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Koman Concrete Face Rockfill Dam Updating the Static and Seismic Evaluations

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ABSTRACT

The static and seismic safety evaluation of the 115 m high Koman concrete face rockfill dam, located at the Drin river in Albania has been checked according to the current state-of-practice for the seismic safety evaluation of large embankment dams. For the dynamic analyses with the equivalent linear method a two-dimensional model of the highest dam section was used. The static analysis was carried out by a Mohr-Coulomb elasto-plastic material model of the rockfill. The safety of the dam was checked for the safety evaluation earthquake (SEE) with a peak ground acceleration of the horizontal component of 0.45 g. Spectrum-compatible artificially generated accelerograms were used determined based on a site-specific seismic hazard analysis. The peak absolute horizontal crest acceleration due to the SEE excitation is about 0.78 g for average material properties, and about 1.16 g for the most unfavorable material properties. The maximum crest settlement resulting from the sliding displacements plus an additional settlement due to the vibration-induced densification of the dam body are calculated as 0.98 m, under the most unfavorable conditions.

INTRODUCTION

The Koman concrete face rockfill dam (CFRD) is located at the Drin river in the North of Albania. It was constructed from 1980 to 1985 for hydropower generation, and when it was completed it was the largest CFRD in Europe. At the time when the dam was designed most dams were designed against earthquakes using the pseudo-static method of analysis with a seismic coefficient of 0.1, a method, which is considered as obsolete today. Moreover, reports on the seismic analysis and design of the dam have also got lost due to various reasons. Therefore, it was decided to check the earthquake safety of Koman dam using current seismic design criteria and methods of dynamic analysis of embankment dams. The layout of the Koman hydropower complex and the dam upstream view are shown in Figures 1 and 2, respectively. The main features of the dam are as follows:

Dam type:	concrete face rockfill dam
Dam height:	115 m
Upstream slope gradient:	1.0V (vertical):1.6H
Downstream slope gradient:	1.0V:2.0H
Dam body volume:	5 million m^3
Normal water level:	175.8 m a.s.l.
Crest length:	250 m
Crest width:	10 m
Power generation capacity:	600 MW

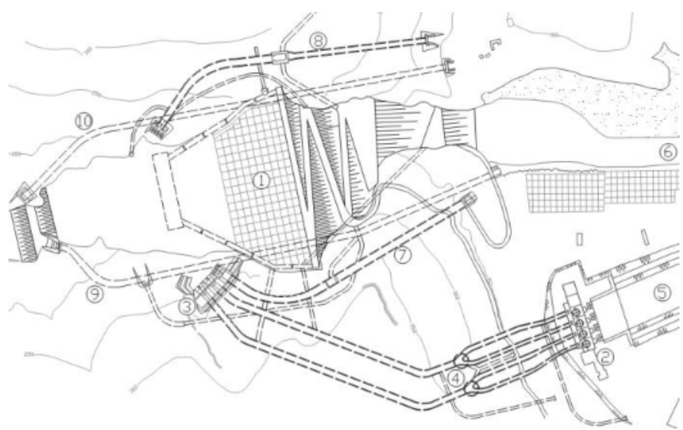


Fig. 1: Layout of Koman HPP – (1) Concrete faced rockfill dam, (2) powerhouse, (3) power waterway intake (4) surge chambers, (5) tailwater channel, (6) Drin River, (7) tunnel spillway No. 4, (8) tunnel spillway No. 3, (9) diversion tunnel No. 2, (10) diversion tunnel No. 1

The dam analyses consisted of the following steps:

1. Static slope stability analysis of critical slopes before and after the earthquake.

2. Static analysis to estimate dam deformations during incremental construction and impounding, and to compute static stresses required to determine the maximum dynamic shear modulus of different finite elements of the dam.
3. Selection of accelerograms compatible with the acceleration response spectrum of the safety evaluation earthquake (SEE).
4. Earthquake response analysis using the equivalent linear method to compute accelerations and dynamic stresses in the dam.
5. Selection of potential sliding masses and calculation of their yield accelerations (Note: The yield acceleration is the pseudo-static horizontal earthquake acceleration for which the factor of safety against sliding failure is equal to 1.0).
6. Calculation of permanent earthquake-induced displacements of potential sliding masses based on Newmark's sliding block concept.
7. Estimate of settlement due to vibration-induced densification of dam materials during earthquake shaking.
8. Determination of loss of freeboard due to sliding movement of critical slopes and settlements resulting from vibration-induced densification.

Fig. 2: Koman dam – upstream view

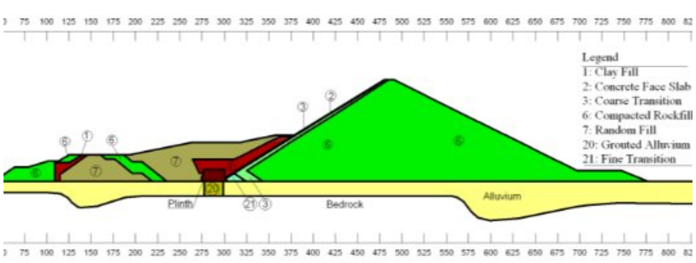


Fig. 3: Highest cross-section of Koman CFRD with different material zones

The analysis of the Koman dam was carried out using a two-dimensional (2D) finite element (FE) model of the maximum cross-section shown in Figure 3. Figure 4 also shows the dam FE model together with the different construction stages of the dam body.

