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Parametric Evaluation of the Static Stability Analysis of the Asphaltic Concrete Core Rockfill Dams

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ABSTRACT

The present study examined the effect of the geometric parameters of the Rockfill dams on the 2D static analysis of slope stability. In this regard, several numerical models analyzed for eight heights, three slopes of upstream and downstream, three vertical and skewed core thicknesses and four different impounding time stages namely, "End of Construction", "Full Reservoir", "Half Full Reservoir" and "Rapid Drawdown" on a bedrock foundation. The Morgenstern-Price graphical-limit equilibrium method was selected to study the slopes stability. According to the obtained results for all the models and cases, asphaltic concrete core thickness and its inclination had no significant effect on the calculated safety factors of the slope stability. Maximum and minimum stability safety factors obtained for the "Full Reservoir" and "Rapid Drawdown" time stages, respectively. In addition, in terms of upstream and downstream stability safety factors for different types of impoundment, "End of Construction" and "Half Full Reservoir" time stages were rated the second and third, respectively. Based on the final results, stability safety factor of "Half Full Reservoir" must be considered to ensure adequate safety of the dam stability. For the "Full Reservoir" time stage, the upstream safety factor was bigger than the downstream one, whereas for "Half Full Reservoir", the upstream safety factor was smaller than the downstream one in all heights and slopes.

Keywords

Rockfill Dam, Asphaltic Concrete Core, Slope Stability, Static Analysis, Limit Equilibrium and Half Full Reservoir

1. Introduction

Nowadays, the public perception is that dam construction is a modern industry based on the current advanced technologies. However, historical records and the artifacts remaining from earlier civilizations such as construction of dam on the Nile River 3000 years ago and 2000-year-old dams in the southern Iran show that dam science and water control have a several thousand years' history. Utilization and controlling water as the most important factor for survival and prosperity had received much attention by human communities so that the construction of barriers and dams during the past and present eras is known as the best achievement for proper use and control of water, which had improved because of the technological advances. Nowadays, among different type of the dams, asphaltic concrete core dams have numerous advantages including high implementation speed, small volume of the required materials, good flexibility and less sensitivity to the climatic conditions. Asphaltic concrete core dams are among the

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newest types of dams, therefore various analyses and more recognition of these dams could be of great importance. Undoubtedly, stability analysis and safety assessment are among the most important items of interest in studies on the asphaltic concrete core rockfill dam and other types of embankment dams.

Valstad et al. (1991) studied the Storvatn Dam in Norway. Using Newmark findings, they showed that the thin cores of the asphaltic concrete experienced cracking in the part near the dam crest in severe seismic loads.

Hoeg (1993) studied the Storvatn Dam and observed that if the slopes are very steep, a large relative shear strain will occur in the upper part of the dam core. He also pointed out that, rockfill dams with asphaltic concrete core have generally a good resistance against the seismic phenomena.

Gurdil (1999) studied Kopru Dam in Turkey based on the equivalent linear method. He concluded that it is possible to develop some cracks near the dam crest level, but the self-maintaining property of the asphaltic concrete core will overcome this problem.

Mahinroosta and Ghanooni (2002) carried out nonlinear dynamic analysis on a 115 m high rockfill dam with an asphaltic concrete core. They concluded that small tensile stresses develop in the upper part of the dam core so that these stresses will be lower than the tensile strength of the materials of asphaltic concrete.

Salemi and Baziar (2003) studied the Meyjaran Dam in Iran with a height of 60 m using numerical analysis and experimental tests. They concluded that the behavior of the asphaltic concrete core in severe seismic loads is acceptable and in such cases, the dam will enjoy relative safety.

Feizi-Khankandi et al. (2004) carried out 2D nonlinear analysis on a rockfill dam with an asphaltic concrete core with a height of 125 m. They concluded that the dynamic behavior of the dam is acceptable before, during and after earthquake and the core deformation depends on the shell displacement.

A. Akhtarpour and A. Khodai (2009) studied the nonlinear numerical analysis of the Shour River Dam, Iran with a height of 85 m using finite difference method and employing a hyperbolic model to study the behavior of the asphaltic concrete core in the seismic conditions in the case of water impounding and construction. The results revealed that in these conditions, some cracks occur in the upper part of the asphaltic concrete core. The maximum vertical deformation occurs near the crest and upper part of the upstream slope, but it has no significant impact on the reservoir in the normal operation conditions of the dam.

M. Salehi and P. Safapour (2003) studied stability of the embankment and rockfill dams with asphaltic concrete core using the limit equilibrium method utilizing Stabl software in four cases of construction and water impounding. They concluded that the dam structure has a proper behavior under different conditions of the static loading.

