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FAILURE THRESHOLDS FOR RCC CREST CAPS

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Abstract. Overtopping protection using roller-compacted concrete (RCC) overlays is a common improvement made to address hydrologic deficiencies at existing small- to medium-height embankment dams. Overlays allow floods to pass over the dam while preventing erosion of the original embankment. The United States has many examples of such projects on both private and public dams, including many associated with federal agencies of the Departments of the Interior and Agriculture. For one project modified with an overlay in 1993, deterioration of the RCC and poor bonding between lifts has caused concern for stability of the 0.3-m (1-ft) thick RCC lifts that cap the dam. Analytical studies from 2013 to the present have assessed potential failure modes during overtopping flow and have produced widely varying estimates of the probability of failure. To address this uncertainty, physical scale modeling in the Bureau of Reclamation Hydraulics Laboratory was used to evaluate crest cap failure modes for the conservative condition of unbonded RCC lifts. The testing defines the threshold conditions for failure, which will enable effective evaluation of risk for similar facilities in the future.

1 INTRODUCTION

Umbarger Dam is a 13.2-m (43.3-ft) high dam on Tierra Blanca Creek in Randall County, Texas, about 16 km (10 miles) upstream and west from the city of Canyon, Texas, population 15,650 (Figure 1). The dam was originally constructed in 1938 by the Public Works Administration and Works Progress Administration as a homogeneous embankment with a height of 15.5 m (51 ft). Materials in the original dam are described as well-compacted sandy clays and lean clays with 75 to 85 percent medium plasticity fines and a plasticity index ranging from 12 to 19. The dam is now owned by the U.S. Dept. of the Interior, Fish & Wildlife Service (FWS) and is a principal feature of the Buffalo Lake National Wildlife Refuge. To address dam safety concerns, the dam was modified in 1993, reducing the crest elevation by 2.4 m (8 ft) and adding roller-compacted concrete (RCC) armoring to the crest and downstream slope (Figure 2). The existing spillway was abandoned and partially covered by the new overlay, since the modified dam was meant to serve as its own spillway, passing necessary flood flows over the armored embankment. The modified dam crest is now 7.6 m (25 ft) wide and about 290 m (950 ft) long.

