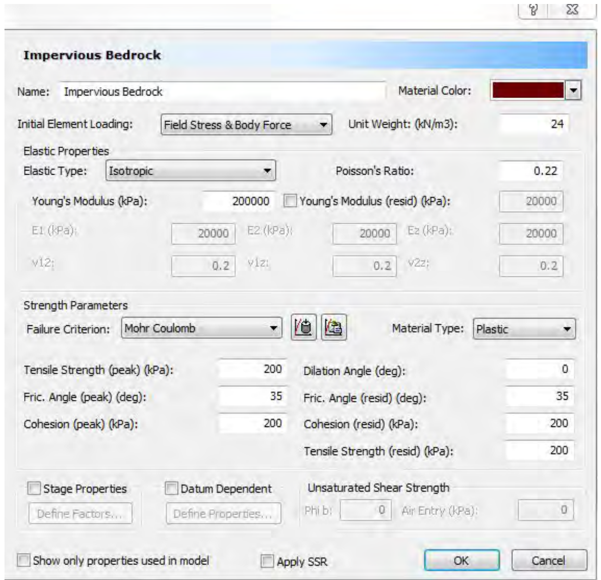
Material Properties











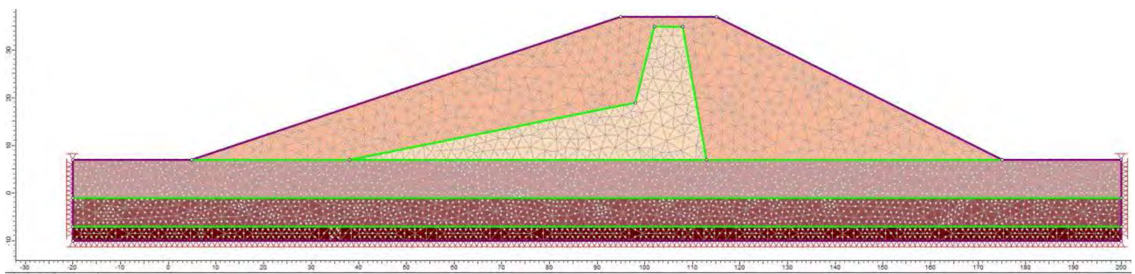


Figure 18: Mesh Material Properties Assigned to the Carsington Earth Embankment Dam model.

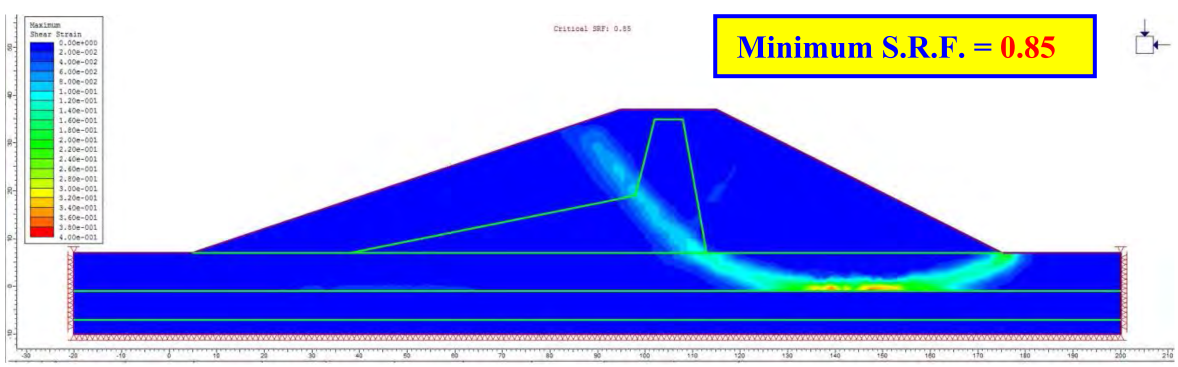


Figure 19: Analysis results of the Shear Strength Reduction Factor. (Indicates Failure).

