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Fracture Mechanics; The Future of Concrete Dam Safety Evaluation; Part II: A Case Study

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Abstract

On the basis of the results of an extensive research program reported in a companion article, fracture mechanics is used to assess the safety of an existing dam which has been targeted for major rehabilitation. It is found that current analysis methods may prove to be too conservative for this dam, and that a small investment in a fracture mechanics based analysis can lead to substantial savings in rehabilitation.

Introduction

In a companion paper [1] the authors reported on a safety research program which applied fracture mechanics to the analysis of concrete gravity dam. As a result of this research, a computer program MERLIN [2], which embodies capabilities to handle most of the experimental findings, was developed. In this paper, we now seek to apply the proposed methodology to assess the safety of an existing dam. It will be shown that for this particular case, the classical analytical approach based on rigid body equilibrium may prove to be too conservative, and a substantial saving in remediation cost could be achieved through a slightly more complex yet appropriate analysis. Two distinct fracture mechanics approach were investigated, linear and nonlinear ones.

Dam Description

The dam being considered is an actual one in the Eastern United States. The first author was given access to preliminary reports highlighting its safety investigation. The safety evaluation reported in this article was performed to provide a comparison of the fracture mechanics approach. The dam owners are aware of the results but have not contracted for the work. Accordingly, we shall refer to it as Greyrock dam, and technical references to those reports will be omitted.

Greyrock dam is a 176 ft high dam which was built in the thirties. It has 53 monoliths with a crest elevation of 1,535 ft and a drain gallery at elevation 1,375 ft. Based on recent hydrologic studies, it was determined (using rigid body equilibrium) that the probable maximum flood (PMF) elevation is at 1,555.8 ft, yet its computed imminent failure flood (IFF), which is the pool elevation which would trigger failure, is at 1,533 ft or 22 ft below the PMF (assuming 50% drain efficiency). Hence, this dam has been targeted for major rehabilitation, Fig. 1.

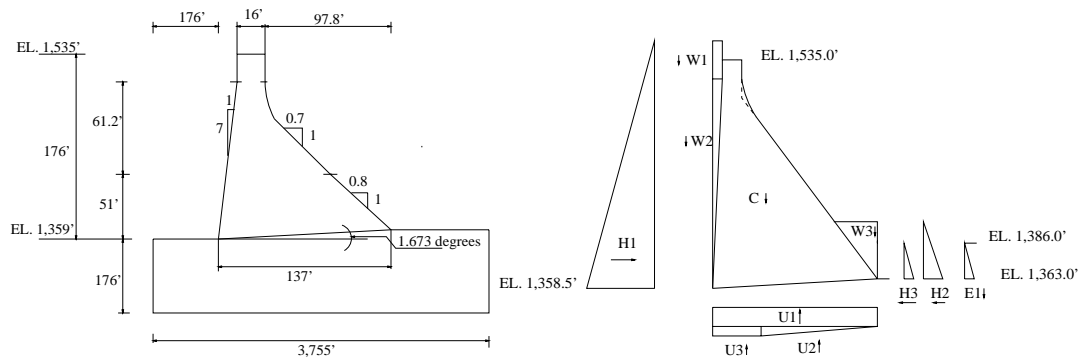


Figure 1: Greyrock Dam; Dimensions

Since the IFF is smaller than the PMF, an extensive and comprehensive study (including economic cost and loss of life) was conducted to assess potential consequences of dam failure. Following those studies, a remediation plan was developed, Fig. 2. The remediation cost for this dam was high (preliminary estimates

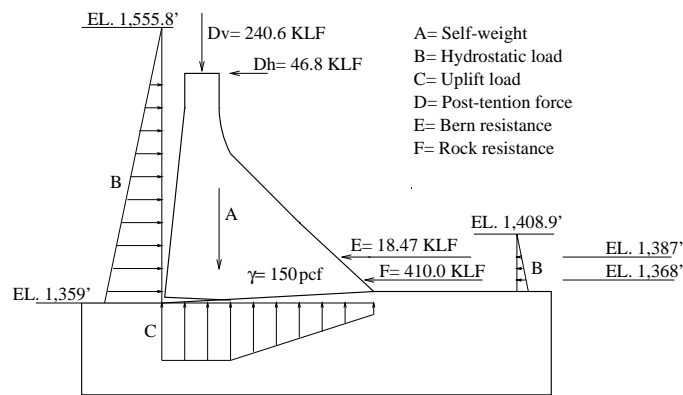


Figure 2: Proposed Retrofitting of Greyrock Dam; Monolith 12; PMF conditions

were higher than \$50 million). Finally, to the best of the authors knowledge, all the calculations were based on simple rigid body equilibrium and no finite element or fracture mechanics analysis were conducted even

though those are explicitly accepted methods by some federal agencies, [3, 4].