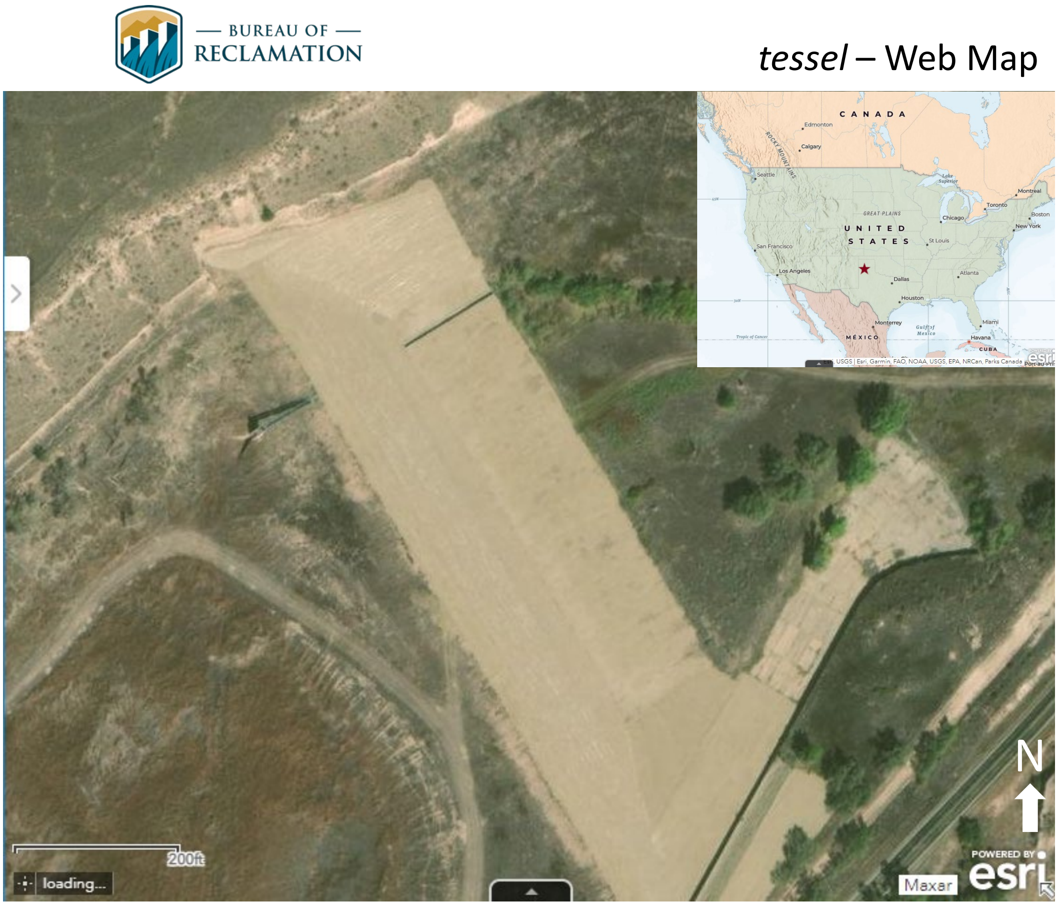


Figure 1: Umbarger Dam aerial photograph and location map (inset).

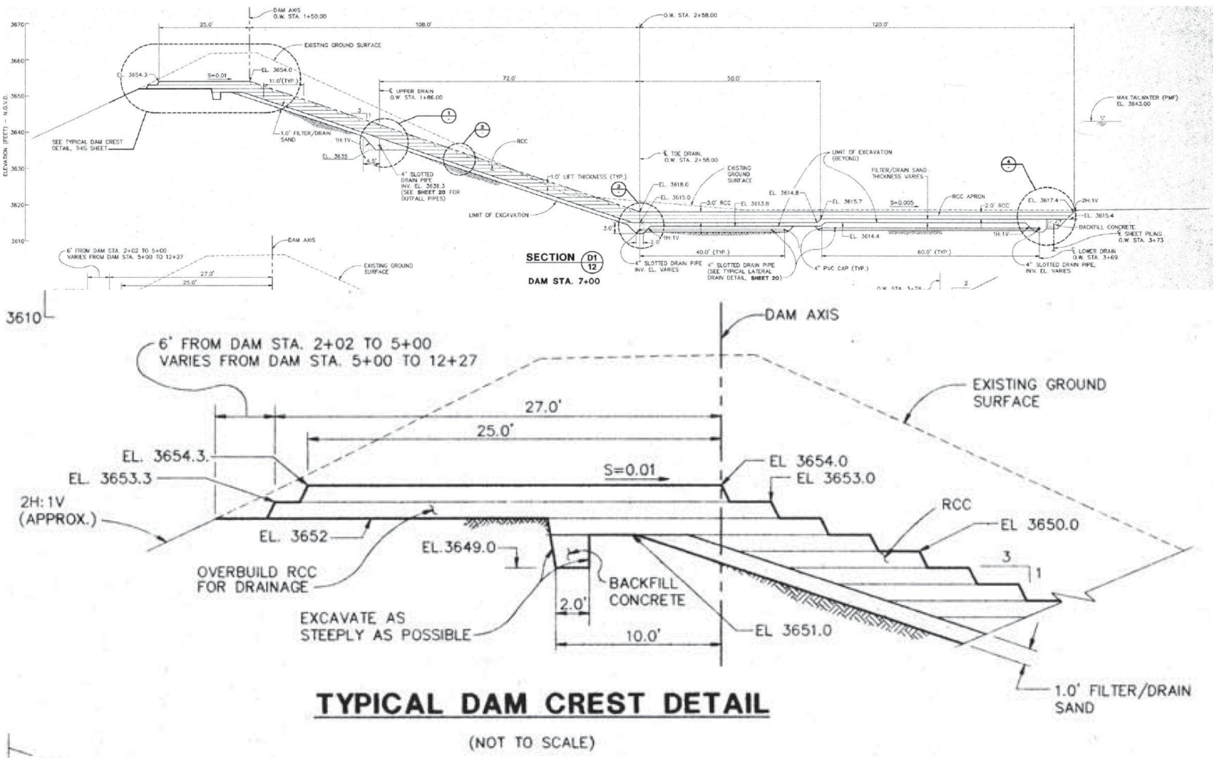


Figure 2: RCC overlay modification of Umbarger Dam.