SLOPE STABILITY ANALYSIS OF THE CARSINGTON EARTH EMBANKMENT DAM DURING LOADING CASE (A) USING TAYLOR'S CURVES

General

Taylor (1937) published a set of charts that relate stability coefficient, Ns to slope angle, β, in order to make the search for the lowest value of FoS easier. The charts assume the soil to be homogeneous and relate to total stress only.

Based on the principle of geometric similarity, Taylor (1937) published stability coefficients for the analysis of homogeneous slopes in terms of total stress. For a slope of height H as shown in Figure 20, the stability coefficient (N_s) for the failure surface along which the factor of safety (FoS) is a minimum is given by

$$N_s = \frac{c_u}{F\gamma H}$$

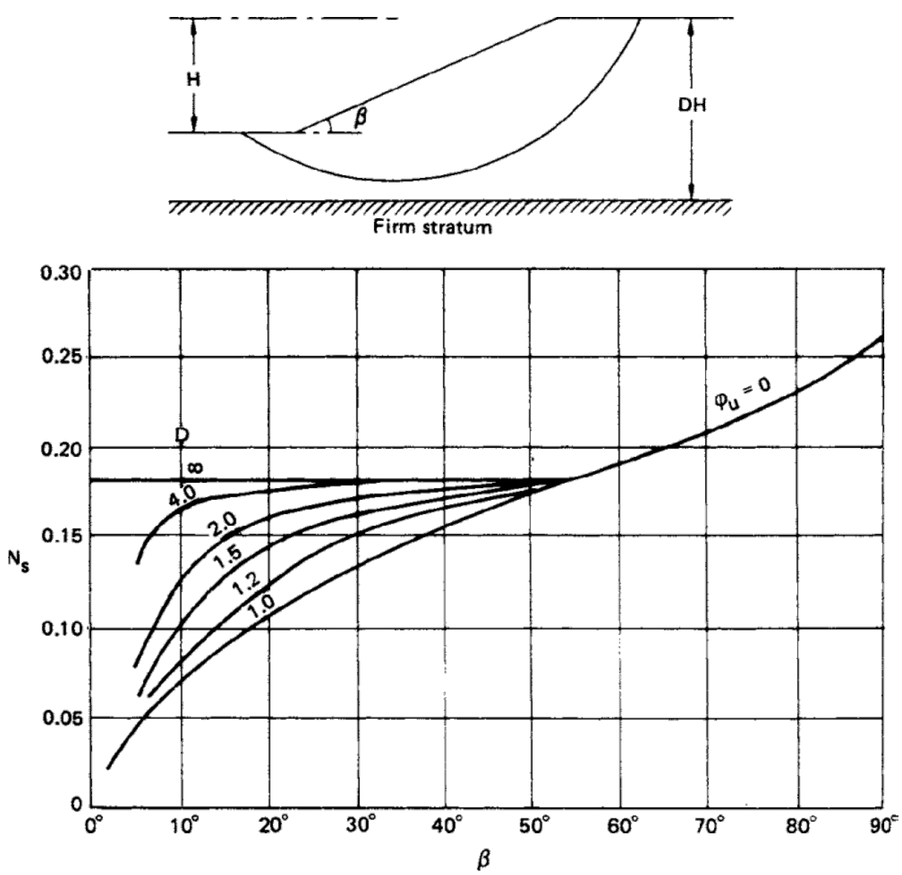


Figure 20: Taylor's stability coefficients for $\varphi_u = 0$ (Craig, 2004).

For the case of $\varphi_u = 0$, values of N_s can be obtained from Figure 20. The coefficient N_s depends on the slope angle β and the depth factor D, where DH is the depth to a firm stratum.

Gibson and Morgenstern (1962) published stability coefficients for slopes in normally consolidated clays in which the undrained strength cu ($\varphi_u = 0$) varies linearly with depth.