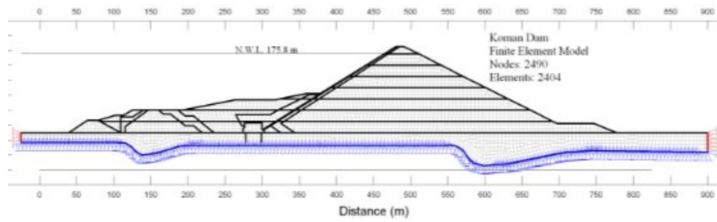


Fig. 4: Finite element model of highest section of Kaman dam

STATIC DEFORMATION AND STRESS ANALYSES

Material Properties

The static analysis was carried out using the Mohr-Coulomb elasto-plastic model of the gravel and the alluvial layer at the base of the dam. The static analysis was carried out by the computer program SIGMA/W (GEO-SLOPE International Ltd., 2007).

The material parameters listed in Table 1, were selected based on engineering judgment by different dam experts and data of similar projects, as the corresponding information from the design of the dam was no longer available.

Table 1: Properties of different material zones for static deformation and stress analyses (zones shown in Fig. 3)

Description	ν	E (MPa)	γ (kN/m³)	c' (kPa)	ϕ_s' (°)	Hydraulic Conductivity (m/s)
Clay fill	0.40	12	18	0	24	1E-8
Fine transition	0.30	50	20	0	38	1E-3
Coarse transition	0.30	45	20	0	40	1E-4
Rockfill	0.30	45	21	0	42	5E-4
Random fill	0.30	30	20	0	36	1E-6
Plinth and gallery block (concrete)	0.20	10000	24	-	-	1E-10
Coarse alluvium	0.30	35	21	0	32	1E-5
Grouted alluvium	0.25	1000	22	-	-	1E-8

Static Loads

The following static loads were analysed:

Gravity load. The gravity load was applied by simulating the dam construction in 13 steps. First, the dam body was built up in the FE model in 8 horizontal layers up to the level of the crest. Then, the concrete face was put in place. At the end, the upstream fine backfill material was built in 4 horizontal layers.

Water load. The water load was applied as hydrostatic pressure acting on the concrete face. The water level was raised to the normal operation water level.

The displacements due to the water load were obtained by subtracting the displacements due to gravity load from the displacements due to the combination of gravity and water loads.

Analysis Results

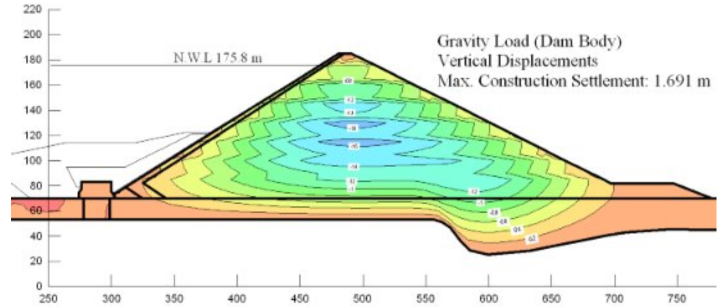
Some of the main static analysis results are presented in Figures 5 and 6.

The main results of the static deformation analyses are as follows (Figures 5 and 6):

- Maximum settlement due to gravity load (incremental construction): 1.69 m
- Maximum horizontal displacement due to dam construction: 0.56 m
- Maximum vertical and horizontal displacements of the face slab due to the water load during the first impoundment of the reservoir: 0.72 m and 0.55 m, respectively
- Maximum displacement perpendicular to the face slab): 0.90 m

The contour plots of the vertical and horizontal stresses for the static load combination - gravity load plus water load - are also given in Figure 6. The normal and tangential displacements along the concrete face slab due to the hydrostatic pressure exerted by the reservoir at the normal operating water level are plotted in Figure 7.

It should be noted that as the sealing element of the dam is located at the upstream face, the reservoir water is supposed not to enter the dam body, and the reservoir hydrostatic pressure is exerted on the concrete face, perpendicular to the face. Therefore, a relatively considerable perpendicular displacement (sagging) is expected at this condition, as calculated by the analyses.



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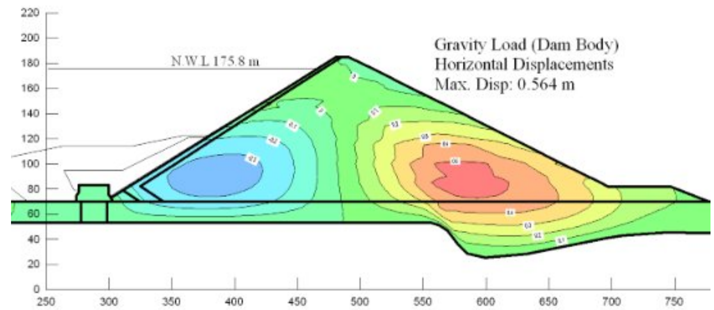