Reservoir inflows have gradually reduced since the 1960s due to construction of many small check dams and deep irrigation wells in the watershed. As a result, Buffalo Lake is now typically dry, and Umbarger Dam only impounds water following major storms. Since the completion of the 1993 modifications, the dam has not impounded any significant pools except after a flood in the late 1990s that filled the reservoir to 25% of full volume, about 6.1 m (20 ft) below the dam crest.

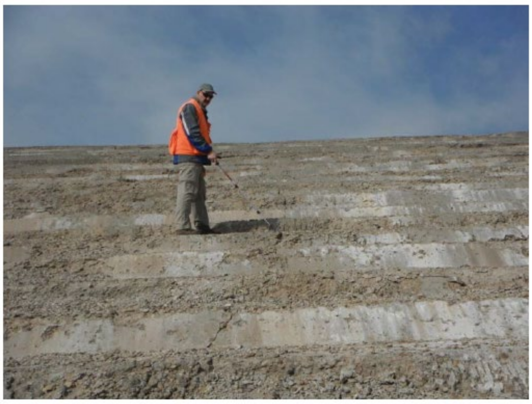
2 PROBLEM DESCRIPTION

The RCC overlay at Umbarger Dam has deteriorated significantly since the 1993 modification (Figure 3). Recent inspections have identified significant longitudinal cracks in the top 0.3-m (1-ft) thick RCC lift that caps the dam, and soundings suggest that the crest-cap lifts are poorly bonded to one another and to the underlying material. Transverse cracks have also been observed on the crest and downstream slopes. Freeze-thaw damage has also affected the RCC and is especially notable on the downstream slope. These observations are not unique for RCC overlays, since cracking is to be expected in a concrete placement that has no joints, but the degree of deterioration has been noteworthy and is probably due to the fact that this was a relatively early application of RCC with a mix design that was less resistant to freeze-thaw than modern mixes.

13. Transverse crack (foreground) in dam crest at STA 6+50 in foreground, looking left. Longitudinal crack extends from transverse crack about 20 feet to left. Top lift is unbonded left of transverse crack and downstream of longitudinal crack.

15. Downstream slope, looking left from near STA 7+50.

14. Longitudinal crack in dam crest near STA 8+50 and B-103, looking right from STA 8+60. This crack is about 0.25 to 0.5 inches wide and 25 feet long.



16. Downstream slope near STA 6+50, looking upstream. Note typical transverse crack.

Figure 3: Selection of photographs showing examples of RCC deterioration observed during 2013 Comprehensive Review inspection.

Instability of the RCC crest caps has been identified as a potential failure mode (PFM) for the dam. Overtopping flow could cause uplift/rotation of the top lift, or alternately, sliding. Uplift and rotation would probably occur almost simultaneously, since the moment arms associated with the major overturning and resisting forces are very similar (see Figure 11, later, for an illustration of the likely pressure distributions). Any uplift or rotation would lead to immediate sliding, since friction is the only force resisting sliding. Loss of the top lift would be presumed to initiate a sequence of subsequent RCC lift failures and erosion that would cause uncontrolled release of the reservoir.

2.1 Risk Studies

A 2011 analysis of the structure by Schnabel Engineering indicated a high risk of failure due to the overtopping PFM. A modification of the dam was proposed that would cap the RCC crest with another conventional concrete overlay.

In 2013 the Bureau of Reclamation (Reclamation) analyzed crest cap failure modes in conjunction with a risk analysis performed during the Comprehensive Review. (Reclamation provides Dam Safety technical services to FWS and other agencies of the Department of the Interior.) In an attempt to reduce analysis uncertainties, computational fluid dynamics (CFD) modeling was performed to determine hydrodynamic loads on the exposed surfaces of the RCC lifts on the dam crest. These loads were used to construct a free-body diagram of the crest-cap lift, but significant assumptions were still required regarding uplift pressures that might result from water penetrating the unbonded seam between the top two lifts. This analysis yielded large factors of safety against sliding and concluded that total risk was well below dam safety guidelines, with reduced justification to take action.