Taylor's chart is also conservative in that an increase in φ_u leads to a reduction in N_s and consequent increase in the factor of safety (FoS), because if a slope stands up at φ_u = zero it will always stand up at greater φ.

Calculation of the FoS against a slip failure occurring during loading case (a) using Taylor's curves

According to research's requirements, the safety factor against a slip failure occurring during loading case (a) using Taylor's curves must be calculated.

Hence, following the above mentioned procedure and Taylor's stability coefficients curves, that was achieved for our case study for the loading case (a), i.e. exactly after construction of the Carsington Earth Embankment Dam but prior to filling with water, as follows.

For this analysis using Taylor's curves, the following input data were taken into account:.

Embankment Fill:

✓ Unit weight	γ	=	18.00	kN/m^3
✓ Total Angle of internal friction:	φ_u	=	25.00°	
✓ Total Cohesion of soil	c_u	=	40.00	kPa
✓ Slope Angle β		=	18.5°	

For applying the above mentioned procedure, the following on scale diagram of the Carsington Earth Embankment Dam cross section was considered, as shown in Figure 21.

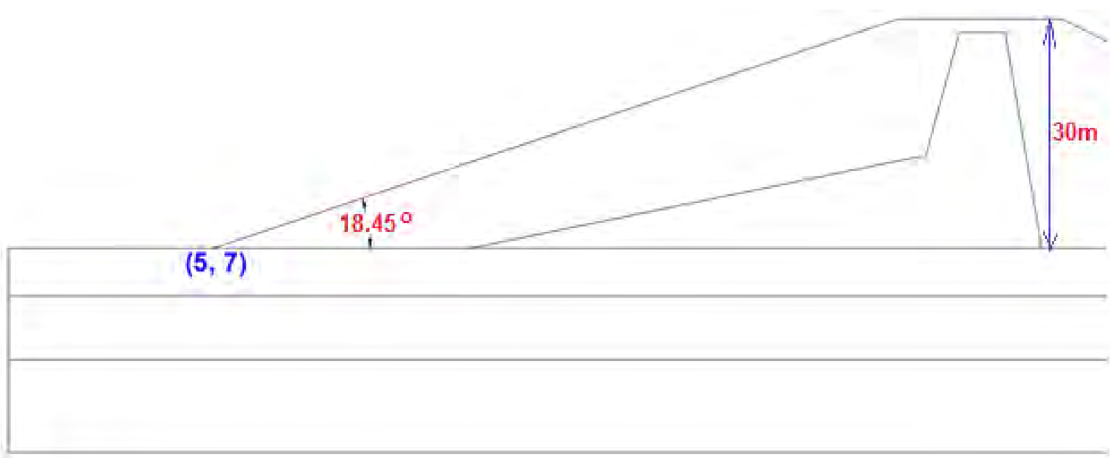


Figure 21: Part of the on scale cross section of the Carsington Earth Embankment Dam for the calculation of the FoS against a slip failure occurring during loading case (a) using Taylor's curves.

Therefore, the minimum factor of safety against a slip failure occurring during loading case (a) using Taylor's curves can be estimated by using the stability coefficient (N_s). From Figure 21, $\beta = 18.45^o$ and D = 1, the value of N_s is 0.10. Then

$$F = \frac{c_u}{N_s \gamma H}$$

$$F = \frac{40}{0.10x18x30}$$

$$\boxed{\mathbf{F = 0.741}}$$

Therefore, the factor of safety (FoS) against a slip failure occurring during loading case (a) using Taylor's curves is as low as F = 0.741, which means failure conditions of the Carsington Earth Embankment Dam exactly after construction of the embankment dam but prior to filling with water.

However, in this point an important conclusion can be drawn. If we compare the factor of safety (FoS) against a slip failure occurring during loading case (a) as calculated by using Taylor's curves method with the FoS as calculated by the moment equilibrium Bishop's method and presented in previous section, we can see that the FoS calculated by Taylor's curves method is lower by 22,8 %.

