Analysis of Failed Teton Dam

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***Abstract*—The failure of Teton Dam in Idaho in June, 1976, during the first filling of the reservoir, was a significant event for geotechnical engineers concerned with the design and construction of earth dams, simply because no dam of such a height (approximately 300 ft above stream bed) had previously failed. Thus the failure was both dramatic and of considerable importance. This paper reviews the events leading to the failure of Teton Dam to determine the cause of failure. Conclusions are presented regarding the probable trigger mechanisms which initiated the failure. In addition, an investigation of the area by geologist of the U.S. Geologic Survey indicated that it was seismically active: five earthquakes had occurred within 30 miles (50 km) of the dam site in the previous five years, two of which had been of significant magnitude. As it was a finite slope structure which failed during the first filling of the reservoir, therefore, in order to review the causes of failure, slope stability analysis of this dam profile has been done under structural stability, seepage and seismic analysis. The aforesaid analyses are accomplished using several methods that are available for stability analysis of finite slopes. The Fellenius and Bishop methods were used. "The Teton Dam Failure – A Retrospective Review" was silent about the seismic effect of Teton dam. Hence the present paper emphasized the seismic analysis by Fellenius method. Seismic analysis conducted in this project confirms the failure of Teton dam due to earthquake forces, as the critical factor of safety so obtained in seepage analysis i.e. 1.43 is reduced to 0.97 in seismic analysis**

Keywords*— *Probable Trigger Mechanism; Seismically Active; Dam Profile; Critical Factor Of Safety;

INTRODUCTION

The Teton dam site is composed of basalt and rhyolite, both of which are considered unsuitable for dam construction because of their high permeability. Test bores, drilled by engineers and geologists employed by the Bureau of Reclamation, showed that the rock at the dam site is highly fissured and unstable, particularly on the right side of the canyon. The widest fissures were determined to be 1.7 inches wide. The Bureau planned to seal these fissures by injecting grout into the rock under high pressure to create a grout curtain in the rock. The Teton Dam was located in a steep-walled canyon cut by the Teton river into a volcanic plateau known as the Rexburg Bench. A cross-section of the canyon approximately along the axis of the dam is shown in the Fig.1.

The walls of the canyon consist of later tertiary rhyolite welded-tuff which is strongly jointed, with joint widths varying at different elevations typically between ¼ to 3 inches but with occasional joints up to 12 inches wide. -Alluvium has been deposited in the river channel to a depth of about 100 ft. and the high lands near the ends of the dam are covered with an Aeolian silt deposit up to about 30 ft. thick. The primary features of the site are the extensive joint system in the rhyolite-tuff which makes it extremely permeable and the abundance of the wind-blown silt deposit which led the designers to use substantial quantities of this material in the dam cross-section. In 1973, when the dam was only half-built, but almost $5 million had already been spent on the project, large open fissures were encountered during excavation of the key trench near the right end of the dam, about 700 feet from the canyon wall. The two largest, near-vertical fissures trend generally east-west and extend more than 100 feet below the bottom of the key trench. Some of the fissures are lined by calcite, and rubble fills others. Several voids, as much as 6 inches wide, were encountered 60 to 85 feet below the ground surface beyond the right end of the dam and grout curtain. The largest fissures were actually enterable caves. One of them was eleven feet wide and a hundred feet long. Another one was nine feet wide in places and 190 feet long. These were not grouted because they were beyond the keyway trench and beyond the area where the Bureau had decided grouting was required. Later, the report of a committee of the House of Representatives which investigated the dam's collapse felt that the discovery of the caves should have been sufficient for the Bureau of Reclamation to doubt its ability to fill them in with grout, but this did not happen and the Bureau continued to insist, even after the dam had failed, that the grouting was appropriate.

The force of the Teton Dam failure destroyed the lower part of the Teton River, washing away riparian zones and reducing the canyon walls. This seriously damaged the stream's ecology, and the native cut throat trout population has been endangered. The force of the water and excessive sediment also damaged stream habitat in the Snake River and some tributaries, at least as far downstream as Fort Hall.