2. Materials and Methods

The purpose of the present study was to evaluate the static stability of the rockfill and embankment dams with asphaltic concrete core considering various heights in different conditions including construction and water impounding. GeoStudio software, which was used in this study, is one of the first professional softwares for the slope stability analysis in the geotechnical engineering. It has great abilities including processing the input data, estimating output and creating numerous graphs from various variables. Nowadays, this software is used by thousands of professionals in different fields of research, teaching and administration (Geo-Slope International Ltd., 2010).

2.1. Height

This research needed studying constructed dams and their geometrical conditions to use proper models with more complete features. This article has tried to select the height such that covers an acceptable range in dam modeling. Therefore, the dams were divided into 8 height classes including 50 m, 60 m, 70 m, 80 m, 90 m, 110 m, 125 m and 170 m according to Table 1.

Table 1. Selected dams for modeling and analysis

Country	Name of Dam	Real Height (m)	Height Class
United Kingdom	Megget	56	50
Norway	Berdalsvatn	65	60
Germany	Schmalwasser	76	70
Macedonia	Zletovica	85	80
Hong Kong	High Island West	95	90
Canada	La Romaine 2	110	110
Norway	Storgolmvatn	125	125
China	Quxue	170	170

2.2. Freeboard depth and crest width

The freeboard depth was considered for all the models identically by assuming a dam average height of 100 meters. This value has been determined and calculated according to the Iranian National Committee on Large Dams criteria as 5 m for all the models. Dam crests' characteristics were according to Table 2.

Table 2. Dam crest widths and height classes

Height Class	50	60	70	80	90	110	125	170
Crest Width (m)	9.5	7.0	8.0	8.50	9.50	6.0	7.0	9.5

2.3. Foundation type and depth

In terms of implementation, Rockfill Dams' foundations are classified in two main groups of bedrock and loose ground. In this study, all models were simulated considering the rigid supports on a bedrock foundation. In addition, the two ends of bedrock foundation were cut in modeling the dams. Therefore, both side regions of the dams' foundations were modeled in infinite mode (Geo-Slope International Ltd., 2010).

Bedrock foundation sealing was done by regular injections and typically injected curtain as deep as a third of the dam's height or at least 6 to 10 meters below the stone surface (Kjaernsli et al., 1991). Therefore, foundation depth of approximately a third of the dam's height was selected in this study.

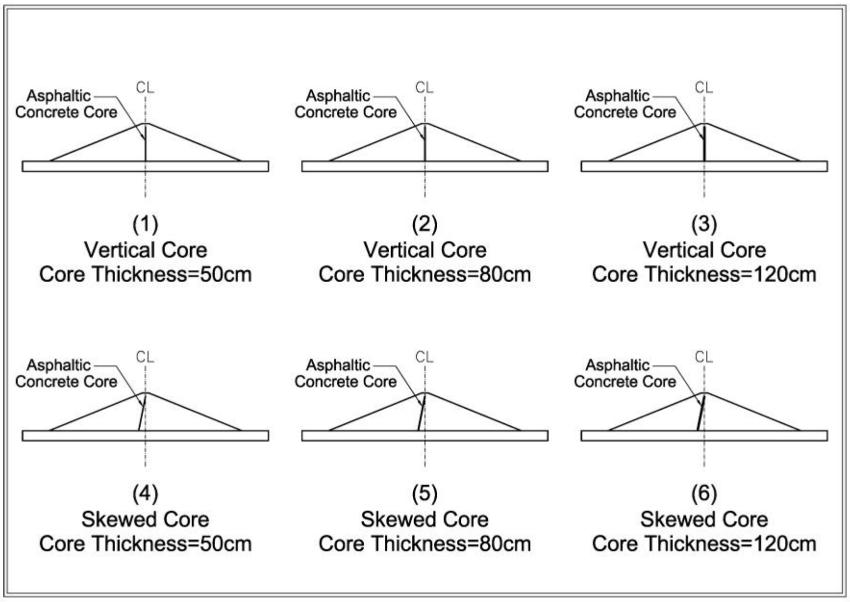

Fig. 1. Asphaltic concrete core types used in modeling based on thicknesses and positions

2.4. Upstream and downstream slopes

To simplify the modeling, upstream and downstream slopes were considered to be identical in all dam models. In addition, details of the zoning materials and core to foundation connections were eliminated from the models.