Actually, this fact is reasonable and it was expected for two main reasons:

a) In Taylor's curves method, the beneficial for the stability effect of the undrained angle of internal friction, which is greater than 0, is not taken into account, and
b) In Taylor's curves method, the beneficial effect of the Clay Core soil, which has an undrained cohesion greater than the undrained cohesion of the Embankment Fill soil, is not taken into account either.

Therefore, the lower resulted value of the FoS when using Taylor's curves method compared to the higher resulted value of the FoS when using the moment equilibrium Bishop's method is fully justified and expected.

SUMMARY, DISCUSSION AND CONCLUSIONS

A detailed slope stability analysis and assessment of the original Carsington Earth Embankment Dam failure in the UK was attempted in this paper.

A 1225 m long, 35 m high zone earth filled embankment was being constructed from 1981 to 1984 from a British Regional Water Authority to regulate flows in the River Derwent in England. Its reservoir capacity was 35 million m^3 and the watertight element was Rolled Clay Core with an upstream extension of boot shaped and shoulders of compacted mudstone with horizontal drainage layers of crushed limestone about 4 metres apart and a cut-off grout curtain (Davey and Eccles, 1983).

However, at the beginning of June 1984, a 400 m length of the upstream shoulder of the embankment dam slipped some 11 m and failed. At the time of the failure, embankment construction was virtually complete with the dam approaching its maximum height of 35 m. The failure surface passed through the boot shaped rolled clay core and a relatively thin layer of surface clay in the foundation of the dam.

By using and applying advanced geotechnical engineering analysis tools and modelling techniques the Carsington Earth Embankment Dam, which is considered a particular geotechnical structure, was analysed in this paper.

In this detailed slope stability analyses, the total and effective stress state soil properties / parameters were used, and the most critical slip circle centre according to Fellenius - Jumikis method was initially determined. Subsequently, the Carsington Earth Embankment Dam and its foundation was analysed and examined against failure by slope instability. Considerations of loading conditions which may result to instability for all likely combinations of reservoir and tailwater levels, seepage conditions, both after and during construction were made, and hence three construction and / or loading conditions were examined in particular:

- The right after Construction Condition [Loading case (a)],
- The Steady Seepage Condition [Loading case (b)], and
- The Rapid Drawdown Condition in the reservoir [Loading case (c)]

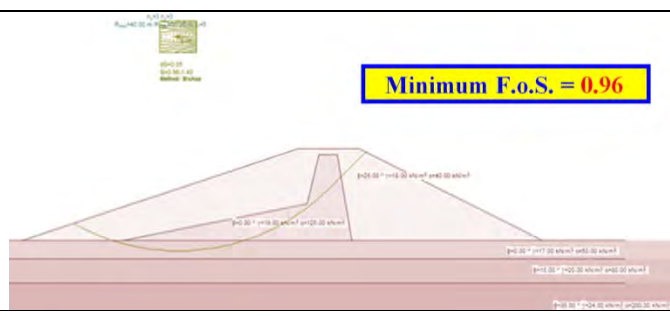
In this context, the slope stability at the three above mentioned discrete loading cases of the Carsington Earth Embankment Dam was analysed and presented, and certain valuable conclusions concerning the overall stability conditions of the Earth Embankment Dam during its original construction were deduced in this research paper.

