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Failures of a Monozone Earth Dam of Expansive Clay

Ruptures d'un Barrage en Terre Monozone d'Argile Gonflante

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SYNOPSIS Failures of Waghad dam constructed of highly plastic clays at construction, steady seepage and drawdown condition have been analysed by Swedish slip circle and Bishop's semirigorous methods. The study indicated that the state of stability and the plane of slip are correctly represented by Bishop's method. Other important conclusions regarding behaviour of monozone dam at different conditions are also presented.

INTRODUCTION

Waghad, the then tallest Indian earth dam, constructed by British engineers in the year 1881 is a homogeneous earth dam of highly plastic expansive black clay, average liquid limit being 70% and plastic limit 35%. The maximum height of the dam above deepest foundation in gorge is 32 m. The foundation of the dam consisted of murum and weathered rock and is thus stronger than embankment material. The gross storage capacity of reservoir is $17 \times 10^6 \text{ m}^3$ and the catchment area is $75 \times 10^6 \text{ m}^2$. The dam has shown several distresses during its life.

The earth dam under study is submerged from 1979 in the reservoir of newly constructed earth dam located at about 1.5 km downstream. The opportunity was availed in the year 1977 to investigate the causes of different slips occurred in the life of the dam. The scope of the present paper is limited to ascertain the applicability of a particular method of analysis (Swedish slip circle or Bishop's semi-rigorous method) to represent appropriately the state of instability of the earth dam and plane of failure.

HISTORY OF INCIDENCES

Construction of the dam was started in the year 1881 as a famine relief work and was to be completed before the monsoon of 1883. Upto 15th May 1883 the dam at gorge was raised, at full section to a height of 19.5 m above river bed. Further height of about 6 m was raised with smaller section. On 7th July it rained heavily causing abnormally large floods which filled the tank quickly and the dam was overtopped on the morning of 8th July.

During the backfilling of breached gorge portion of the earth dam in the fair season when the dam top was almost reached, a slip occurred in April 1884 of downstream slope for

a length of 61 m. The downstream toe of the dam moved about 15.25 m and the top of the embankment subsided by 6.4 m.

In August 1907, a slip of downstream slope, accompanied with subsidence of top of the dam, took place between chainages 320 to 360 m. For the previous two years, 1905 and 1906, the rainfall was 796 mm and 847 mm respectively. In the year 1907 prior to the slip it rained heavily such that the rainfall in the month of August was 739 mm (54%) as against the total yearly rainfall of 1361 mm. The intense heavy rainfall might have triggered the slip.

The next slip on the upstream side occurred on 24th April 1919 between chainages 810 to 840 m and 1110 to 1140 m. Longitudinal crack 75 mm wide was formed at the upstream edge and the bank settled by 4.8 m.

On 31st July 1976, a slip occurred between chainages 319 to 412 m which was the same location of the slip that took place in the year 1907. The rainfall on the previous days of the slip was 14 mm on 28th July, 81 mm on 29th July and 199 mm on 30th July. Out of total 1406 mm rainfall in that year, 508 mm rainfall was in the month of July 1976. The rainfall in the previous year of slip was as small as 889 mm. This slip might have also occurred due to intense rainfall after the years of scanty rains as in case of failure of slope in 1907.

INVESTIGATIONS

The dam was purposely breached at chainage 335 m and the final width of the breached portion was from chainage 320 to 412 m. This was the location at which slips of downstream slope took place in the years 1907 and 1976.

A trench about 35 m wide at top and 18 m deep was dug in the gorge portion and at every 1.5 m depth, at three locations, undisturbed soil samples were collected by jacking. Field density, moisture content, permeability and

vane shear tests were carried out. Collection of undisturbed samples and field tests were done in a similar way at breached portion. The zoning of the earth dam at both the faces of gorge and breached portions as well as distribution of cracks observed on the face nearby chainage 1110 m were noted.

16 direct shear and 11 triaxial shear tests, both consolidated drained, were carried out on undisturbed soil samples. Table I indicates the range of index and engineering properties obtained on the soil samples in breached and gorge portions. On the basis of these test results, 75% reliable values of density and shear strength parameters were used for the analyses of slips at these locations. Table II indicates the year of slip, conditions assumed and corresponding critical factors of safety.