Each class of height was classified into three different cases of upstream and downstream slopes as below:

- Slope Case 1: (V:H) 1.0:2.5
- Slope Case 2: (V:H) 1.0:2.0
- Slope Case 3: (V:H) 1.0:1.5

2.5. Asphaltic concrete core forms

The core of a rockfill dam can be constructed in one of the following forms depending on the environmental conditions or construction limitations:

- Vertical core in the center of the dam
- Skewed core towards the upstream slope

Cases described in the previous section were considered versus six different conditions depending on the thickness and placement of the asphaltic concrete core according to Fig. 1.

In the case of dams with heights more than 30 m, the core thickness is determined based on the height and usually is chosen in the range of 60 to 100 cm. However, it is not recommended to change the core thickness from top to bottom. Since the symmetric deformation usually occurs in the center of the dam, the core is implemented at this section. The core is usually built vertically in dams with less than 60 m height. However, in the case of taller dams, the core is inclined somewhat toward tail water in the upper part to reduce detachment of mirage's bank from core in the crest region (Creegan et al., 1996).

Although the inclined core leads to good transfer of water hydrostatic loads toward the downstream embankment, but it includes additional costs. Moreover, constructing a vertical core provides possible maintaining of the core by a simpler injection. The thicknesses of 50 cm, 80 cm and 120 cm were used in the vertical (V:H) 1:0 and skewed (V:H) 1:0.2 (Hoeg, k., 1993) forms for the asphaltic concrete core.

2.6. Construction and water impounding time stages

The six conditions described in section 2.5 were studied for the following four time stages of impoundment:

- End of Construction
- Steady State with Full Reservoir
- Steady State with Half Full Reservoir
- Rapid Drawdown