This paper focuses on the fracture mechanics analysis of the most critical monolith (12) and will determine if: 1) the dam is indeed unsafe as is, and 2) whether the rigid equilibrium analysis procedure is too conservative, and thus projected remediation cost can be reduced. It should be noted that in all our analyses, the drain was conservatively assumed to be ineffective (but not in the original analysis conducted by the dam owners).

LEFM Analyses

Using the finite element mesh shown in Fig. 3, a series of linear elastic fracture mechanics analysis are

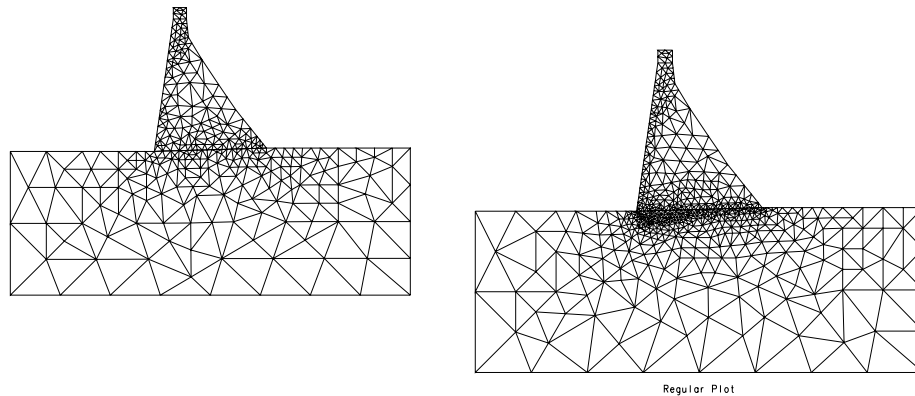


Figure 3: Finite Element Meshes Used in Greyrock Dam: LEFM and NLFM Analyses.

performed, and a curve plotting the stress intensity factor (SIF) in terms of crack length was generated. The crack is assumed to propagate as long as the SIF exceeds the fracture toughness (usually taken as zero). It should be noted that once the crack length is determined, there is no assurance that failure may not occur through sliding. Under the assumptions of LEFM, a number of analyses were performed (using a zero value for the fracture toughness). The results are summarized in Fig. 4.

Crack Lengths

The analysis of the original dam under the predicted IFF pool elevation (1,533 ft) yielded a total crack length of 65.0 ft. This value is quite close to the one predicted by rigid body equilibrium as reported (albeit with a 50% effective drain). A similar correlation of crack lengths by those two procedures was also found

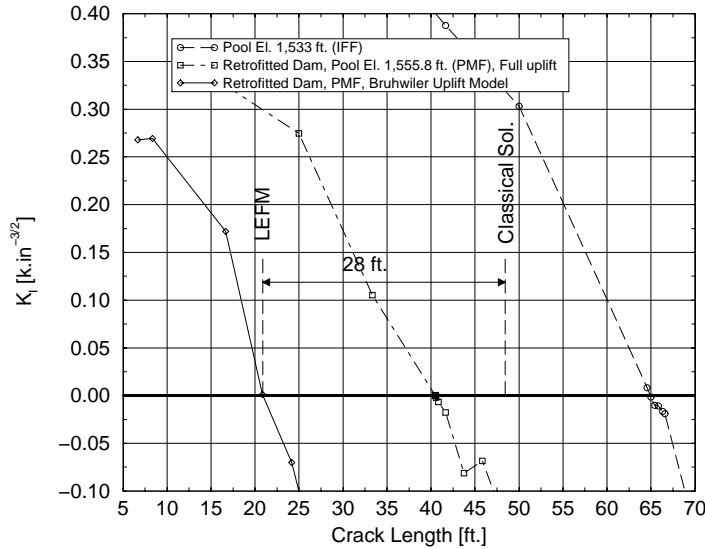


Figure 4: Greyrock Dam, LEFM Analyses, Original (IFF) and Retrofitted (PMF)

in the analysis of Pine Flat dam [5].