No plans have been made for rebuilding the Teton Dam, but its reconstruction has been brought up on at least one occasion. The original dam is the terraced, which is pyramidal in shaped monolith in the centre of the canyon in the centre of the photograph (Figures 1, 2) obtained from the web link http://www.geol.ucsb.edu/faculty/sylvester/Teton Dam /narrative.pdf. The cut on the right was made after the failure to determine the structure of the embankment. Seed and Ducan (1982) investigated the possibility of sealing the

International Journal of Engineering Research & Technology (IJERT)
ISSN: 2278-0181
Vol. 5 Issue 09, September-2016

upper foundation rock by grouting, an extensive pilot grouting program was conducted on the left abutment. Twenty-three holes were drilled, grouted and pressure-tested. There were significant stakes in several holes. In fact, the amount of grout injected in two holes exceeded the amount estimated for the whole program. By thickening the grout using cement-sand mixes and calcium chloride the leaks tended to seal. However, one persistent leak between 30 and 70 ft. depths could not be filled to refusal.

Subsequently the area where the pilot-grouting program had been performed was core-drilled and water-pressure tested. Most of the test intervals showed little water loss. On the basis of this test program, it was concluded that it would be more economical to remove the top 70 ft. of rock in the abutments above El. 5100 rather than attempt to grout in this zone, leading to the subsequent adoption of a design incorporating a 70 ft. deep key trench to prevent seepage. As per the investigation carried by the U.S Geologic Survey found that the dam area was seismically active and five earthquakes had occurred within 30 miles of the dam site in the last five years., two of them were of significant magnitude. So in the present paper seismic analysis was made to find the safety of the dam.

II. OBJECTIVE OF THE PRESENT STUDY

Teton dam was an earthen dam in Idaho, United States. As it was a finite slope structure which failed during the first filling of the reservoir, therefore, in order to review the causes of failure, slope stability analysis of this dam profile has been done under three ways; those are structural stability, seepage and seismic analysis. The aforesaid analyses are accomplished using several methods that are available for stability analysis of finite slopes. The commonly used methods are: Fellenius Method and Bishop Method.

III EVALUATION OF SLOPE FAILURE ANALYSIS

The evaluation of slope failure analysis was started by Carl Culmann, a German structural engineer in 1866. This method is suitable for very steep slopes. However, in most cases, failure surfaces are curved. Collin observed that the rupture mass slide down a sliding surface, in a definite pattern resembling that of a cycloid. Circular rupture surface was first proposed by Petterson in 1916. Further field investigations by Swedish geotechnical commission justified circular arcs as closed approximation of actual slip surface in homogeneous and isotropic soil conditions. For simple idealized problems, the assumption of a circular failure surface is sufficiently accurate. In 1955,Alan Wilfred Bishop a British Geotechnical Engineer gave a simplified method of analysis which considered the forces on the sides of each slice. This method satisfied the equilibrium requirements of all the slices

IV EMBANKMENT DESIGN

On the basis of the site exploration program, the final design of the embankment, Seed and Ducan (1981) had the configuration shown in figures 3 and 4. A wide core zone of the Aeoliansilt (upstream slope 1 on 1-1/2 and downstream 1on 1) was supported by upstream and downstream shells consisting mainly of sand, gravel and cobbles. In the main section of the dam, the impervious core was extended through the foundations alluvium by means of a 100 ft. deep cut-off trench backfilled with the silt. On the abutments above El.5100, a similar section was adopted but key trenches with a base width of 30 ft. and side slopes of 1/2 on 1 were excavated through the upper 70 ft. of permeable rock and backfilled with the silty material used in the core of the dam.

Figure 1: View of failed Teton dam

Figure 2: Present day (Photo by U.S. Bureau of Reclamation)

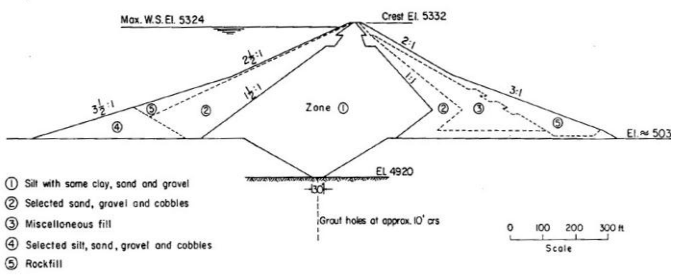

Figure 3: Cross section through of centre portion of embankment founded on Alluvium

International Journal of Engineering Research & Technology (IJERT)
ISSN: 2278-0181
Vol. 5 Issue 09, September-2016