Schnabel Engineering revisited the issue in 2016, describing a failure mechanism involving the opening of a tension crack beneath the upstream edge of the top lift that would allow full reservoir head uplift pressure to be applied in the seam between the lifts. This would lead to uplift/overturning. This analysis concluded that risks were about 3 orders of magnitude larger than the 2013 result, placing total risk near the threshold at which actions to reduce or mitigate risk would be considered.

These analyses were all dependent on significant assumptions regarding the failure mechanism (uplift/rotation vs. sliding) and the magnitude and distribution of pressure forces above and below the top RCC lift. The large uncertainty in these estimates and the possibility for this failure mode to be relevant for other dams prompted Reclamation's Dam Safety Office to fund a series of simple physical model tests undertaken in Reclamation's Hydraulics Laboratory.

3 EXPERIMENTS

3.1 Initial Testing

Experiments were designed to represent the worst-case assumption that the top RCC lifts capping the dam crest are unbonded to one another and the embankment below. Initially, small structures composed of stacked brick pavers and other similar materials were assembled in a 0.3-m (1-ft) wide by 0.46-m (1.5-ft) deep rectangular flume in the hydraulics laboratory (Figure 4) with dimensions approximately representing the 2:1 (h:v) upstream slope of the dam and 3:1 downstream slope (exact matches of both slopes were not possible with standard-dimension bricks). The blocks fit loosely between the flume sidewalls (approximately 5 mm (0.2 in.) free space) so that there was no friction along the sides and no possibility for bricks to become bound between the walls. No attempt was made to bond the layers of brick pavers to one another or to control seepage through the joints between adjacent bricks and layers. Hydraulic pressure was allowed to penetrate the stacked bricks along all seams.

Figure 4: Initial flume testing, with illustration of key variables.

The initial flume required tests to be run quickly before excessive tailwater (not representative of the Umbarger Dam situation) developed in the downstream channel. The flow rate was increased steadily until the top block in the assembly slid or lifted off the top of the model dam. The exact failure mechanism (sliding or uplift) was difficult to discern, but results were obtained that suggested an approximately linear relation between relative failure head, *H/t*, and relative width of the crest, *w/t*. The flume could not produce sufficient head to cause failure for crest widths scaled to represent the thickness and width of the crest at Umbarger Dam, so tests were extended to include blocks of different thickness and lower density. A linear relation was then found between relative failure head, $H/(\gamma' t)$, and relative width, w/t, with the density adjustment factor $\gamma' = (\gamma - \gamma_w) / (\gamma_{RCC} - \gamma_w)$, γ = unit weight of the test block, γ_{RCC} = unit weight of typical RCC (145 lb/ft^3 = 2.32 g/cm^3), and γ_w = unit weight of water (62.37 lb/ft^3 = 1 g/cm^3). The initial tests showed (Figure 5) that the proportionality constant between relative failure head and relative block width [i.e., $H/(\gamma' t)/(w/t)$, which is the slope of a linear relation between relative failure head and relative width, depicted later in Figure 10] was not correlated with the height of the dam, represented nondimensionally by H/P. This indicated that approach velocity effects were negligible for failure head to dam height ratios of 2 or less. Such a ratio was maintained for all subsequent testing.

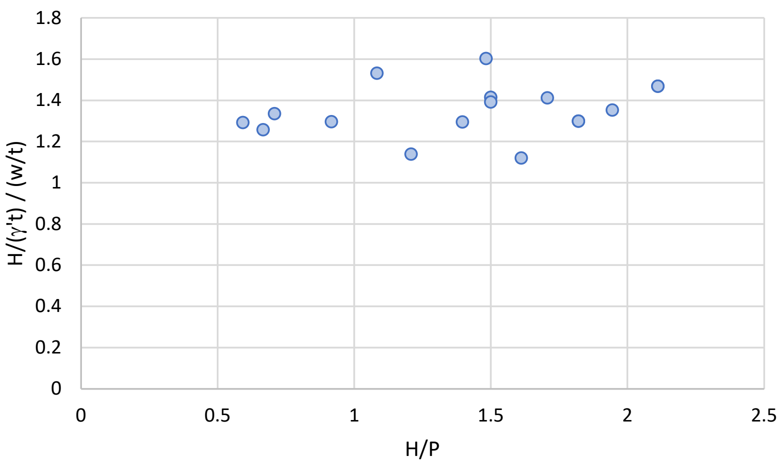


Figure 5: The ratio of relative failure head, $H/(\gamma' t)$, to relative crest width, w/t, is independent of the overtopping head to dam height ratio, H/P. Thus, approach velocity effects are negligible for $H/P \leq 2$.