In the analyses of the retrofitted dam with full uplift, Fig. 2, under PMF conditions, a 40.4 ft. long crack developed. This can be compared to rigid body equilibrium result of 48.9 ft.

Effect of Uplift Pressure

In the opinion of the authors, none of the published guidelines [6, 7, 8] provide a clear indication on modeling uplift pressure within the context of finite element analysis. Adopting the reduced uplift model [1] in which the uplift pressure in a concrete crack is a function of the crack opening displacement (COD), the full uplift is slightly reduced around the crack tip, Fig. 5. Hence, whereas the resultant uplift force is practically the same as the one resulting from a full uplift model, this model has smaller hydrostatic pressures around the crack tip, and this is enough to reduce the crack length.

Using this model, it was found that for the retrofitted dam, the total crack length is further reduced from 40.4 ft. to 25 ft.

Effect of Individual Loads in the Retrofitted Dam

In order to better assess the importance of individual loads in the analysis of the dam under PMF conditions, Table 1 tabulates the stress intensity factors for each load case. We observe that the berm offers very little in

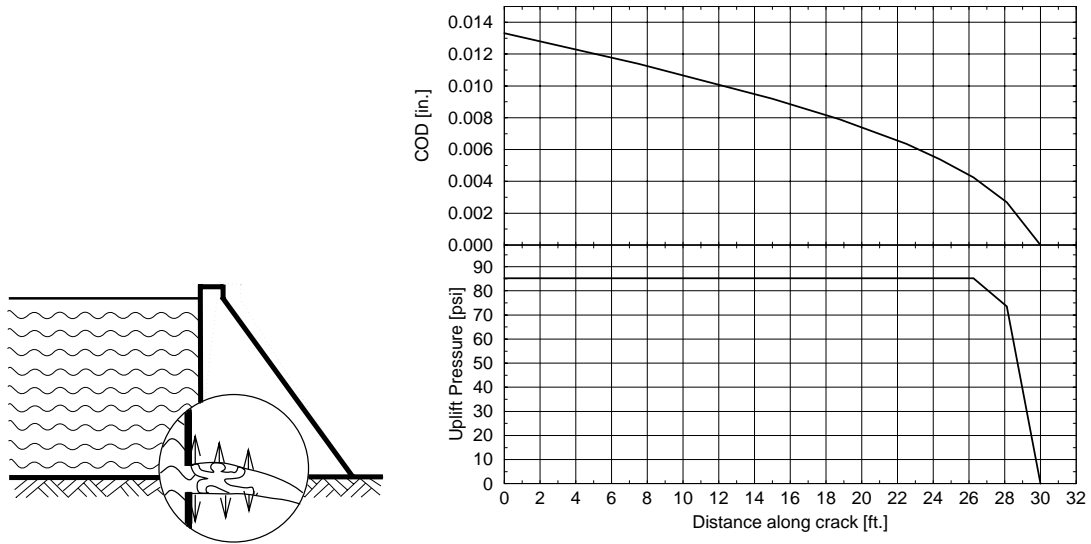


Figure 5: COD and Uplift Along the Crack

Load Case	SIF k.in ^{-3/2}	%
Hydrostatic Upstream	4.93	50.3
Hydrostatic Downstream	0.092	0.94
Uplift	4.78	48.8
Total	9.80	100
Self Weight	-7.27	74.2
Post-Tensioning	-2.25	23.0
Bern Resistance	-0.029	0.3
Rock Resistance	-0.25	2.5
Total	-9.81	100

Table 1: Stress intensity Factors for Individual Load Cases; PMF Study, Crack Length 40.4 ft.

terms of reducing the crack length in comparison to its potential cost. However, it may provide substantial resistance to horizontal forces and correspondingly increase the shear friction factor (SFF).

LEFM Based IFF

Next, we seek to determine the IFF, assuming a total crack length of 64.62 ft. Thus, in this analysis, we fix the crack length, modify the pool elevation, and use the reduced uplift model [1]. From Fig. 6 we observe that the pool elevation is now 1,540 ft (instead of 1,533 ft as determined from the rigid equilibrium method.)