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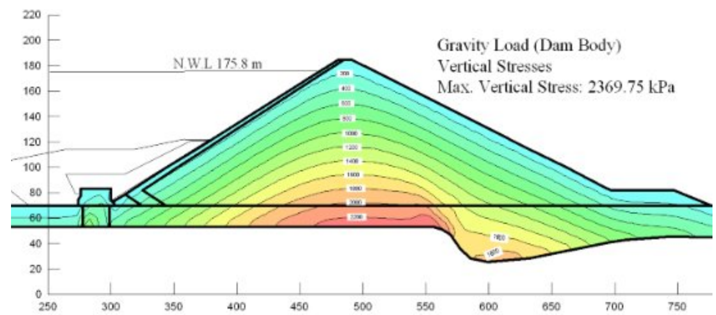

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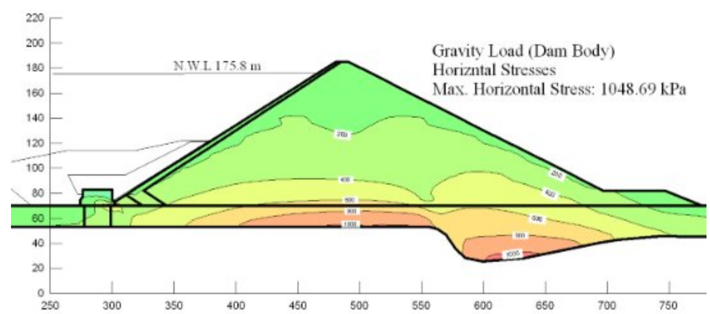
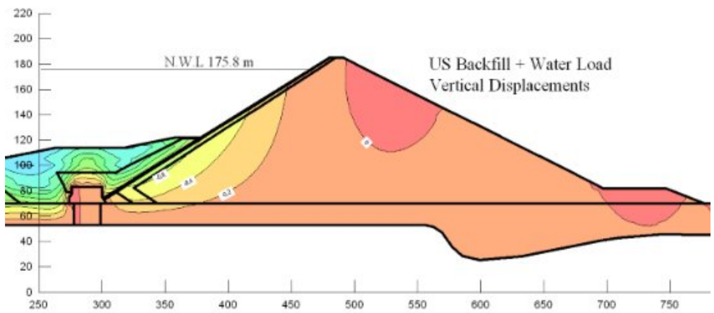


(d)

Fig. 5: Contour plots of (a) vertical and (b) horizontal displacements, and (c) vertical and (d) horizontal stresses due to incremental construction of the dam



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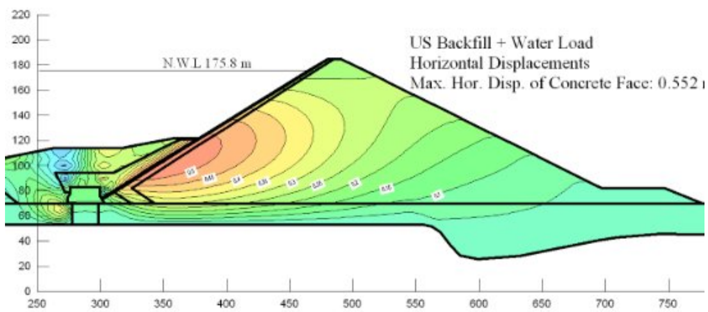

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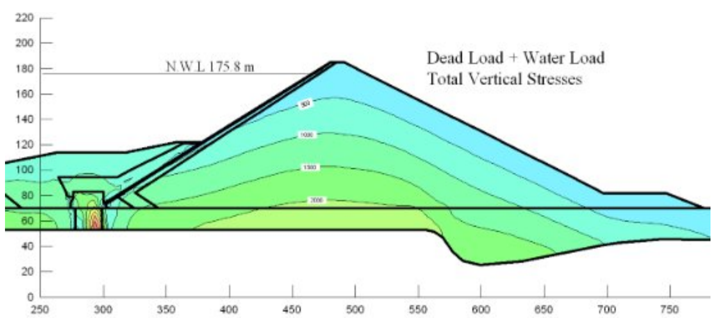

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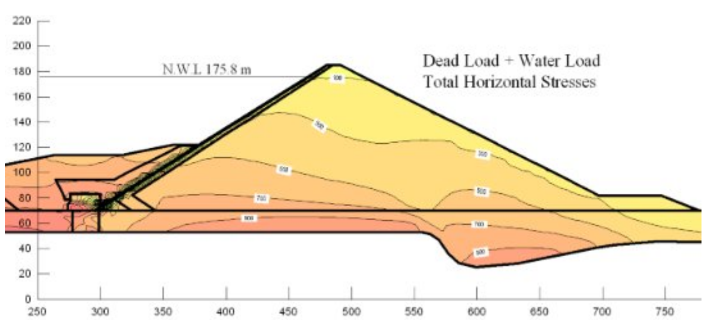


(d)

Fig. 6: Contour plots of (a) vertical and (b) horizontal displacements, and (c) vertical and (d) horizontal stresses due to gravity and water loads

DYNAMIC MATERIAL PROPERTIES

The maximum dynamic shear modulus G_{max} for coarse grain materials can be expressed as:

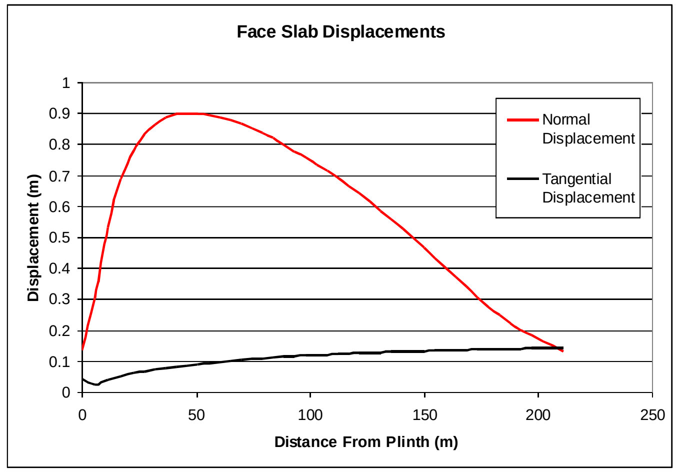


Fig. 7: Normal and tangential displacements of face slab at maximum cross-section due to hydrostatic pressure exerted by the reservoir at normal operating water level (175.8 m asl).

$$G_{max} = 220\ k_{2max}\ {\sigma'_m}^{0.5} \quad (1)$$

where k_{2max} is a material coefficient that depends primarily on the void ratio e and σ'_m is the mean effective static stress. Conventionally, the shear modulus for a cyclic shear strain amplitude of 0.0001 % is designated as G_{max}, since the dynamic shear modulus is practically constant for strain amplitudes below this level. Values of k_{2max} for different gravel soils are illustrated in Figure 8.

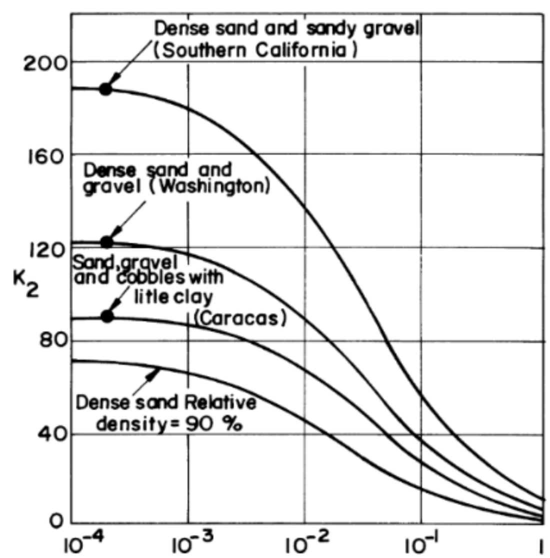


Fig. 8: Values of k_{2max} for gravelly soils (after Seed and Idriss, 1970)

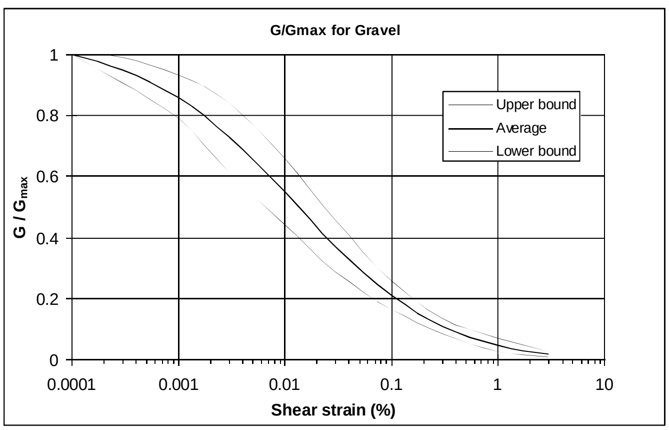
The material properties used for the coarse grained materials are given in Table 2.

The plinth and gallery block, the concrete face slab, and the grouted alluvium confined between diaphragm walls (under the plinth) were assumed to be linear-elastic materials.

The dynamic shear modulus and the damping ratio of gravelly soils were assumed to vary with the cyclic shear strain amplitude as depicted in Figure 9. In these figures upper and lower bound and average shear strain-dependent material properties are shown.

Table 2: Dynamic material properties

Description	k_{2max}			Poisson ratio	E (MPa)	Material model
	Min	Ave	Max			
Concrete face	-	-	-	0.20	5000	Linear
Plinth and gallery block	-	-	-	0.20	5000	Linear
Grouted alluvium	-	-	-	0.25	1000	Linear
Fine transition	52	58	68	0.30	-	Nonlinear
Coarse transition and rockfill	120	140	160	0.30	-	Nonlinear
Random fill	50	65	80	0.30	-	Nonlinear
Alluvium	60	80	100	0.30	-	Nonlinear



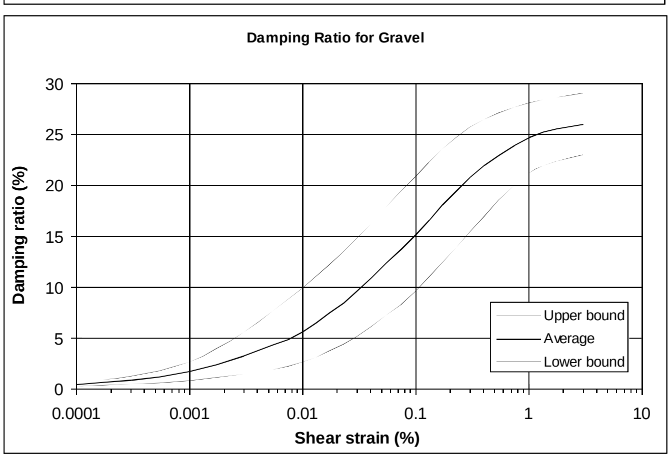


Fig. 9: Shear strain-dependent shear modulus (top) and damping ratio (bottom) for gravels

<u>Analysis Cases</u>

Four different cases with different dynamic material properties were selected, in order to estimate the material-related uncertainties in the dynamic response of the dam. The different cases are listed in Table 3..