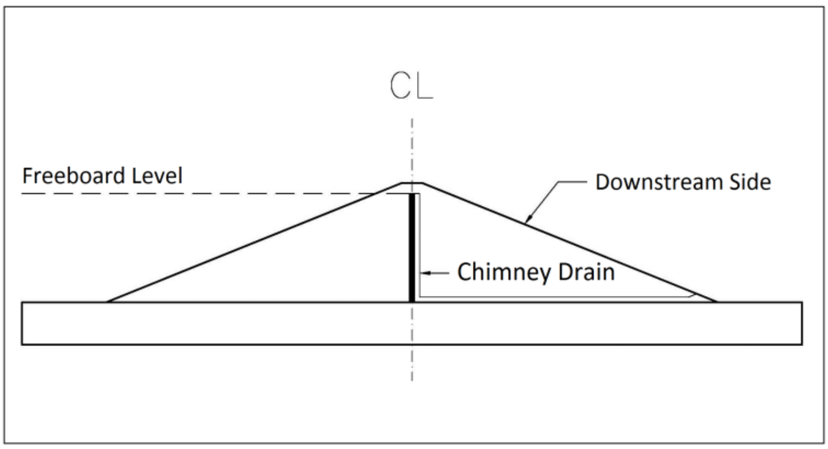

Fig. 2. Chimney drain within an asphaltic concrete core dam

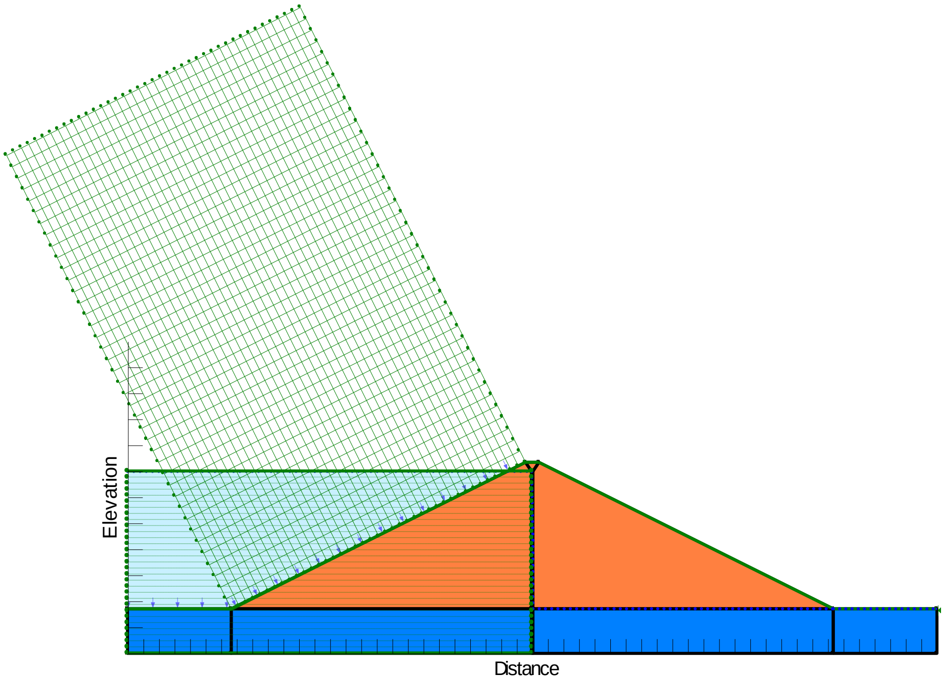

Fig. 3. Upstream slope - 80 m height class - full reservoir

For "Full Reservoir" time stage, the reservoir is full except 5 m freeboard. For "Half Full Reservoir" time stage, water level in the reservoir is equal to half of the water surface in "Full Reservoir" time stage. In addition, for "End of Construction" time stage, dam construction is finished and there is no water in the upstream and downstream. Finally, for impounding time stage of "Rapid Drawdown", the reservoir water level is equal to one-eighth of the water surface in "Full Reservoir" time stage.

A vertical chimney drain that extends completely from one abutment to the other is a common element for most high and significant hazard embankment dams in different probable water storage levels (FEMA484, 2005). Chimney drain is a very effective element for seepage control, increasing the downstream slope stability and reducing pore water pressure during construction and water impounding time stages (Ziaie Moayed et al., 2012). In this study, chimney drain was selected for all the models according to Fig. 2.

3. Numerical modeling

3.1. Domain of circle's center and radius

To achieve more accurate results for slip circles and correct calculation of the stability safety factors, it is necessary to select proper position of the Grid and Radius lines for the stability analysis. Therefore, various domain of center and radius of circles (Grid of centers: 10 to 100 and Radius: 10 to 30) were studied.

After comparison and validation of the obtained results, number of grid and radius lines were selected. For example, Fig. 3 shows the upstream Grid and Radius lines for a model in 80 m height class and "Full Reservoir" time stage. It should be noted that the minimum required safety factor of 1.5 was considered.

3.2. Analysis assumptions

To determine and propose the geo-mechanical parameters of the core, shell and foundation materials, recommended values by previous researchers in similar studies were used for Mohr-Coulomb model according to Table 3. The Mohr-Coulomb model is the most common model for shear strength of geotechnical materials which is expressed as:

$$\tau = c + \sigma_n \times \tan\phi$$

Table 3. Material properties

Type of Material	Unit Weight	Cohesion Coefficient	Internal Friction Angle
	γ	C	Φ
	KN/m3	KN/m2	°
Asphaltic Concrete Core	24	360	28
Shell	22	40	43
Bedrock	22	350	35

In this study, Morgenstern-Price analysis method was selected, which is a graphical-limit equilibrium method. Formulation of this method satisfies equilibrium of the horizontal forces and moments acting on the individual blocks created by dividing the soil above the slip surface and allows for a range of inter-slice shear-normal force conditions (Sinha, B. N., 2008). Both calculated quantities will converge into a final stability safety factor.

Proposed empirical equation for the interslice force relation is expressed as $X = E \times \lambda \times f(x)$, where X is the interslice shear force, E is interslice normal force, λ is the percentage of function used and *f(x)* is interslice force function representing the value of the function at the location of a particular slice by forms of half-sine function according to Fig. 4.

To obtain safety factor, λ should be specified in all the cases. The best result is obtained when λ is chosen such that forces equilibrium is equal to moments equilibri-um.

For example, Fig. 5 shows the critical slip circle in the downstream slope and calculated stability safety factor for a model with 80 m height and "Half Full Reservoir" time stage.

3.3. Stability analysis results

As mentioned before, static analysis was carried out based on Morgenstern-Price method after allocating the location and specifications of the zoning materials in all the models.

Stability safety factors of the upstream and downstream slopes were divided to three different cases of slopes and four time stages of impoundment, as can be seen in Tables 4, 5 and 6. Figures 6 to 11 summarize and present the stability safety factors versus eight different height classes of dams.

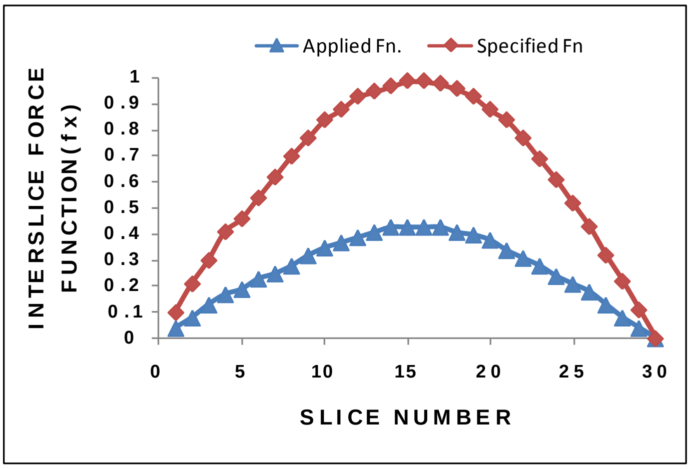

Fig. 4. Half-sine interslice force function (Geo-Slope International Ltd., 2010)