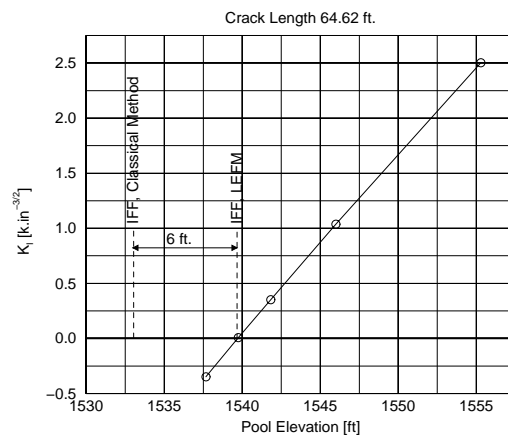


Figure 6: Determination of IFF for a Total Crack Length of 64.62 ft. (Original Condition)

Crack Between Two Dissimilar Materials

It is well known that an oscillatory stress singularity exists at the tip of a crack sandwiched between two dissimilar materials (such as rock and concrete), thus classical linear elastic fracture mechanics is no longer applicable. If this oscillating singularity is to be accounted for, special care must be exercised in properly modeling it numerically, [9]. Again, using MERLIN, and considering the reported IFF elevation of 1,533 feet we determined the SIF, Figure 7.

We again consider two possibilities:

Straight Crack: In which we neglect the possibility of kinking, and assume the crack propagates along the rock/concrete interface. First, using the criteria of $K_{Ic} = 0$ the crack length determined is 65 feet. However, from [10] the critical fracture toughness for limestone/concrete interface was found to be 248 psi \sqrt{in} (0.28 MPa \sqrt{m}). With this value a smaller crack length of 56 feet is obtained.

Kinking Crack: Based on [10] the crack will kink when the energy release rate G is no longer constant. From Fig. 7 we observe that this occurs when the horizontal crack has reached a length of 33 feet. At that point (B), it will kink into the rock. Figure 7 also shows the kinked crack profile. Full uplift

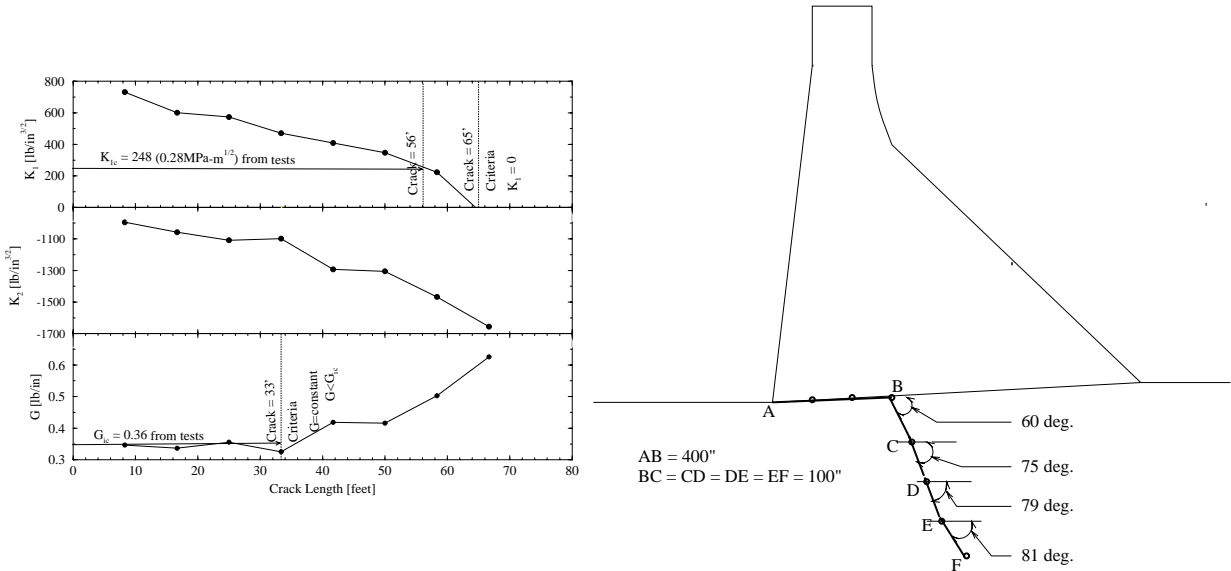


Figure 7: Straight and Kinking Cracks Between Two Dissimilar Materials

water pressures were used inside the cracks, and in this case the uplift pressure was the driving force for the kinking crack. It should be noted that the presence of tectonic lateral stresses in the foundation would eventually close a dipping crack. Since these stresses were not accounted for, the analysis was terminated when the crack had reached point F.