Table 3: Earthquake analysis cases and main results

Case	G_{max} and G/G_{max} vs. shear strain curve (Fig. 4)	Damping ratio vs. shear strain curve (Fig. 4)	Peak absolute horizontal crest acceleration (g)
A	Average	Average	0.78 g
B	Average	Lower bound	0.96 g
C	Upper bound	Average	0.91 g
D	Upper bound	Lower bound	1.16 g

EARTHQUAKE RESPONSE ANALYSIS

<u>Earthquake Load</u>

The earthquake acceleration time histories used in earthquake response analyses have been obtained using the computer program SIMQKE. The artificial accelerograms have been scaled and adjusted to the site-specific seismic parameters. The horizontal and vertical peak ground accelerations of the SEE were taken as 0.45 g and 0.30 g, respectively.

<u>Equivalent Linear Analysis and Dynamic Model</u>

The dynamic response of the maximum dam section subjected to the SEE ground motion was carried out using the equivalent linear method. This method consists of an iterative computational procedure to adjust the damping ratio and the dynamic shear stiffness of each finite element until these dynamic properties are compatible with the dynamic shear strains. The equivalent linear method is widely used in practice for the dynamic analysis of embankment dams because a great deal of information is available in the literature on the material properties required for this analysis. The horizontal and vertical components of the ground motion were applied at the bedrock surface.

<u>Earthquake Response Analysis Results</u>

The dynamic analysis was carried out by the computer program QUAKE/W, (GEO-SLOPE International Ltd., 2007). Selected results of the earthquake response analysis in case A are given here, in Figures 10 to 12. The peak absolute horizontal crest acceleration due to the SEE excitation is about 0.78 g for the average dynamic material properties, and about 1.16 g for the most unfavorable material properties.

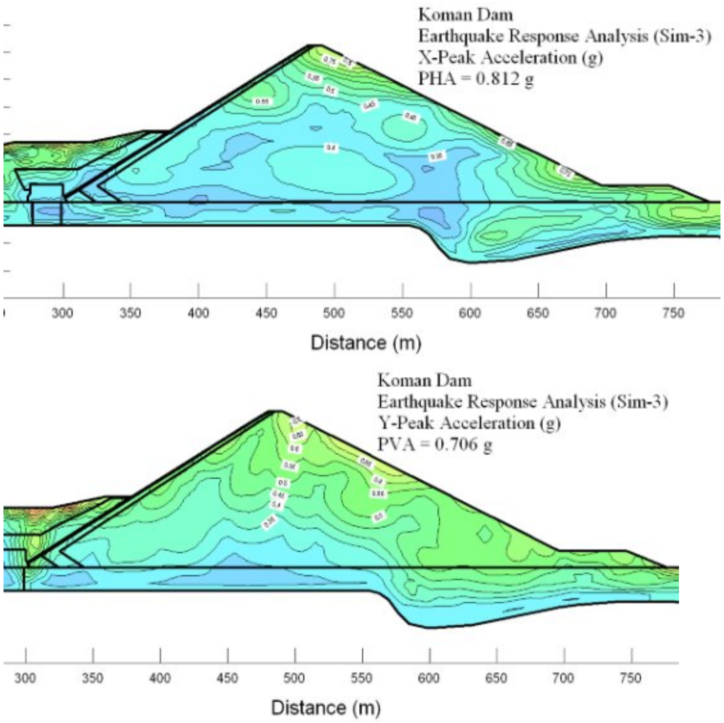


Fig. 10: Contour plots of absolute maximum horizontal (top) and vertical (bottom) accelerations in dam body (SEE and average material properties)