3.2 Wider Flume Testing and Video Recording

Following the initial series of tests, additional work was performed in a 1.2-m (4-ft) wide flume where the downstream channel led to a free overfall that prevented excessive tailwater (Figure 6). The dam body was constructed from concrete pavers and wood fastened to the flume floor to create the basic dam shape with a smooth 2:1 upstream slope. For the crest, these tests used similar materials as the initial tests: concrete paver blocks, ceramic tile, and a concrete composite tile backer board material (Hardiebacker). The tile backer board is significantly lighter than standard concrete but absorbs water gradually while submerged. For tests run with this material, pieces were submerged for at least 30-45 minutes before testing and the unit weight of the tested piece was determined immediately after each trial was completed. Typically, the unit weight after saturation was about 1.68 g/cm^3 (105 lb/ft^3).

Figure 6: Overtopping testing conducted in wide flume with minimal tailwater.

For tests using concrete paver blocks and pieces of tile, multiple blocks were needed to span the width of the flume. Some tests using the tile backer board were performed with pieces that spanned the full width of the flume, minus a small gap at the edges to prevent binding of the pieces between flume walls. The surface friction properties of all tested materials were similar to one another and to the expected prototype RCC material; porcelain tile pieces had one smooth, glazed surface, but the piece that represented the lower of the two lifts capping the dam was inverted (smooth side down, rough side up), so that the two rough surfaces were in contact with each other.

Several tests in the wider flume were recorded using GoPro cameras at rates up to 240 frames per second to allow better observation of the failure mechanics. For most of the tests (except those described in sections 3.3 and 3.4), the video recordings clearly showed that failure occurred by sliding (Figure 7).

Figure 7: Sliding failure of a 40-mm (1.57-inch) concrete paver block. Flow is left to right, and time increases from left to right. Elapsed time from the first image to the last is about 1 second.

3.3 Overturning Failure Mode Tests

The simple stacking of unbonded blocks to create the dam structure allowed flow through the seams between the lifts that cap the dam crest. This was expected to allow some relief of the uplift pressure that was transmitted into the upstream edge of the seam between blocks. No attempt has ever been made to seal the upstream or downstream edge of the seams between these blocks at Umbarger Dam. However, to evaluate the potential effects of a sealed downstream edge, about 3 to 4 tests were conducted with a piece of duct tape applied to the downstream seam as shown in Figure 8. These tests produced uplift/rotation of the top lift around the downstream bottom corner at a very low head and the threshold was observed to be almost constant for increasing values of crest width, *w*.

Figure 8: Dam and crest cap configuration for a sealed-downstream-edge test.

3.4 Uplift Upstream from a Longitudinal Crack

To simulate a longitudinal crack like those observed at Umbarger Dam, several tests were conducted with the top lift split lengthwise into two pieces so that it would be possible for the upstream or downstream portion to slide or uplift sooner than a fully intact piece. Failure in these tests occurred by uplift of the upstream piece (Figure 9). Following removal of the upstream piece, the downstream piece failed by sliding.

Figure 9: Failure sequence for uplift of crest cap upstream from a longitudinal crack. Elapsed time is less than 1 second.

4 ANALYSIS OF RESULTS

A total of 25 tests were conducted with non-sealed downstream edges, and the results are plotted in Figure 10. Materials included 9.5-mm-thick ceramic tile, 10.3-mm Hardiebacker board, 38- and 40-mm brick and concrete pavers, and 57x89x190-mm, and 57x95x190-mm bricks installed in various orientations. The top lift thickness for Umbarger Dam is 305 mm (12 inches), so these materials represent model:prototype scale ratios between 1:32 and 1:5.33. The crest width at Umbarger Dam is 7.6 m (25 ft); the model tests spanned a range of w/t ratios so that relations could be developed with general applicability to other dams.