TABLE I

Index and Engineering Properties of Soil in Breached and Gorge Sections

Sr. No.	Property	Unit	Range	
			Breached Section	Gorge Section
1	Liquid Limit	-	67 - 84	60 - 77
2	Plasticity Index	-	24 - 42	30 - 48
3	Clay Content	%	39 - 44	43 - 60
4	Shrinkage Limit	-	-	8 - 15
5	Specific Gravity	-	2.58-2.83	2.32-2.96
6	Field Density	kg/m ³	1100-1580	1110-1500
7	Moisture Content	%	30 - 40	30 - 39
8	Direct Shear	C'	0.0 - 0.0	0.0 - 2.5
		φ'	24.2-25.6	11.6-29.96
9	Triaxial Shear	C'	-	0.0-2.5
		φ'	-	17.33-22.6
10	Swelling Pressure	kPa	-	10.0

In order to understand the development of pore water pressure during construction, the initial negative pore water pressures of soil samples remoulded at known wet densities were measured in triaxial apparatus with high air-entry value porous stone. The decrease in the value of negative pore water pressure with step by step increase in the cell pressure was noted. The increase in cell pressure from zero to 30 kPa resulted in small increase in the pore water pressure i.e. from -2.3 kPa to +2.3 kPa, because of expansive nature of the soil. It is, therefore, expected that the development of construction pore water pressure would be small even though the soil was compacted at water content wetter (approximately by 6%) than

optimum moisture content.

ANALYSIS OF SLIP DURING CONSTRUCTION

Slip of 1884

The stability analyses of upstream and downstream slopes of the dam section, as constructed in the year 1884, at construction condition with pore water pressure factors (ru) varying from 0.5 to 0.1 were carried out by Swedish slip circle and Bishop's semirigorous methods. Factors of safety obtained are given in Table II.

TABLE II

Conditions Assumed and Factors of Safety Obtained for Various Slips

Year of slip	Conditions assumed	Critical Factors of safety	
		Bishop's semi-rigorous method	Swedish slip circle method
1884	Construction		
	a) Downstream slope		
	i) ru = 0.1	0.75	0.72
	ii) ru = 0.3	0.66	0.62
	iii) ru = 0.5	0.56	0.52
	b) Upstream slope		
i) ru = 0.1	1.25	-	
ii) ru = 0.3	1.13	0.89	
iii) ru = 0.5	1.01	0.76	
1907	Steady seepage		
	a) Buoyant condition above phreatic line	0.83	0.72
	b) Without buoyant condition above phreatic line	1.18	1.04
1919	Drawdown with observed pore pressures	0.99	0.61
1976	Steady seepage		
	a) Buoyant condition above phreatic line	0.99	0.89
	b) Without buoyant condition above phreatic line	1.29	1.12

As per the results of stability analysis for upstream slope by Swedish slip circle method, with variation of pore pressure factors from 0.1 to 0.5, the upstream slope is unstable and should have failed which is not the case. This method of stability analysis thus does not represent the correct state of stability of the upstream slope. The corresponding values of factors of safety by Bishop's semirigorous method are more than one which correctly represent stable upstream slope.

The factors of safety for downstream slope by both the methods with the variation of pore pressure factors are so low that the dam section should have failed at a height much