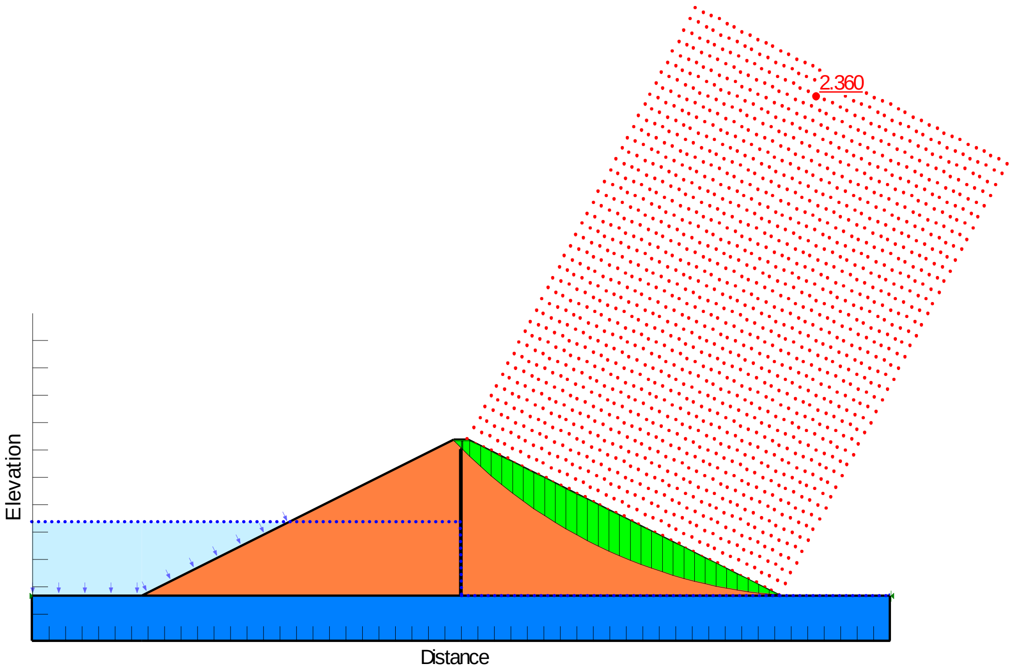

Fig. 5. Downstream critical slip circle - Morgenstern-price analysis

Table 4. Upstream and downstream safety factors for case 1 versus different classes of heights

Dam Height	Safety Factor - Slope : 1:2.5							
	Steady State-Full Reservoir		Half Full Reservoir		End of Construction		Rapid Drawdown	
	Upstream	Downstream	Upstream	Downstream	Upstream	Downstream	Upstream	Downstream
50	3.266	3.032	2.835	3.032	2.997	2.998	1.768	3.032
60	3.198	2.969	2.774	2.969	2.938	2.938	1.705	2.969
70	3.113	2.900	2.708	2.900	2.882	2.874	1.646	2.900
80	3.070	2.871	2.673	2.871	2.843	2.846	1.621	2.871
90	3.010	2.828	2.633	2.828	2.801	2.805	1.572	2.828
110	2.955	2.779	2.596	2.779	2.763	2.758	1.533	2.779
125	2.904	2.744	2.558	2.744	2.722	2.726	1.496	2.744
170	2.798	2.668	2.487	2.667	2.665	2.652	1.429	2.667