Flow capacity and tailwater control limits of the laboratory facilities prevented achieving failure in 8 tests. Failure occurred in 17 tests, with 3 of those tests involving crest caps that were composed of pieces separated by a simulated longitudinal crack. For the 14 sliding failures, the dashed line in Figure 10 representing the failure threshold relationship is defined by the equation $H/(\gamma' t) = 1.19(w/t)$. Although the failure mechanism was uplift (Figure 9), failure of the three tests involving a longitudinal crack occurred at a similar head threshold as predicted by the relation for sliding failures of complete blocks.

The tests performed with downstream edges sealed by duct tape are not included in Figure 10; the failure heads were very small, but this failure mode is not believed to realistically represent real-world conditions.

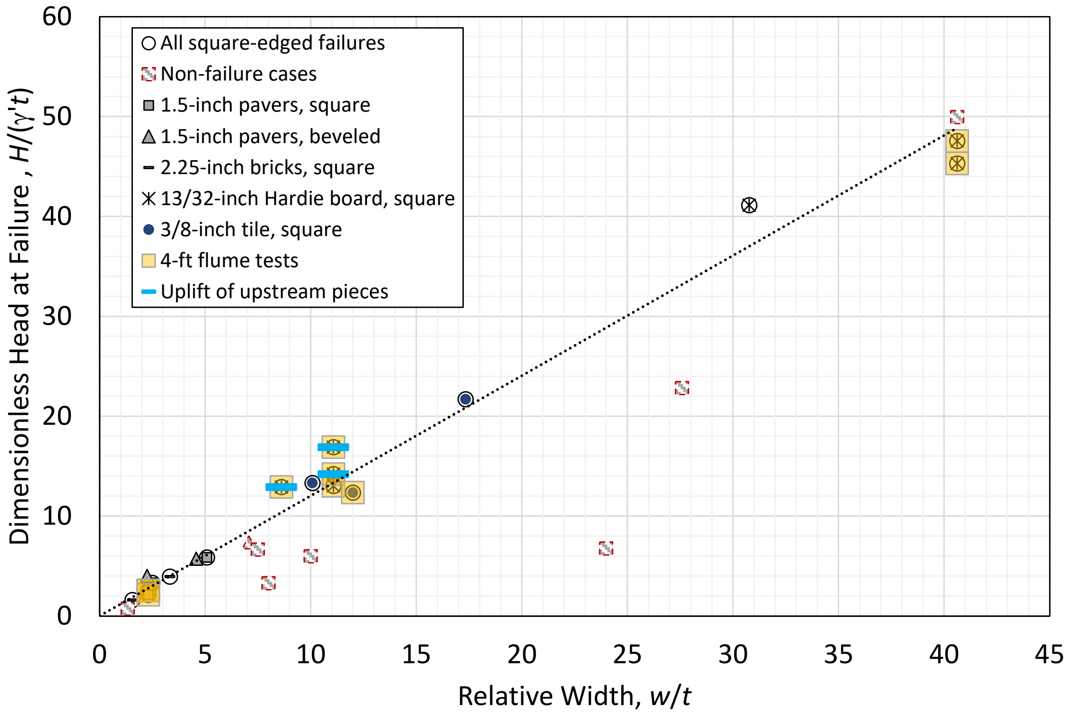


Figure 10: Failure head vs. relative width. Cross-hatched symbols with dashed red edges are cases that did not fail. (Dimensions in legend are material thickness in inches. 1 inch=25.4 mm)

Two of the tests included in Figure 10 used beveled-edge blocks cut to match the profile of the finished RCC lift faces at Umbarger Dam. These tests showed that the bevels had only a minor effect and tended to provide greater block stability.

5 DISCUSSION

5.1 Free Body Diagram Analysis - Sensitivity to Pressure Distributions

For the tests with sealed downstream edges, the failure by uplift/rotation at low heads is consistent with the previous analyses in 2011 and 2016 which specifically considered an uplift failure mechanism (imbalance of vertical forces and moments about the downstream edge). The tests with non-sealed downstream edges are more consistent with the 2013 analysis that considered sliding failure (an imbalance of horizontal forces). This prompts the question of why uplift/rotation is not the controlling failure mode for a non-sealed downstream edge.