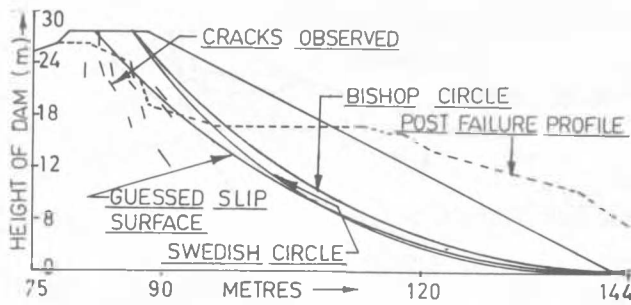


Fig. 1 Stability Analysis of 1884 Slip

lower than the height at which it failed. The dam actually failed almost at its full height. This indicates that the pore pressure developed in the dam during construction might be smaller than 0.1 and may actually be on the negative side. Figure 1 shows the slip surfaces based on crack pattern observed and obtained by both the methods of stability analysis for $ru = 0.1$. All the three surfaces are almost coinciding. Although reported by Strange (1898) that the quantity of water added into the soil during compaction was rather large still immediately after compaction of the soil negative pore pressures might have been developed. The expansive nature of soil might be responsible for development of rather large negative pore pressures which would not have become positive even after the addition of moderate overburden load (not more than 45 kPa) due to embankment construction. It, therefore, seems that even if there would not have been development of positive pore water pressure, 2:1 slope on the downstream side was inadequate and 3:1 slope as given to upstream side would have been just sufficient to make it stable.

The slope of the dam was not designed at that time (one century before) but was adopted empirically. This is an interesting and unique case where the downstream slope failed during construction not because of high construction pore water pressures but due to the inadequate section with regard to the shear resistance available. From this behaviour of the dam it may also be inferred that the embankment of moderate height constructed of expansive soil may not fail during construction because of development of high construction pore water pressures.

ANALYSIS OF SLIPS AT STEADY SEEPAGE CONDITION

Slip of August 1907

In order to investigate the cause of the slip of the downstream slope in 1907, the stability analysis was carried out considering the section existing in that year. The then engineers installed stand pipe piezometers in the year 1907 to observe the position of phreatic line particularly at full supply level which was adopted in the stability analysis.

The downstream slope at steady seepage condition might have failed because of inadequate section of this height of dam (14.37 m) for available shear resistance. Alternately

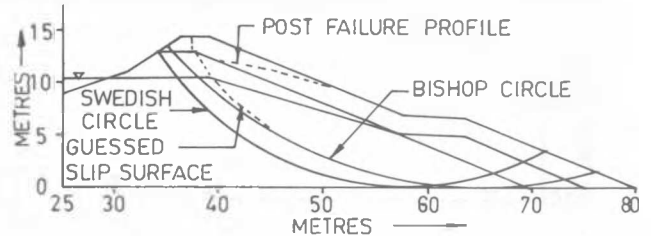


Fig. 2 Stability Analysis of 1907 Slip

the cracks developed upto phreatic line throughout the slope during long drought period get filled up during heavy rainfall with consequent increase in pore pressures so as to make the cracked portion buoyant and lead to failure. To assess the role of first possibility, the stability analysis was carried out at steady seepage condition assuming that cracks are not developed on the slope surface by Swedish slip circle and Bishop's semirigorous methods. The critical factors of safety obtained are 1.04 and 1.18 respectively indicating that downstream slope is stable for this height of dam.

The effect of second possibility was assessed by incorporating a buoyant zone over the downstream slope upto phreatic line in stability analysis. The critical circles obtained by Swedish slip circle and Bishop's semirigorous methods are shown in Figure 2. The critical factors of safety obtained by these two methods are 0.72 and 0.83 respectively.

The results of analysis indicate that for steady seepage condition for the height of dam between chainages 320 to 412 m, the downstream slope is stable but when the steady seepage condition is accompanied by heavy rainfall particularly after scarcity years, the downstream slope becomes unstable. This inference is further confirmed by the fact that adjacent sections of dam of lesser height did not fail in similar circumstances.

The stable profile of the slope after failure is shown in Figure 2. The probable plane of failure is determined by drawing a cycloid by Collin's method (1956) and is shown in the same figure. The plane of failure thus guessed coincides with critical circle obtained by Bishop's semirigorous method and not with that obtained by Swedish slip circle method.

Slip of 31st July 1976

The stability analyses by Swedish slip circle and Bishop's semirigorous methods were carried out with section existing in that year and assuming presence and absence of cracks. The factors of safety for these conditions are given in Table II. The conclusions arrived at after studying slip of 1907 are applicable to this slip also.