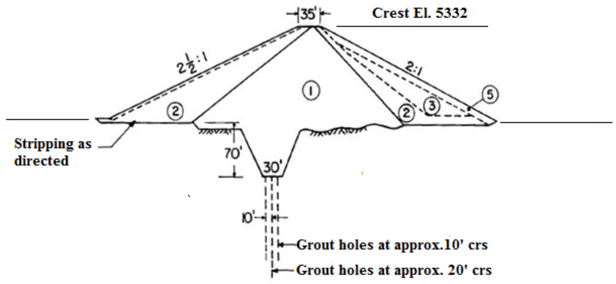

Figure 4: Typical cross section over abutment sections founded on jointed Rhyolite

Downstream of the core was a drainage zone of selected sand and gravels. However, no transition zone was provided between the core and the sand and gravel, nor between the impervious core and the river bed alluvium or between the key trench fill and the rock walls on the downstream side of the key trench. However, the core material in the key trench was placed directly against the rock using special compaction of a 2 ft. wide zone of core material placed at water content above optimum. Compaction of this zone was by hand-operated compactors or rubber-tired equipment.

In addition, the design required that joints encountered in the bottom of the key trench be treated by cleaning and low-pressure grouting. A grout curtain was also installed along the full length of the dam, some holes extending to depths of 300 ft. Grout holes were along a single line with primary holes 10 ft. apart, and split spacing where the primary holes did not indicate a tight curtain. However, lines of barrier holes, intended to prevent excessive flow of grout from the main grout curtain, were installed on 20 ft. centers 10 ft. upstream and downstream of the main grout curtain. It was not required that either the upstream or downstream rows of holes should form tight curtains.

To help prevent seepage, the key trenches and grout curtain were continued well beyond the ends of the embankments, the curtain extending 1000 ft. into the right abutment and 500 ft. into the left abutment.

Thus, as noted in the report of the independent panel, "The final design depended for seepage control almost exclusively on the impervious core, the key trench backfills and on the grout curtain the only downstream defense against cracking in the impervious fill or against concentrated leakage through it was the drainage zone and this did not extend into the key trenches." In the key trenches the silt backfill was in direct contact with the jointed rock.

V. SOIL PROPERTIES AS PER REFERENCE

Unit Weight γ (kip/ft^3)	Friction Angle φ (˚)	Cohesion C (kip/ft^2)	Modulus of Rigidity G (kip/ft^2)
0.117	31^{8}	1.65	8×10^5

VI. GRAPHICAL ANALYSIS

Analysis of slope stability has been carried out in three departments, those are, structural stability, seepage and seismic analysis of entire structure. Structural stability analysis takes into account several factors such as angle of slope, unit weight of soil, cohesion, friction angle etc. Also, apart from these factors the seepage analysis takes into consideration the uplift pressure and pore pressure, which is due to presence of water in soil. Under the seismic analysis response spectrum method has been used to determine the earthquake force upon the structure, since this method is most applicable for this case study.

VII. DISCRETIZATION OF THE SLOPES AND BOUNDARY CONDITIONS

The coordinates of different points of profile along X-axis and Y-axis is digitalized using the "**Digitizeit**" software as shown in figure 5.

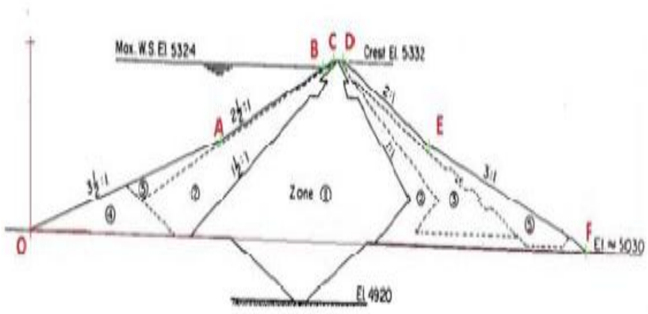

Figure 5: Discretization of the Teton dam.