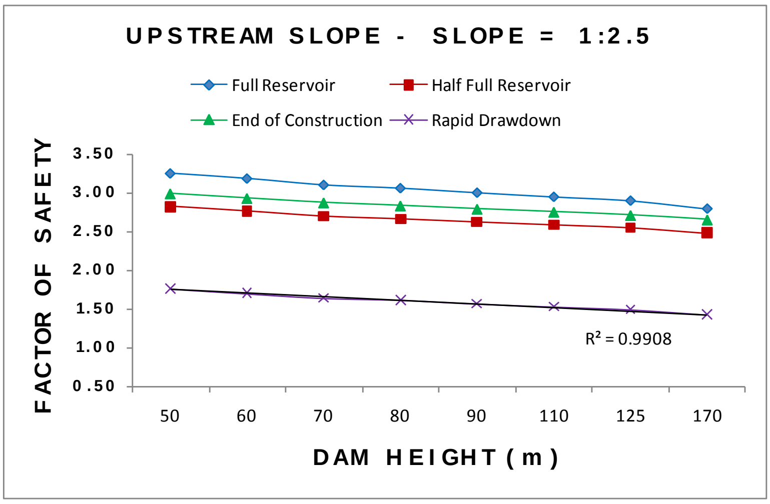

Fig. 6. Dam height - upstream stability safety factor - Case 1

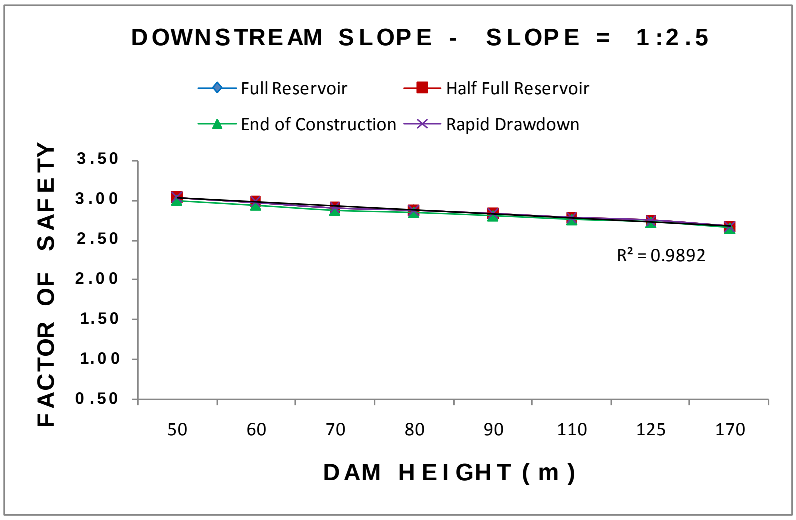

Fig. 7. Dam height - downstream stability safety factor - Case 1

Table 5. Upstream and downstream safety factors for case 2 versus different classes of heights

Dam Height	Safety Factor - Slope : 1:2.0							
	Steady State-Full Reservoir		Half Full Reservoir		End of Construction		Rapid Drawdown	
	Upstream	Downstream	Upstream	Downstream	Upstream	Downstream	Upstream	Downstream
50	2.749	2.525	2.398	2.525	2.495	2.495	1.443	2.525
60	2.692	2.461	2.341	2.467	2.439	2.439	1.383	2.461
70	2.597	2.403	2.280	2.403	2.378	2.379	1.327	2.403
80	2.558	2.360	2.245	2.360	2.338	2.337	1.291	2.360
90	2.506	2.333	2.208	2.333	2.315	2.312	1.258	2.333
110	2.456	2.294	2.171	2.294	2.275	2.275	1.220	2.294
125	2.408	2.247	2.134	2.247	2.238	2.229	1.186	2.247
170	2.309	2.178	2.065	2.178	2.177	2.164	1.120	2.178

Figures 7, 9 and 11 show very close results for the calculated stability safety factors in the downstream slope for each slope case, and different impounding time stages due to the same assumed water level in the downstream slope.

During different types of impoundment in the reservoir, identical trends were obtained for the upstream slope, but they varied in the domain of safety factor values according to Figs. 6, 8 and 10. For "Rapid Drawdown" time stage, safety factors were in the minimum range due to the sudden drop of the reservoir water level, therefore its results varied from the other impounding results.

Results show a decrease in the upstream and downstream stability safety factors by increasing the height at a constant slope for all the impounding time stages.

The results also showed an increase of the stability safety factor by increasing the upstream and downstream slopes for a given height for all types of the impoundment.