NLFM Analyses

Contrarily to the LEFM analysis where all the energy is transferred at the crack tip, in a nonlinear fracture mechanics (NLFM) analysis, energy is also transferred along the crack. The model adopted is the Interface Crack Model (ICM) [11] which is implemented in MERLIN. It is important to note that in this model, cohesive stresses along the fracture process zone are modelled, and hence there is no stress singularity. The criterion for crack propagation is a strength-based one in which the tensile stress can not exceed the tensile strength. Furthermore, this element is capable of modelling shear failure along the dam/rock interface.

Hence, the ICM elements may fail either due to crack openings or due to excessive shear stresses. In other words this is a truly comprehensive mixed mode model.

As part of the output, MERLIN tabulates for each node along the dam rock foundation: normal, tangential stresses, crack opening and sliding, and the uplift pressure. From those parameters the crack length, and the SFF factor can readily be determined.

Original Dam

Again two studies will be performed. In the first one the crack is assumed to propagate colinearly, and in the second it is allowed to kink.

Straight Crack: In the no-kinking analysis the crack is assumed to propagate along the interface of the concrete dam and rock foundation. With this assumption, interface elements are provided all along the interface. The analysis is carried out by incrementally increasing the water elevations, with the first increment being the gravity dead load and subsequent incremental loads corresponding to increased pool elevation. Failure (usually by sliding) will occur when the program fails to converge.

In this analysis, the nonlinear solution failed to converge at a pool elevation of 177 ft (IFF=1,536 ft) for $G_F^I = 0.35$ lb/in and 179 ft (IFF=1,538 ft) for $G_F^I = 0.70$ lb/ft. The corresponding crack lengths along the interface were 88 feet and 109 feet respectively.

Kinking Crack: In the crack kinking analysis the crack is assumed to originate at the junction of the concrete dam and rock foundation near the heel of the dam. Hence interface elements are provided along the interface. Analysis is again carried out incrementally.

Figure 8 shows the final crack profile with respect to changing water elevations. It is seen that the crack remains within the interface for 8.3 feet (Point A), and then kinks into the adjoining rock when the water elevation is at 175 feet. This elevation is one foot above the IFF determined using classical equilibrium analysis.

The kinked crack propagates alone until the water elevation reaches 184 feet. At this point the crack branches out i.e., the interface crack starts to propagate again. When the water level reaches 190.8 feet (which is 8 feet below the PMF), the nonlinear solution failed to converge due to the presence of high shear stresses. This indicated final failure due to sliding.

At final failure the lower face of the interface crack was pushed upwards by the uplift forces within the kinked crack. This helped in partially closing the interface crack.

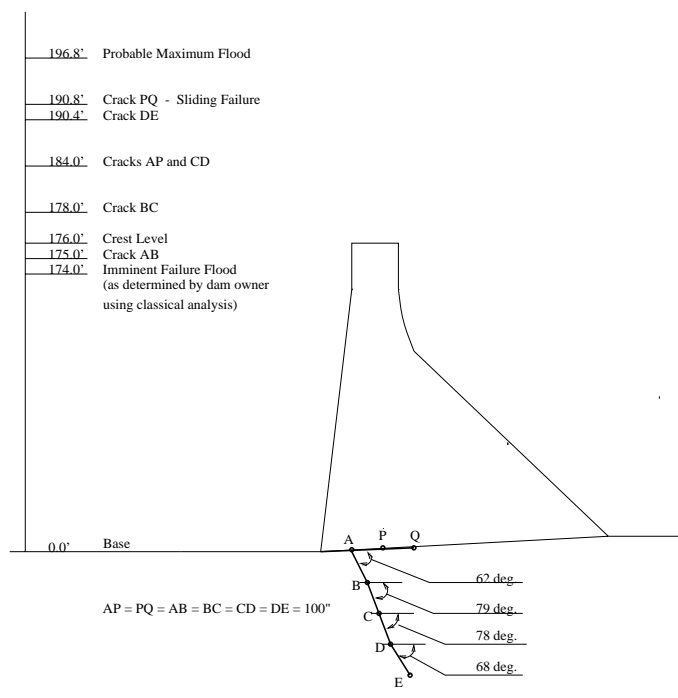


Figure 8: Final crack profile with respect to changing water elevations

Retrofitted Dam

Similar analyses are conducted for the retrofitted dam, and results are shown in Fig. 9 and 10. We observe