To resolve this question the previous analyses were revisited by considering the forces on the top RCC lift as a free body, with the pressure distributions against all faces of the lift defined as linearly varying fractions of the upstream reservoir head (Figure 11). Coefficients defining the relevant fractions are indicated by the parameters *a-f* in Figure 11, which can each be varied from 0.0 to 1.0. Pressure forces against the upstream and downstream faces are assumed to be hydrostatic (coefficients *c* and *d*), while the uplift pressure below the slab and the weight of water above the slab are each assumed to vary linearly from upstream to downstream, as defined by the variables *a*, *b*, *e*, and *f*. Tractive force (fluid drag) along the top surface of the slab is neglected.

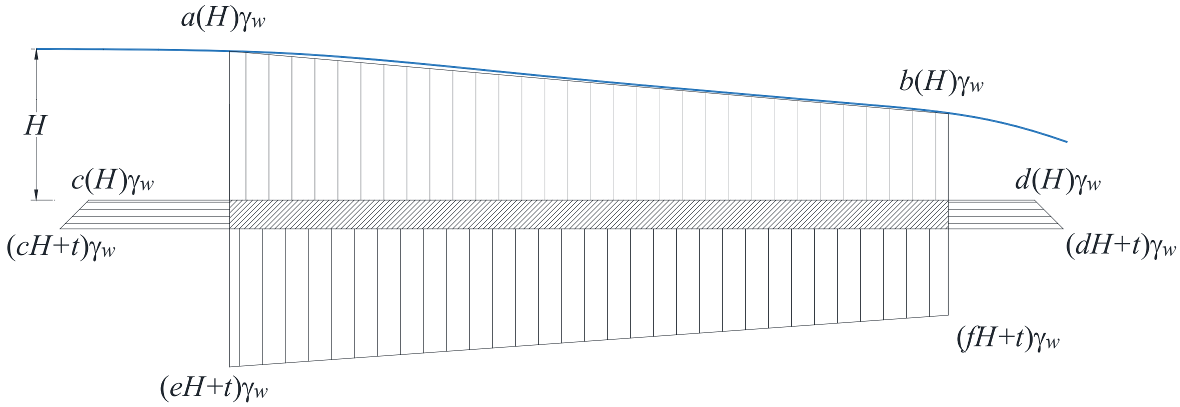


Figure 11: Sketch of pressure distributions applied to RCC crest cap lift. Coefficients *a-f* can each vary from 0.0-1.0. The blue line illustrates an approximate water surface profile.

Programming the equations for these pressure distributions into a spreadsheet model, the thresholds of sliding failure and uplift/rotation failure can be computed as the variables *a-f* are adjusted. The 2011 and 2016 analyses were based on assumptions of full reservoir uplift at the upstream bottom edge, and critical depth at the downstream bottom edge; full reservoir hydrostatic pressure on the upstream face; hydrostatic pressure equal to critical depth applied to the downstream face; and weight of water equal to critical depth over the full length of the top surface. Setting the values of *a-f* to match these assumptions (*a,b,d,f* = 0.667; *c,e*, = 1.0) causes the uplift/overturning failure mode to control, with a low failure threshold of about $H/(\gamma' t) = 6$, independent of *w/t* for large values of *w/t*.

Observations of the tests suggested that head applied to the upstream edge of the crest cap was slightly less than full reservoir head, while head applied to the downstream edge was less than critical depth (i.e., supercritical flow at the downstream edge of the crest). A plausible set of coefficients consistent with these observations was constructed, with *a* = 0.95, *b,d,f* = 0.6, and *c,e* = 0.96, and the sliding and uplift/overturning failure thresholds were recomputed (Figure 12) (Values of *b*, *d*, and *f* are made equal because the pressures at these locations are all associated primarily with the depth of flow at the downstream edge of the crest. The values of *c* and *e* are associated with the depth of water in the reservoir approaching the crest and the stagnation of the approach flow against the upstream face of the lift. The value of *a* is associated with the depth of water above the leading edge of the crest, which is slightly reduced from the reservoir head approaching the crest.).