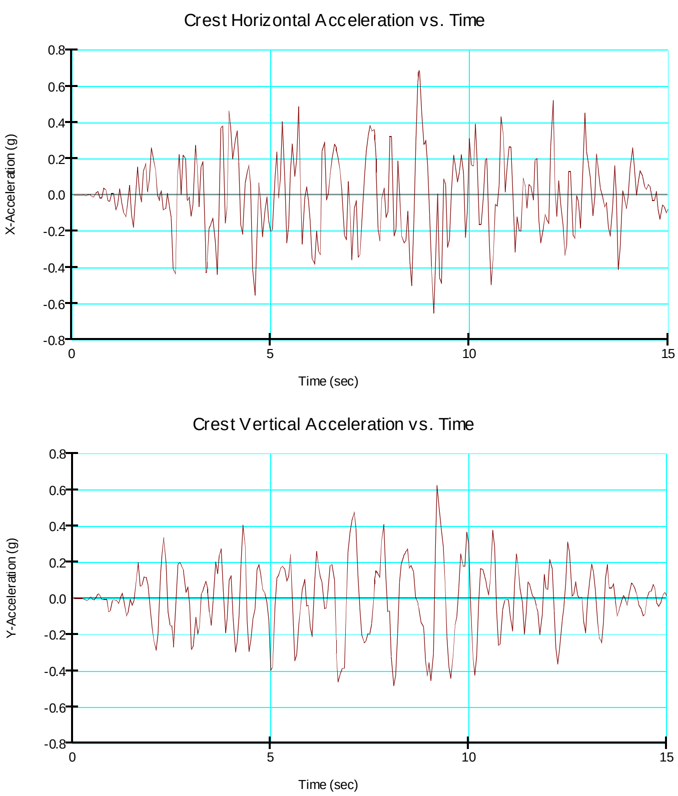


Fig. 11: Time histories of horizontal (top) and vertical (bottom) crest acceleration (SEE and average material properties), peak acceleration horizontal 0.78 g vertical 0.63g

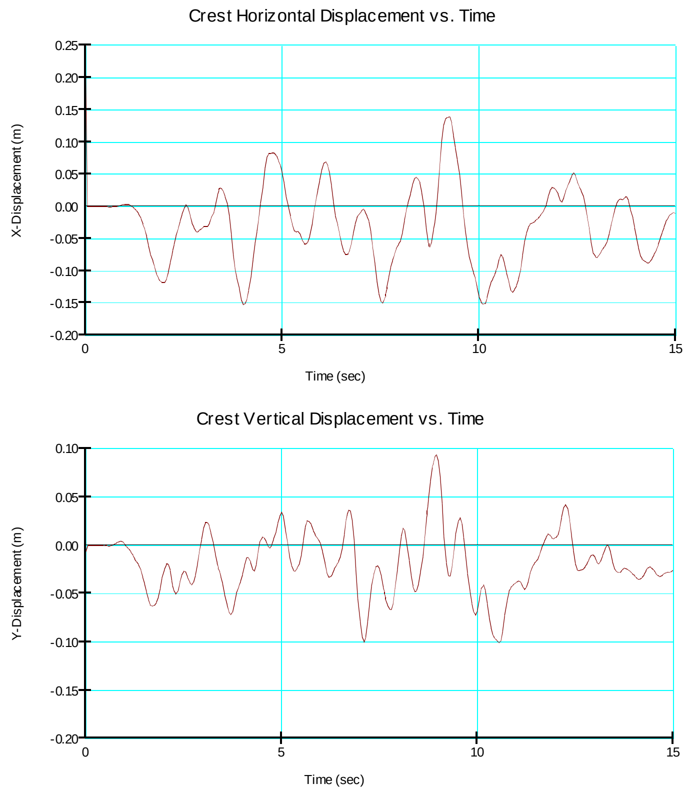


Fig. 12: Time histories of horizontal (top) and vertical (bottom) crest displacements (SEE and average material properties), peak displacement horizontal: 0.153 m, vertical: 0.101 m

DYNAMIC SLOPE STABILITY ANALYSIS

Calculation of Yield Accelerations

For the dynamic stability analysis of a potential sliding mass, its yield acceleration is first determined. The shear strength parameters for the dynamic slope stability analysis are presented in Table 4. The shear strength of coarse grain materials is expressed in terms of a stress-dependent friction angle (Barton and Kjaernsli, 1981) as follows:

$$\phi' \quad \phi'_o - \Delta\phi' \log_{10}(\sigma'_n/p_a) \tag{2}$$

where σ'_n is the effective normal stress and p_a is the atmospheric pressure (0.1 MPa). Thus, ϕ'_o is the friction angle corresponding to $\sigma'_n \quad p_a$ and $\Delta\phi'$ is the reduction of the friction angle for every ten-fold increase of the confining stress.

Table 4: Material properties for dynamic slope stability analysis

Description	γ (kN/m³)	c' (kPa)	ϕ_o' (°)	$\Delta\phi$' (°)
Clay fill	18	0	24	-
Fine transition	20	0	40	4
Coarse transition	20	0	43	5.5
Rockfill	21	0	46	7
Random fill	20	0	38	4
Plinth and gallery block	24	250	40	-
Coarse alluvium	21	0	32	-
Grouted alluvium	22	150	38	-

Newmark Sliding Block Analysis

If the inertial forces acting on a potential sliding mass on a dam slope during an earthquake become sufficiently large, the total (static plus dynamic) driving force would exceed the available resisting force. In other words, once the horizontal acceleration is larger than the yield acceleration, the factor of safety would drop below 1.0, implying that the potential sliding mass starts to move. The relative velocity of the sliding mass grows as long as the earthquake acceleration remains above the yield level. When the acceleration falls below the yield level, the motion gets braked, and after some time, the sliding mass sticks to the underlying material again.

The main results of Newmark's sliding block analysis for the Koman dam are given in Table 5, for different analysis cases. Also, a representative set of Newmark analysis results for the dam downstream critical slip surface at analysis case "A" (average dynamic material properties) is illustrated in Fig. 13.