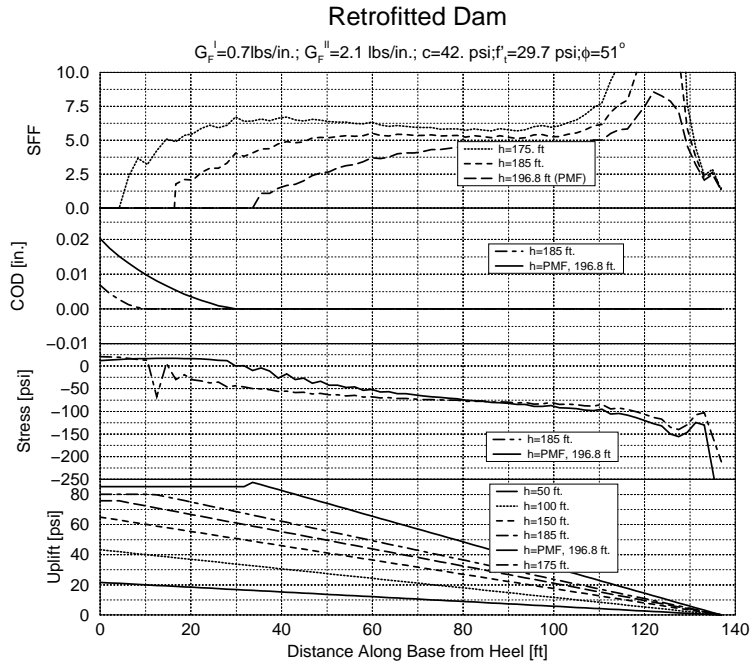


Figure 9: Nonlinear Analysis; Stress, COD, SFF, and Uplift Distributions

that the uplift pressure is constant throughout the crack, and then decreases linearly along the uncracked ligament. For the PMF condition, the crack length is 33.0 ft. The crack opening displacements are small, and the shape of the crack profile is different from the one observed for the case without post-tensioning. The stress distribution shows some high values at the toe. Those are caused by geometrical discontinuity and should be smoothed. Otherwise, the normal stress distribution is quite linear, and the shear is relatively constant. Based on the stresses (which are the effective ones), the locally determined SFF distribution is shown in Fig. 10. When the retrofitted dam was analysed the IFF was found to be 1,569 ft. for G_F^I equal to 0.35 lbs/in. For the last three elevations the SFF values are given in Table 2