VIII. STRUCTURAL STABILITY ANALYSIS

Structural stability analysis has been accomplished in compliance with Fellenius method; the critical slip surface is identified thereafter as shown in figures from 6 to 9. The profile of the dam as obtained from the "Digitizeit" is drawn by taking suitable scale of 1 cm = 50 ft both in horizontal and vertical direction

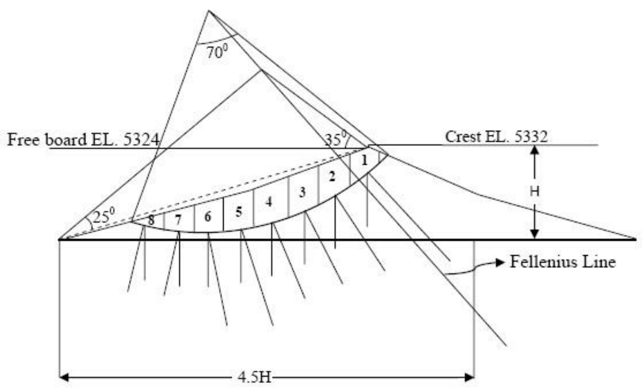

Figure 6: first trial failure surface by Fellenius method.

International Journal of Engineering Research & Technology (IJERT)
ISSN: 2278-0181
Vol. 5 Issue 09, September-2016

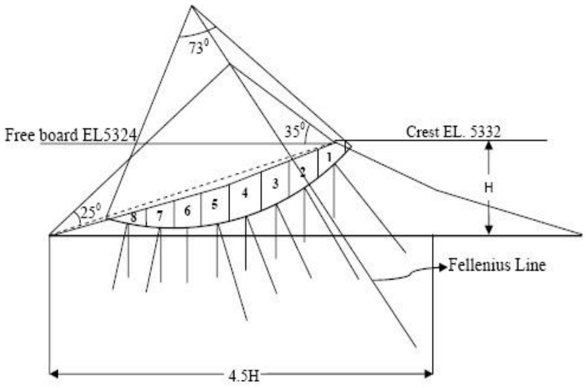

Figure 7: Second trial failure surface by Fellenius method.

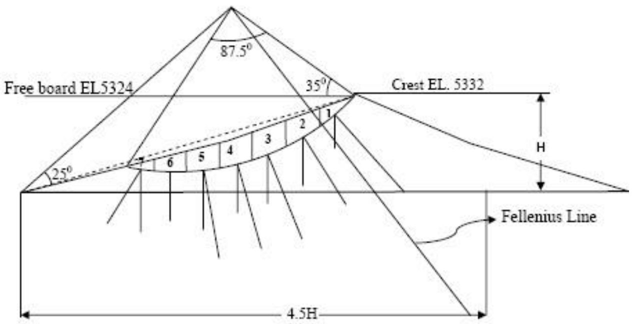

Figure 8: Third trial failure surface by Fellenius method.

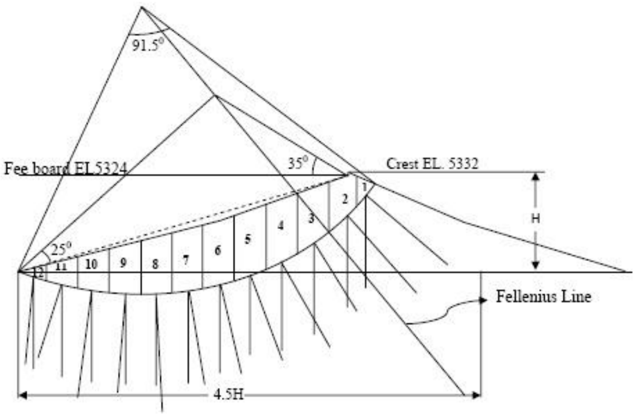

Figure 9: Fourth trial failure surface by Fellenius method.

The Trial slip circles of different radii are drawn using a common centre on the Fellenius line as shown in the figures. The trial failure wedge above the slip surface is divided into a number of vertical slices. These trial failure wedges are of equal width. The angle θ subtended by the arc is measured and thereby arc length L is calculated by simple geometry. Four numbers of trial failure surfaces are considered for determining the factor of safety by using programmed excel sheets.

IX. SEEPAGE ANALYSIS

The Bishop's simplified method is worked out in a programmed excel sheet which is recursive in nature. Hence, an initial trial factor of safety is provided to the excel program.