Table 5: Summary of results of Newmark's sliding block analyses of critical slopes at upstream and downstream faces of Koman dam

Case	G_{max} and G/G_{max} vs. shear strain curve Fig. 4	Damping ratio vs. shear strain curve Fig. 4	Horizontal yield acceleration of critical slopes (g)		Maximum sliding of critical slopes in slope direction (cm)	
			U/S	D/S	U/S	D/S
A	Average	Average	0.93	0.65	1.3	2.9
B	Average	Lower bound	0.89	0.65	2.2	12.8
C	Upper bound	Average	0.93	0.65	1.3	14.8
D	Upper bound	Lower bound	0.93	0.65	16.0	48.5

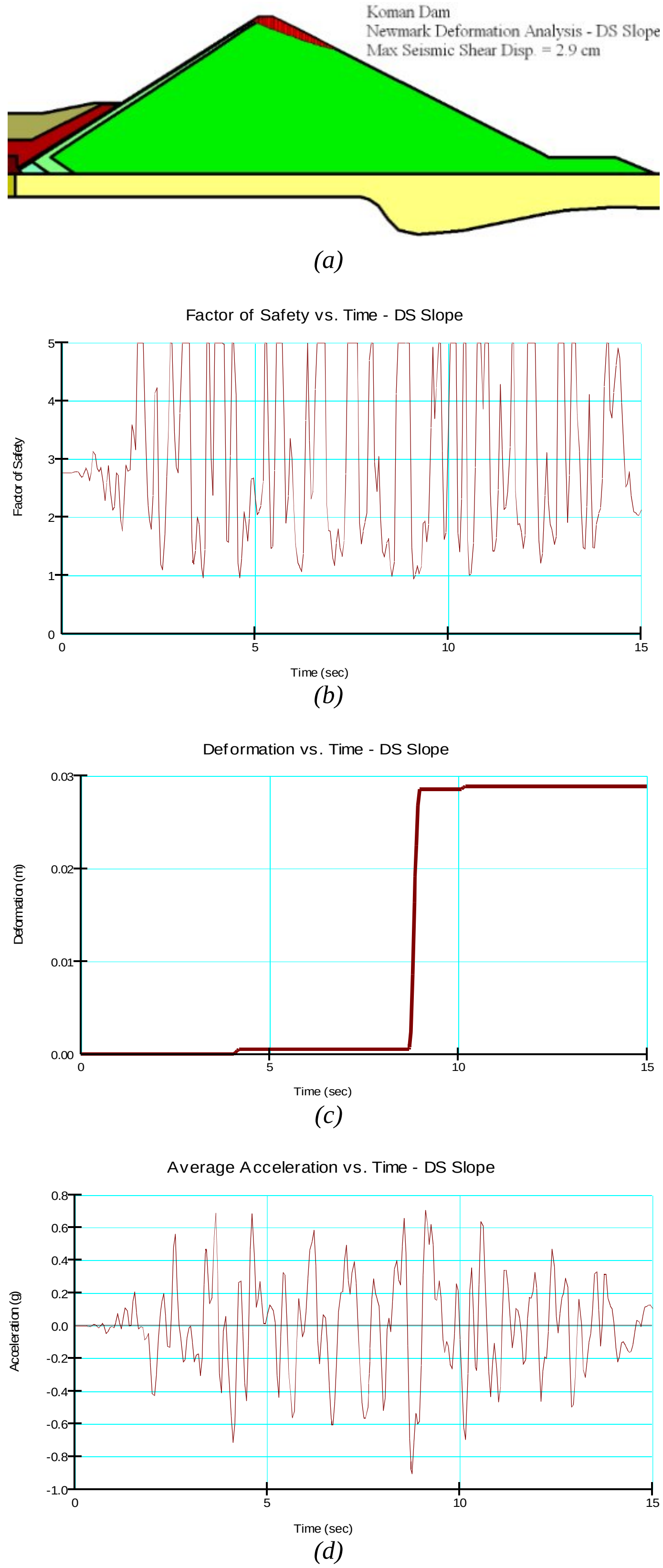


Fig. 13: Newmark sliding block analysis for DS slope: (a) critical slip surface, (b) Time history of factor of safety, (c) Time history of sliding movement, and (d) Average acceleration time history of critical slip surface

Estimate of Loss of Freeboard

The sliding displacements of the critical upstream and downstream slopes occur at different times and, thus, can be assumed to have a cumulative effect on the vertical displacement of the crest region. Accordingly, the total reduction of the freeboard due to earthquake-induced sliding movements is obtained as the sum of the vertical projections of the sliding movements of the upstream and downstream slopes. The angle of the slope movement with the vertical axis is about 66° for the critical slopes at the upstream and downstream faces of the dam. It should be pointed out that the critical slopes were selected in such a way that they include the crest of the dam and that the slope movements lead to a vertical displacement of the complete crest.

In the worst case, i.e. Case D in Table 5, the total vertical movement due to slope movements is 26 cm which is the sum of the vertical components: 6.3 cm (upstream slope) plus 19.7 cm (downstream slope).

In the dynamic analysis earthquake ground motions with durations of 15 s were selected. As the earthquake-induced slope movements are roughly proportional to the duration of strong ground shaking, therefore, it can be assumed that an earthquake of 30 s duration will cause vertical movements, which are roughly twice as large, than those given above.