Pool El.	1,534	1,544	1,555.8
Height	175	185	196.8
SFF	6.7	4.7	2.6

Table 2: SFF for Retrofitted Dam

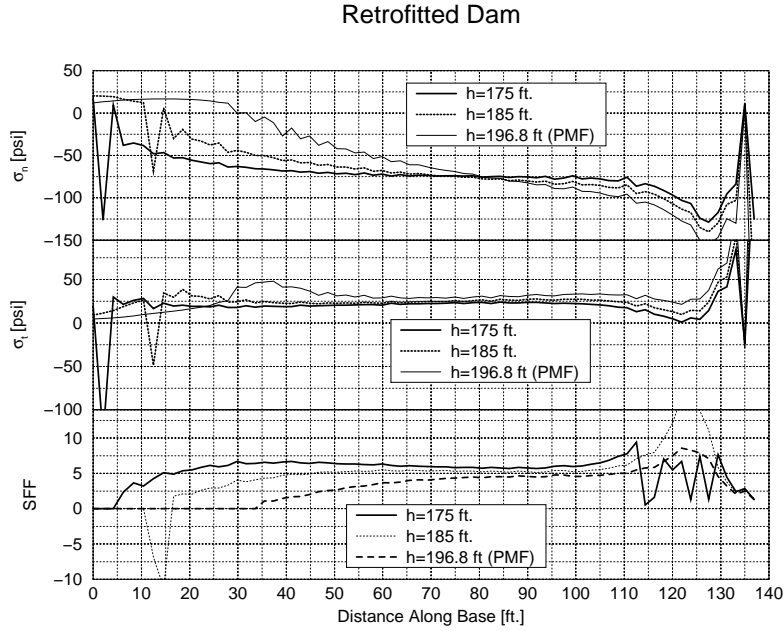


Figure 10: Nonlinear Analysis; Stress and SFF Distribution

Finally, the crest displacements and deformed meshes at different pool elevations are shown in Fig. 11 and 12 respectively.

Uplift Pressure in a Dam Following an Earthquake

Using the model developed for the transient uplift pressure in a dam, as reported in [12] we next investigate the case in which the dam has been hit by an earthquake and assuming [6] that there was no uplift pressure during the seismic excitation, we now seek to determine the time it would take for the full uplift to be applied when the water head is at 50 and 60 m. Such a study should be relevant to assess the ability of a dam to sustain after shocks. We consider two scenarios:

1. The 20 m crack develops during the earthquake, and we seek the time it will take for the full pressure to develop over the entire crack length.
2. We assume that we already had a fully saturated initial crack of 10 m (which might have been caused by a previous overtopping), and we now seek to determine the time it will take for the remaining 10 m to be fully pressurized.

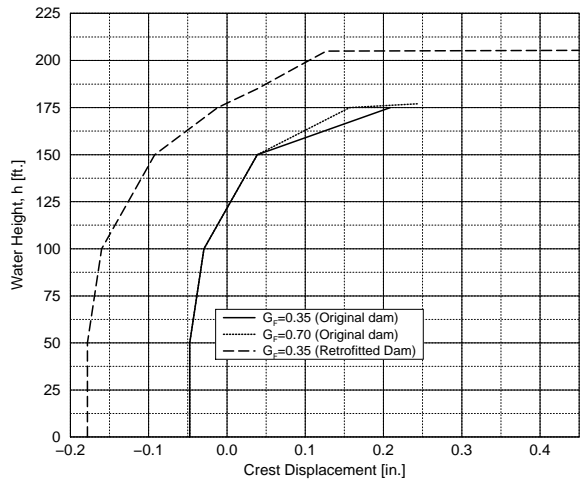


Figure 11: Crest Displacements in Terms of Water Height

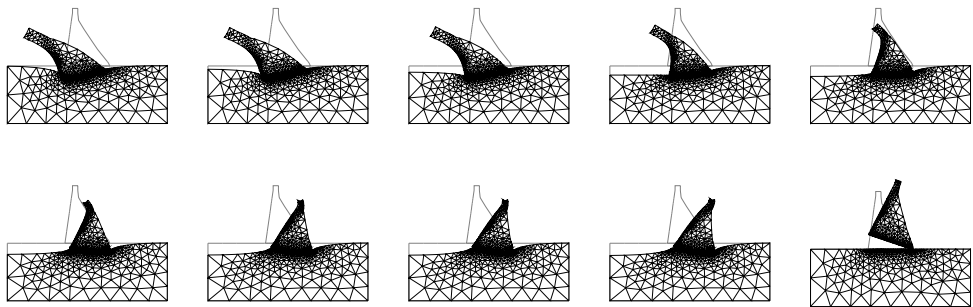


Figure 12: Deformed Meshes at water height: 0, 50, 100, 150, 175, 185, 196.8, 200, 205, and 210 (failure) ft