From Table II it is noted that difference in values of factors of safety is of the order of 0.3 assuming presence and absence of cracks in the analyses of slips which took place in 1907 and 1976. Usually the downstream slope for steady seepage condition is designed for the

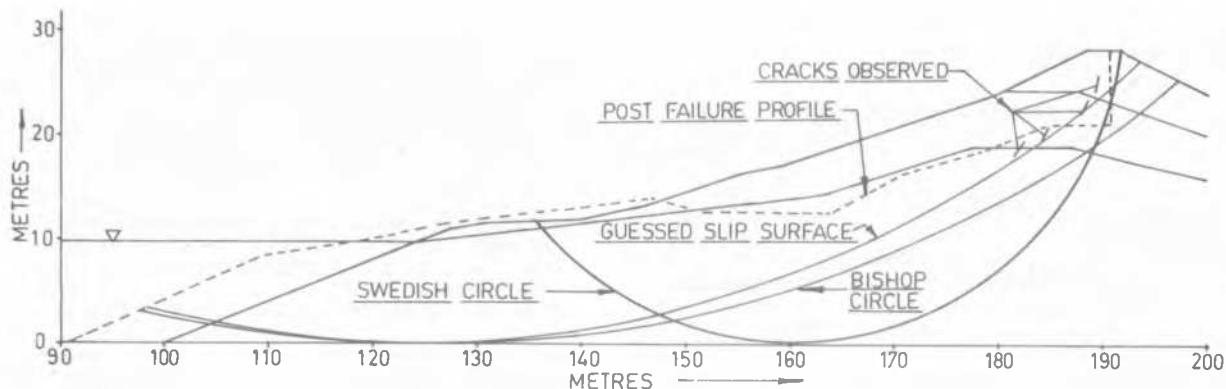


Fig. 3 Stability Analysis of Slip of 1919

factor of safety of 1.5 but without assuming excess pore pressures above phreatic line in the cracked portion in the analysis. It is, therefore, proposed that limit of factor of safety for the downstream slope be raised from 1.5 to 2.0. This increase in factor of safety may also cover the safety of dam against creep (Peterson et al, 1960). Alternately a cover of adequate thickness of casing material may be provided so that prolonged dry weather will not affect equilibrium moisture content of highly plastic material of which monozone dam is constructed.

ANALYSIS OF SLIP AT DRAWDOWN CONDITION

Slip of 1919

The stability analysis of upstream slope was carried out with the cross section existing before the slip. The position of phreatic line at drawdown condition was determined from the observational data of Casagrande type piezometers installed at this location. The pore pressure during drawdown in between the positions of phreatic lines observed at steady seepage and drawdown conditions was taken as zero which is realistic. While below the phreatic line at drawdown condition it was considered equal to the height of phreatic line above the slip planes. The critical factors of safety obtained by Swedish slip circle and Bishop's semirigorous methods are 0.61 and 0.99 respectively (Figure 3). The slip plane determined on the basis of cracks observed is shown in the same figure. From Figure 3 it may be noted that Bishop's semirigorous method not only represents the state of instability correctly but also the plane of failure.

CONCLUSIONS

1) From the results of stability analysis of downstream slip in the year 1884, it may be concluded that in case of homogeneous earth dams of moderate height constructed of highly plastic expansive clays with moisture content much above optimum, the critical condition for the safety of dam is steady seepage for downstream slope and drawdown for upstream slope rather than construction condition generally considered since the construction

pore pressures are much less than those developed at steady seepage or remain locked up at drawdown condition.

- 2) In case of such dams, proper vigilance is required during dry season, especially when it is prolonged as cracks may develop and subsequently slip may occur when followed by heavy rains.
- 3) Adequate cover, minimum 2 to 5 m, of casing material should be provided for decreasing or avoiding altogether formation of cracks due to desiccation or the limit of factor of safety may be raised upto 2.
- 4) The state of failure of upstream slope in the year 1919, was accurately represented with reference to factor of safety (0.99) as well as position of slip surface by Bishop's semirigorous method. It may be noted that in this analysis observed pore pressures are considered. Thus there is a scope of improving the present design assumptions as regards position of phreatic line during drawdown. For this purpose more similar cases need be studied.
- 5) Potential failure is more correctly represented by Bishop's semirigorous method than Swedish slip circle method and critical circle obtained by Bishop's semirigorous method fairly coincides with guessed/actual slip surface for all the conditions analysed.

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