Bureau (1997) developed an empirical relationship between the local intensity of shaking, expressed in terms of the Earthquake Severity Index (ESI), and the permanent crest settlement. The ESI is defined as:

$$\mathrm{ESI} = A\,(M - 4.5)^3 \tag{3}$$

where A is the peak ground acceleration in g and M is the earthquake magnitude. The chart shown in Figure 14 was prepared on the basis of physical model tests, taking into account also recorded seismic performance of several concrete face and earth core rockfill dams founded on rock and numerical studies using elasto-plastic dynamic models (Bureau, 1997).

In the present case, with A = 0.448 g and M = 7.0, ESI is equal to about 7. Assuming a friction angle of 42°, the relative crest settlement (i.e. settlement expressed as a fraction of the dam height) is equal to about 0.18%. As the dam is 115 m high, the seismic settlement estimated by the Bureau method is about 20.7 cm.

At extreme condition with M = 7.5, ESI is equal to about 12.1 and the relative crest settlement is equal to about 0.40%, which results in the permanent seismic settlement of 46 cm.

Therefore, if all unfavorable cases are combined we will get for an earthquake with duration of 30 s and a magnitude of 7.5 a crest settlement of 52 cm due to slope movement and 46 cm due to material densification, i.e. a total of 98 cm. If for design purposes of the freeboard a safety factor of 2 is assumed, then based on the previous estimates a freeboard of roughly 2 m would be adequate to cope with the SEE ground motions. Such a freeboard is available at Koman.

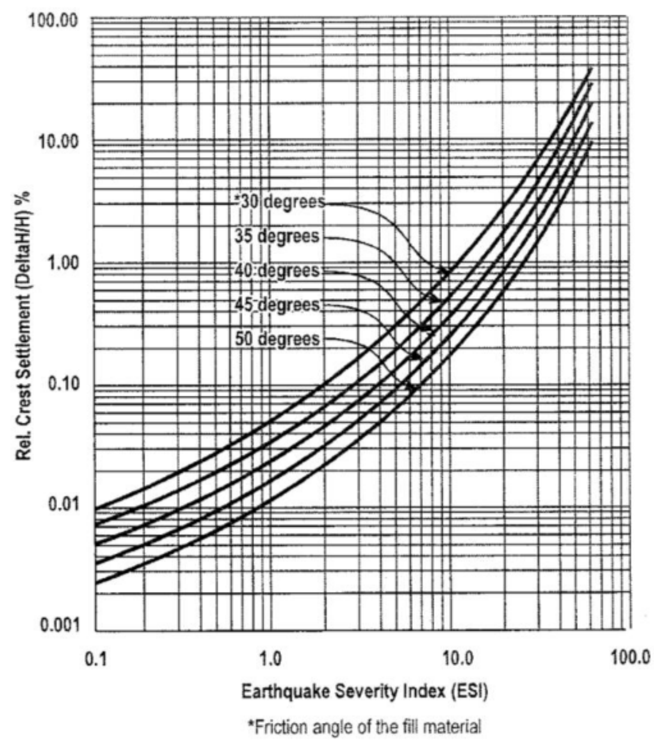


Fig. 14: Chart for estimation of crest settlement by Bureau method (Bureau, 1997)

CONCLUSIONS

The main conclusions of the static and dynamic analyses of Koman CFRD are as follows:

1. The maximum settlement due to the self-weight during dam construction is 1.69 m in the maximum cross-section at about mid-height of the dam. The maximum horizontal displacement was computed as about 0.56 m within the downstream shell.
2. The largest displacements of the face slab due to the water load in the vertical and horizontal directions during the first impoundment of the reservoir were computed as 0.72 m and 0.55 m, respectively. The largest displacements perpendicular to the face slab is 0.90 m.
3. The maximum absolute horizontal crest acceleration due to the SEE excitation is about 0.78 g for the average dynamic material properties, and 1.16 g for the most unfavorable material properties.
4. The dynamic slope stability calculations show that the maximum crest settlement resulting from the sliding displacements induced by the SEE ground excitation will be about 52 cm. The settlement due to

the vibration-induced densification of the dam body, using Bureau's method, is 46 cm. Thus the total loss of freeboard is estimated as 98 cm.

5. The present analyses have been carried out using material properties selected on the basis of very limited data available for Koman dam, literature, and engineering judgment, which limits the accuracy of the static and dynamic deformation analyses.
6. The above explained stability calculations show Koman dam is safe under static and seismic (SEE) loading conditions.

It must be pointed out that a two-dimensional dam model is only adequate for the assessment of the deformations of the dam body but not for the safety of the concrete face. For the latter a three-dimensional model of the dam is required as the stresses in the concrete face depend on (i) the deformational behavior of the dam during the SEE, (ii) the detailing of the vertical joints, and (iii) the cross-canyon component of the SEE. The damage observed at the 156 m high Zipingpu CFRD, caused by the May 12, 2008 Wenchuan earthquake in Sichuan, China, has shown that these three factors are governing responsible for the damage at the vertical and longitudinal joints of the concrete face. This type of damage cannot be assessed based on a conventional two-dimensional dam model. When the dam material is well graded and well compacted, and when the joints remain open during the SEE (this requires an adequate joint width) then the damage to the concrete face would be small even in the case of strong ground shaking (Wieland, 2003).

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