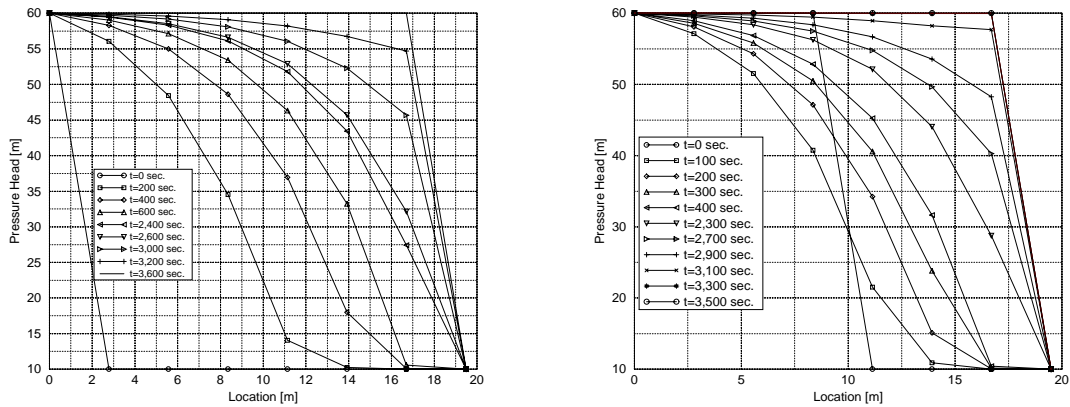


Figure 13: Pressure Heads Along the Crack for a 50 m. Reservoir Level; Scenario 1 and 2

Fig. 13 illustrates the pressure distribution along the crack (noting that the pressure head prior to crack opening is equal to the atmospheric one of 10 m.). The most important observation is that within an hour following the earthquake, the full uplift pressure develops along the crack. Furthermore, there is no big difference in the pressure built-up time for the two cases. This is caused by the rapid build up in pressure along the first 10 meter where the crack opening displacements (and thus conductivity) are relatively large. Finally we note that in the second case the pressure in the first segment temporarily decreases while water flows into the newly formed crack, and then rises again.

Summary and Conclusions

Summary of the various analyses is shown Table 3, and 4

Analysis	Pool El.	Height
Original Dam		
Classical	1,533	174
LEFM	1,540	181
Retrofitted Dam		
NLFM ($G_F^I=0.35$)	1,569	210

Table 3: Summary of IFF Evaluations for a 63.5 ft. Crack Length

From these tables, it is quite apparent that in all cases a fracture mechanics based investigation yields shorter crack/higher IFF than those predicted by classical rigid body equilibrium (even though our analysis

	Original				Retrofitted
IFF (feet)	1533.0	1536.0	1538.0	1549.9	1,555.8
Height (feet)	174.0	177.0	179.0	190.8	196.8
Classical Analysis					
Equilibrium analysis	65				48
LEFM Analysis					
Homogeneous ¹ ,					
$K_{1c} = 0$, Full Uplift	65				40.4
$K_{1c} = 0$, Reduced Uplift	60				21
$K_{1c} = 100$, Full Uplift					32.5
$K_{1c} = 100$, Reduced Uplift					18
Bimaterial, No - Kinking Analysis					
$K_{1c} = 0$, Full Uplift	65				
$K_{1c} = 248 \text{ psi}\sqrt{\text{in}}$, Full Uplift	56				
$G_c = 0.36 \text{ lb/in}$, Full Uplift	33				
Bimaterial, Kinking Analysis ²					
$G_c = 0.36 \text{ lb/in}$, Full Uplift ³	33				
NLFM Analysis					
No - Kinking Analysis					
$G_F^I = 0.35$, Full Uplift		88			35
$G_F^I = 0.70$, Full Uplift			109		
Kinking Analysis					
$G_F^I = 0.35$, Full Uplift ³				25	
¹ In this case the effect of having two different material on either side of the interface is not considered. ² Although the homogeneous and bimaterial cases lead to the same crack length, the angle of crack propagation will be different. ³ Crack kinking into rock					

Table 4: Summary of Crack Lengths (All dimensions in feet)

neglected the presence of a drain).

We also note that a variety of fracture mechanics investigations can be conducted, linear versus non linear, homogeneous crack versus one between dissimilar material, straight versus kinking, full uplift versus reduced uplift. Whereas each of those methods is acceptable, we should not base our final conclusions on a single one. Rather, the results of all those analyses. combined with field investigation, laboratory testing and plain old engineering common sense could provide a clear picture as to the dam safety and possible need and extent of remediation.

Finally, it is hoped that this paper highlighted the need for engineers to accept the undertaking of more complex analysis procedures than the ones they may be accustomed to. This necessity does not only stem from fiscal realities which prompt us to carefully examine expensive remediation work, but also from recent

technical advances.

Hence, a small incremental increase in analysis cost could result in major savings in rehabilitation cost.

Acknowledgments

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