In this zone flow net diagram is used to determine the pressure head at the centre point of the slip surface of each slice due to seepage. To determine the pressure head at any point, first of all an equipotential line on the flow net passing through that point is selected. If this equipotential line meets the phreatic surface, then the vertical distance between the two points gives the pressure head, because the phreatic surface itself is subjected only to atmospheric pressure. The base parabola portion of the flow net is drawn by taking toe of the Zone-I as focus. Phreatic line is obtained taking upstream and downstream corrections into account. Flow net is drawn taking upstream and downstream slopes of Zone-I, impervious layer of foundation and phreatic line as boundary conditions. A schematic representation of flow net diagram is shown in Fig. 10.

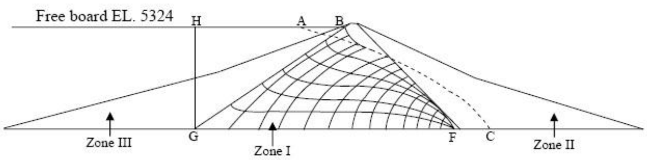

Figure 10: Schematic Presentation of Flow net diagram.

Total weight of failure wedge is calculated as

$$\frac{1}{2} \text{ x Width of slice x [Near side height + Far side height]}$$
$$\text{x Unit weight of dam material} \qquad (1)$$

Pore Pressure Ratio is found out as given below

$$\text{Pore Pressure Ratio } (r_u) = \frac{\text{uplift pressure x width of slice}}{\text{total weight of slice}} \qquad (2)$$

Factor of Safety can be obtained as given below

$$F_s = \frac{1}{\sum W \sin\theta} \sum \left[\frac{cb + W(1 - r_u)\tan\varphi}{M_\theta} \right] \qquad (3)$$

Where,

$$M_\theta = \left(1 + \frac{\tan\theta \tan\varphi}{F}\right)\cos\theta$$

c = Cohesion
b = Width of slice
W = Total weight of slice
r_u = Pore pressure
θ = Failure surface angle
φ = Friction angle

X. SEISMIC ANALYSIS

Fellenius method has proved to be simple and reliable for the seismic analysis of a dam structure. Hence, again, the entire Fellenius method is worked out in a programmed excel sheet.

Width of Slice: Efforts have been made to maintain uniform width of each slice.

The numbering of slices is done taking the crest of dam as reference, so the slices are numbered starting from the crest, proceeding in left direction. As per this fashion, the right hand side of every slice is the near side height. In the same manner mentioned above the left hand side dimension of every slice is the far side height The slope of the Teton dam profile is taken to be constant in order to satisfy the assumption of the Fellenius method. The Failure surface angel is measured geometrically from the triangles beneath the slices. For a particular slice, height of the slice from top is the vertical distance between the crest of the dam profile and its centroid. For individual slices, the horizontal seismic coefficient is measured at their respective centroid level using formula mentioned below;

Horizontal seismic coefficient

$$(K_y) = (2.5 - 1.5 \times \frac{y}{H}) \times \alpha_h \quad (4)$$

Where,

y = Height of slice from top
H = Height of the dam
α_h= Seismic coefficient of dam

Seismic Coefficient of Dam (α_h)

It is obtained using response spectrum method as per IS:1893-1984. This involves the following steps

1. Determining fundamental period of the structure 't' from the following equation:

$$t = 2.61 \times H \times \sqrt{\frac{\gamma}{G}} \quad (5)$$

H = Height of dam
γ= Unit weight of soil
G = Modulus of rigidity of soil
As per the material properties and geometry of the dam, the fundamental time period is obtained as

$$t = 2.61 \times 303 \times \sqrt{\frac{0.117}{800000}} = 0.3s$$

2. Determining S_a/g , for this period considering 10 % damping and using figure 11,as per IS:1893-1984
3. The design value of horizontal seismic coefficient is computed using the following expression

$$\alpha_h = I\, \beta\, F_0\, (S_a/g) \quad (6)$$

I = Importance factor
β = a coefficient that depends on soil foundation system

for dams, I = 3 & β= 1

F_0 = Seismic zone factor computed using table 2.3.
F_0 is taken for zone II
S_a/g = Average response acceleration coefficient

By proper substitution of corresponding numerical values in equation (6), α_h can be found as 0.06.
Flow net diagram as shown in figure 10.is used to determine the pressure heads at the extreme points on the slip surface of slices. Than the uplift force (U) is calculated by using following expression.