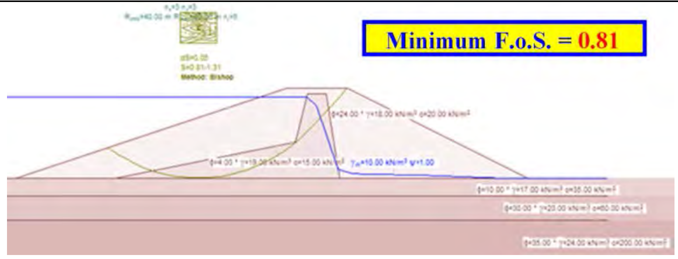
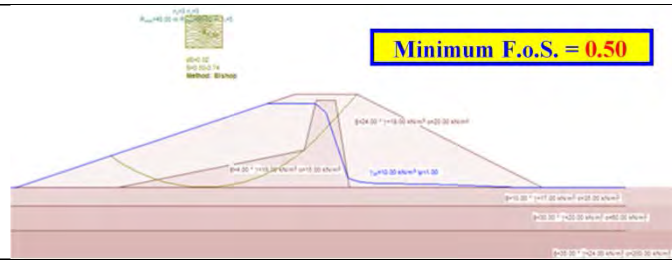
In addition, for comparison reasons, a Slope Stability Analysis of the Carsington Earth Embankment Dam during loading case (a) using Taylor's curves was performed. Furthermore, the Shear Strength Reduction (S.S.R.) Analysis Method, based on the Finite Element Analysis technique (F.E.A.), was executed, for purposes of verification of the Global Slope Stability Analysis of the whole Carsington Earth Embankment Dam for the Loading case (a), i.e. right after construction condition of the Dam but prior to its filling with water, which proved that the results between the Shear Strength Reduction (S.S.R.) Analysis Method and the Limit Equilibrium Analysis Method (LEM) based on the method of slices are comparable and similar.

From these detailed slope stability analysis results, the reasons as to why the Carsington Earth Embankment Dam failed during its original construction became obvious.

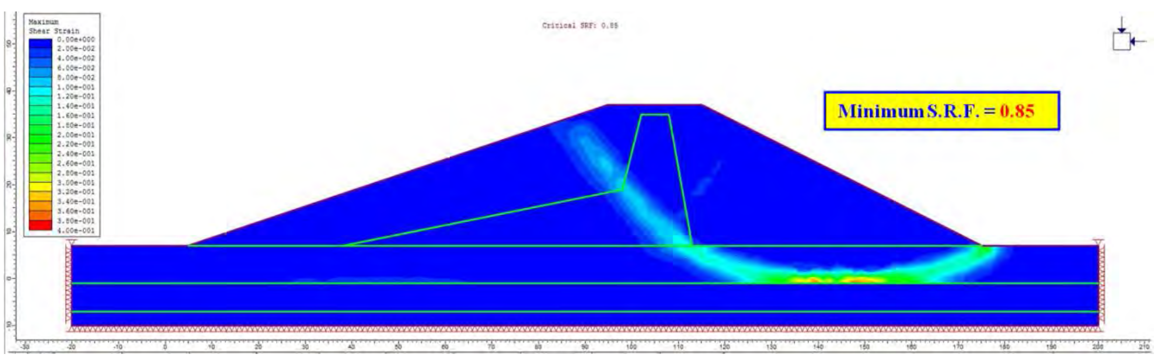
According to the detailed slope stability analysis results, the following conclusions can be deduced when considering the three examined construction and / or loading conditions:

a) The overall minimum stability factor of safety for the loading case (a), i.e. exactly right after construction of the Carsington Earth Embankment Dam but prior to filling with water, was calculated equal to **0.96**, which means imminent failure conditions.



b) The overall minimum stability factor of safety for the loading case (b), i.e. when the reservoir is full of water and some steady state seepage into the Carsington Earth Embankment Dam is established, was calculated equal to **0.81**, which means failure conditions.	
c) The overall minimum stability factor of safety for the loading case (c), i.e. when there is a Rapid Drawdown in the reservoir water level, was calculated equal to **0.50**, which means absolute & immediate failure conditions.	

For purposes of verification of the Global Slope Stability Analysis of the whole Carsington Earth Embankment Dam structure at the Loading case (a), the Shear Strength Reduction (S.S.R.) Analysis Method was executed, based on the Finite Element Analysis technique (F.E.A.). As proved, the global Factor of Safety (FoS) or the Strength Reduction Factor (S.R.F.) for the whole Carsington Earth Embankment Dam structure was determined equal to **0.85**, which means failure conditions for the dam.