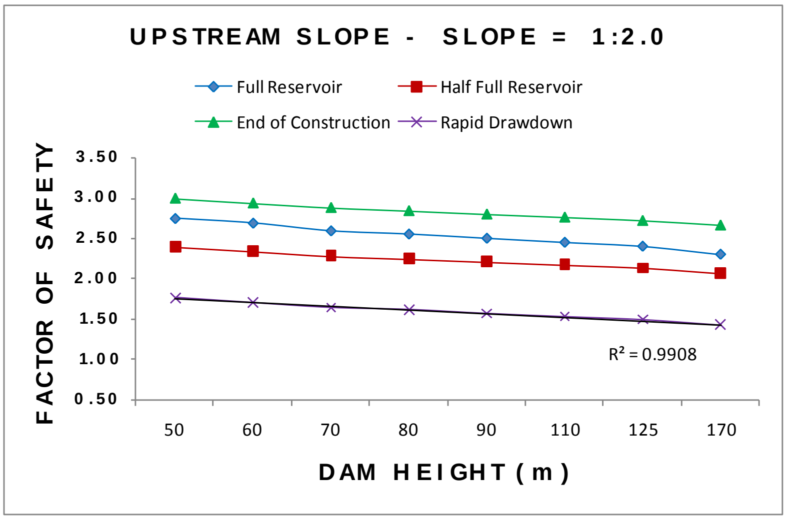

Fig. 8. Dam height - upstream stability safety factor - Case 2

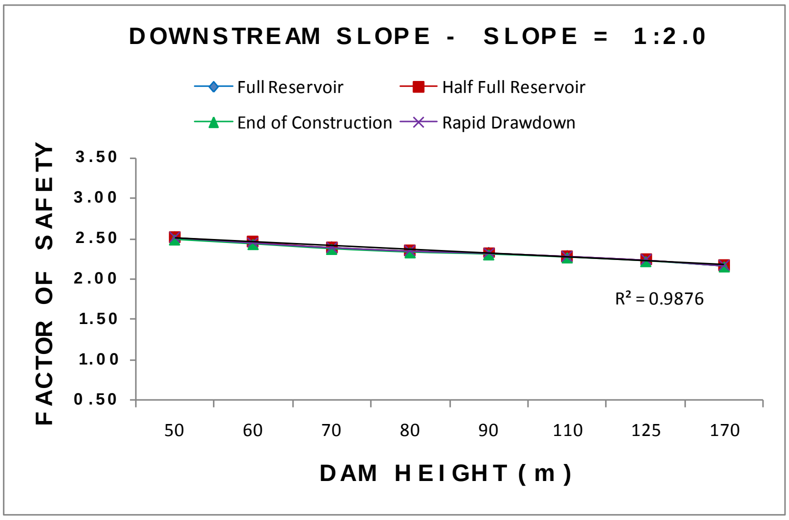

Fig. 9. Dam height - downstream stability safety factor - Case 2

Table 6. Upstream and downstream safety factors for case 3 versus different classes of heights

Dam Height	Safety Factor - Slope : 1:1.5							
	Steady State-Full Reservoir		Half Full Reservoir		End of Construction		Rapid Drawdown	
	Upstream	Downstream	Upstream	Downstream	Upstream	Downstream	Upstream	Downstream
50	2.229	2.017	1.954	2.017	1.986	1.986	1.098	2.017
60	2.178	1.956	1.895	1.956	1.931	1.931	1.041	1.956
70	2.097	1.900	1.842	1.900	1.876	1.876	0.988	1.900
80	2.045	1.867	1.803	1.867	1.841	1.844	0.954	1.867
90	2.003	1.832	1.774	1.832	1.815	1.810	0.922	1.832
110	1.955	1.799	1.735	1.799	1.779	1.780	0.886	1.799
125	1.912	2.247	1.699	2.247	1.751	1.753	0.854	1.770
170	1.818	1.694	1.632	1.694	1.691	1.681	0.798	1.694