$$U = \frac{1}{2} \times (\text{head at near side} + \text{head at far side}) \times \text{unit weight of water} \times \sec\theta \quad (7)$$

(Unit weight of water is taken as 0.062 kip/ft^3)

XII. FACTOR OF SAFETY

Factor of safety of the embankment with the effect of pore water pressure along with seismic condition can be given as

$$F_s = \frac{\Sigma((N - U - K_y x T) \times \tan\varphi) + cL}{T + K_y x\, N} \quad (8)$$

Where,

N = Normal component of total weight of slice (Wcos θ)
T =Tangential component of total weight of slice (Wsin θ)
U = Uplift force
K_y= Seismic coefficient
Φ = Angle of friction
c = Cohesion
L= Length of the slip surface

For the ease of excel programming, the factor safety formula has been fragmented and the calculation is done and presented in Table 2.

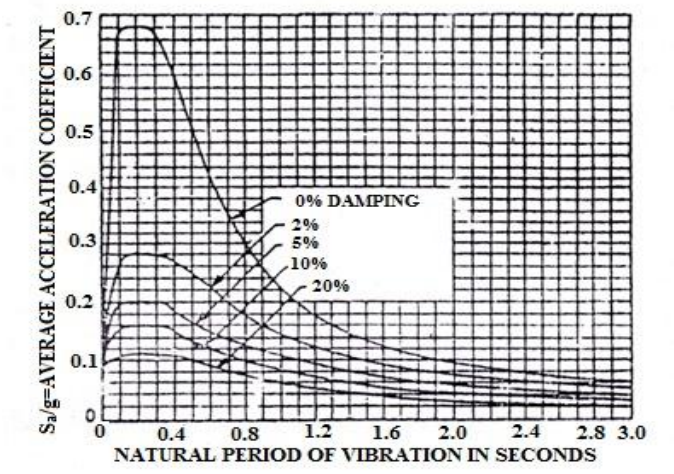

Figure 11: Natural period of vibration against acceleration factor

XII. RESULTS AND DISCUSSIONS

The results in terms of factor of safety determined by various methods of analysis have been discussed and tabulated as follows

Table 2 Various factor of safety as obtained by different methods

Number of Trials	Structural stability analysis	Seepage analysis	Seismic analysis
	(Fellenius method)	(Bishop's simplified method)	(Fellenius method)
1	2.79	1.5	0.92
2	2.88	1.43	0.97
3	3.00	1.65	1.11
4	3.01	2.13	1.35

- Structural analyses carried out for all the three trials yield impressive values of Factor of Safety. It can be inferred that the Teton dam profile was structurally stable. Among all the trials, the fourth one has the greatest Factor of Safety rendering it to be the most stable, structurally.
- From Seepage analyses trial 1 and 2 indicate the failure due to seepage cause. As in trial 1 and 2, more no. of slices is under zone I, where the seepage force is maximum. At the same time trial 4 is stable because there is more no. of slices in it, and moreover greater weight to counteract the seepage force.
- From the seismic analysis of four trials, trial 1, 2, 3 show less stability in terms of their Factor of Safety values, because overall all the slices are at greater elevations. Greater elevation implies greater seismic coefficient thereby less Factor of Safety, but in trial 4 due to less average height of all the slices, there is less seismic coefficient and hence greater stability in terms of Factor of Safety.

XIII. CONCLUSIONS

Designing and constructing earth dams is one of the most challenging tasks a civil engineer can undertake and all of us involved in this type of work can count ourselves fortunate if we escape with minor mishaps as the only result of our activities and decisions. In the present project work, attempts have been made to study the failure of Teton Dam. The dam is analysed by Fellenius and Bishop's simplified method. "The Teton Dam Failure - A Retrospective Review" was silent about the seismic effect of Teton dam. Hence the present paper emphasized the seismic analysis by Fellenius method. Seismic analysis conducted in this project confirms the failure of Teton dam due to earthquake forces, as the critical factor of safety so obtained in seepage analysis i.e. 1.43 is reduced to 0.97 in seismic analysis.

Indeed, it can be said that Teton dam was constructed as specified and failed as a result of inadequate protection of the zone I impervious core material from internal erosion. The most probable physical mode of failure was cracking of zone I material that allowed the initiation of erosion; however, the erosion could have been initiated by piping at the contact of the zone I and the rocks surface.

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