In addition, for comparison reasons, a Slope Stability Analysis of the Earth Embankment Dam during loading case (a) using Taylor's curves was performed, and showed that the factor of safety (FoS) against a slip failure occurring during loading case (a) was as low as F = 0.741, which means failure conditions of the Carsington Earth Embankment Dam.

However, if the factor of safety (FoS) against a slip failure occurring during loading case (a) as calculated by using Taylor's curves method, is compared to the FoS as calculated by the moment equilibrium Bishop's method, it can be seen that the FoS calculated by Taylor's curves method is lower by 22,8 %. Actually, this fact is considered reasonable and it was expected for two main reasons:

a) In Taylor's curves method, the beneficial for the stability effect of the undrained angle of internal friction, which is greater than 0, is not taken into account, and

b) In Taylor's curves method, the beneficial effect of the Clay Core soil, which has an undrained cohesion greater than the undrained cohesion of the Embankment Fill soil, is not taken into account either.

Therefore, the lower resulted value of the FoS when using Taylor's curves method is fully justified and expected.

Moreover, the reasons why the Fellenius - Jumikis method is inaccurate can be summarised as follows:

a) The soils of the Carsington Earth Embankment Dam may not be purely $\varphi_u = 0$ soils, as Fellenius (1936) proposed,
b) The soils of the Carsington Earth Embankment Dam may not be homogeneous c' - φ' soils, (because there might be different soil layers in the sloping embankment body and different soils in the impermeable core) as Jumikis (1962) assumed to further extended Fellenius's method,
c) The existence of water tables and pore water pressures are not taken into account in Fellenius - Jumikis method,
d) Fellenius - Jumikis method becomes less reliable for non-homogeneous conditions, such as irregular slope profile,
e) Surcharging and other type of loading of slopes is not taken into account in Fellenius - Jumikis method, and
f) Seismic loading of slopes is not taken into account in Fellenius - Jumikis method. Seismic loading deteriorates stability conditions and a special treatment is required, (Anastasiadis et al, 2006).

Therefore, due to the above mentioned reasons of inaccuracy the determination of the location of the most critical slip circle centre in an earth embankment dam by using Fellenius - Jumikis method could not be accurate enough. But nevertheless, this determination can dramatically help to advantage and reduce the number of computational trials needed.

According to all above mentioned results, derived from the slope stability analyses, became obvious and well documented why the original Carsington Earth Embankment Dam failed just upon its construction completion at the beginning of June 1984.

Based on the results of this research study as well as the observations made by Johnston et al., 1999, the causes of failure of the Carsington Earth Embankment Dam can be summarized as follows:

- Two different figures of Factor of Safety were used for the upstream and downstream slopes showing that the slope and zoning had not been designed to meet a minimum Factor of Safety.
- During construction, the Contractor found the requirements for testing and removing clay from the foundation to be inconsistent resulting in weak material being left in place.
- None of the assumed slip surfaces, of which the Consulting Engineers stated that were analyzed, passed through the upper part of the core boot section, or the upstream fill/foundation junction.
- It was apparent that the Factors of Safety were obviously too low for the structure.

Finally, the technical and engineering lessons learned from this large scale Earth Embankment Dam body failure can be summarized as follows:

- Correct and conservative shear strength parameters should be used,
- Realistic pore pressure assumptions should be made,
- Accepted minimum Factors of Safety should be used,
- Critical slip surfaces and global slope stability should be rigorously analyzed,
- Instrumentation should be planned related to the design, and
- A design report should be prepared and reviewed by a panel of experts or other advisors and geotechnical engineering consultants.

Essential lessons learned during construction include:

- The design should be reassessed when further strength and performance data are obtained,
- The design Factors of Safety should be re-examined, and
- The technical data and review should be available to the site, contractor and client as required.

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