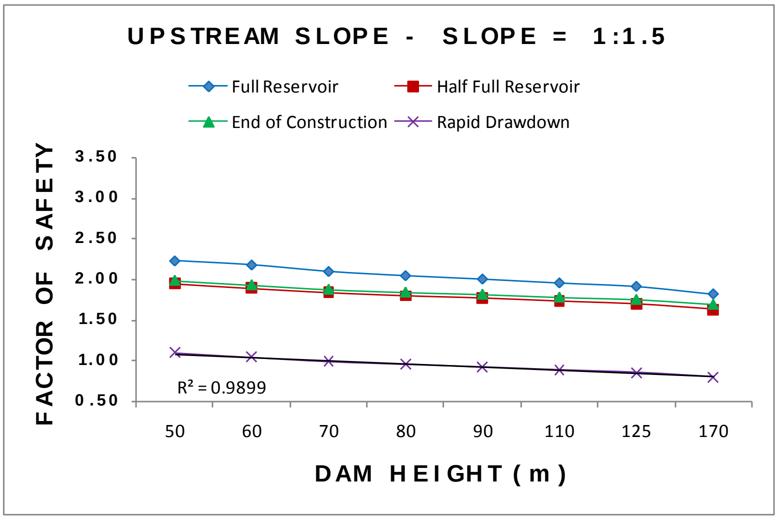

Fig. 10. Dam height - upstream stability safety factor - Case 3

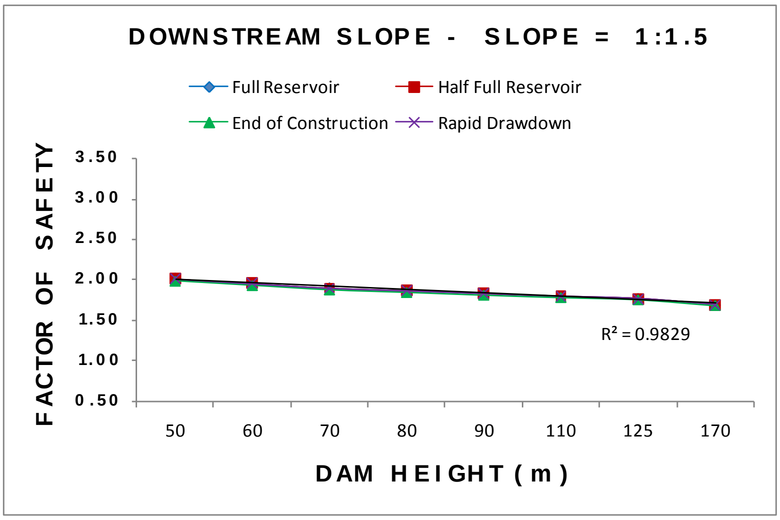

Fig. 11. Dam height - downstream stability safety factor - Case 3

4. Conclusions

Static stability analysis of the rockfill dam models with asphaltic concrete core, considering different impounding time stages (Full Reservoir, Half Full Reservoir, End of Construction and Rapid Drawdown), different heights and slopes showed that:

The maximum and minimum stability safety factors are related to "Full Reservoir" and "Rapid Drawdown" time stages, respectively.

In terms of the upstream and downstream stability safety factors for different types of impoundments, "End of Construction" and

"Half Full Reservoir" time stages will be second and third ones, respectively.

According to the stability analysis results of the "Half Full Reservoir" time stage and a remarkable probability of happening this case for many dams, it is recommended to perform complement-ary studies and measures to ensure adequate safety of the dam stability.

For the "Full Reservoir" time stage, the upstream stability safety factor is bigger than the downstream one for all the heights and slopes, which indicates the criticality of the downstream slope and need for concentration on its stability.

The calculated stability safety factors for different models with vertical and skewed asphaltic concrete cores in different impounding time stages, heights and slopes showed that the thickness of asphaltic concrete core and its inclination has no significant effect on the slope stability safety factors. It is attributed to the position of the critical slip circle, which did not pass through the asphaltic concrete core, as can be seen in Fig. 12.

For the "Rapid Drawdown" time stage, the upstream stability safety factor is less than the downstream one for all the models.

By considering different heights and slopes, for the "Half Full Reservoir" time stage, the upstream stability safety factor is less than downstream, which indicates the criticality of the upstream slope stability for this case.

For the "End of Construction" time stage, the stability safety factor is approximately identical for both the upstream and downstream slopes considering different heights and slopes.

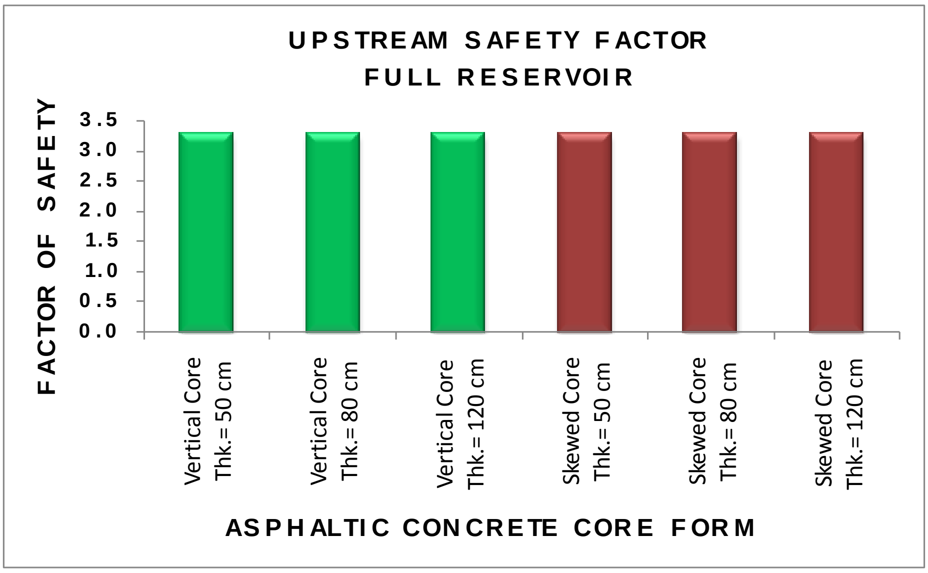

Fig. 12. Upstream safety factor - core thickness and inclination

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