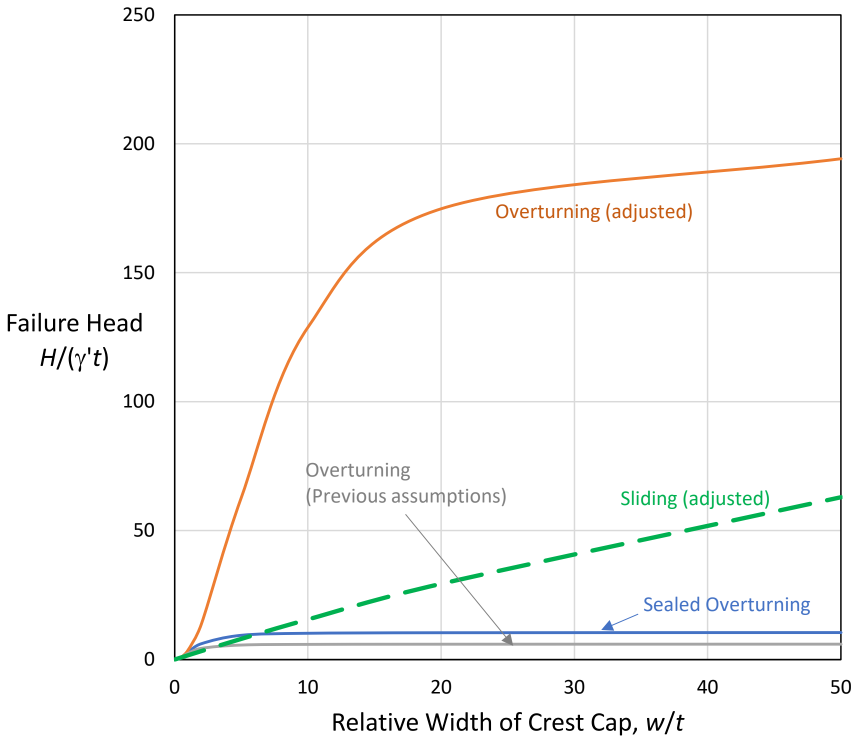


Figure 12: Relative failure thresholds for previously assumed pressure distributions and adjusted pressure distributions. Note the similarity of the adjusted sliding failure threshold and the results shown in Figure 10.

With the adjusted coefficients, the uplift/overturning failure threshold increases dramatically, and the sliding failure mode controls for all but the smallest values of w/t. The predicted failure threshold curve exhibits a linear increase with the w/t ratio, similar to the results of the physical model tests. This result is very sensitive to the values of *a* and *e*, the coefficients defining the pressure distribution at the upstream bottom edge of the slab and the weight of water applied to the upstream top edge. These locations have the longest moment arms and thus have the most influence on the overturning failure mode. Specifically, the result is sensitive to the difference e-a, which must be positive for overturning failure to occur.

To represent the tests conducted with the sealed downstream edge, the value of *f* (pressure applied to bottom face at downstream corner) can be set to 0.96, matching the value of *e* (pressure at the upstream bottom corner). With only this one change, the uplift or overturning failure mode threshold is again quite low, approximately $H/(\gamma' t) = 10$, almost independent of w/t. This is consistent with the results of the physical model tests.

5.2 Sensitivity to Crest Geometry

Most of the testing was performed with crest cap lifts having square upstream edges located so that the upstream corner was aligned with the overall slope of the upstream and downstream dam face. Limited tests were performed in the initial phase of the study (in the narrow flume) using beveled-edge pavers (bevel angle of 1h:2v) , and the beveled edges had little effect.

In the wider flume, limited tests were conducted of square-edged lifts that were intentionally made too narrow for the crest of the dam, so that either the upstream or downstream edge was not aligned with the prevailing slope of the lower body of the dam. Both situations led to failure by sliding at much lower overtopping heads. This is believed to be due to a change in the structure of the vortex that forms in the corner of the step, as illustrated in Figure 13. The vortex typical of the aligned-step condition apparently reduces the fraction of reservoir head that penetrates the upstream seam and may also increase the tendency for uplift pressure to be relieved at the downstream edge of the seam.

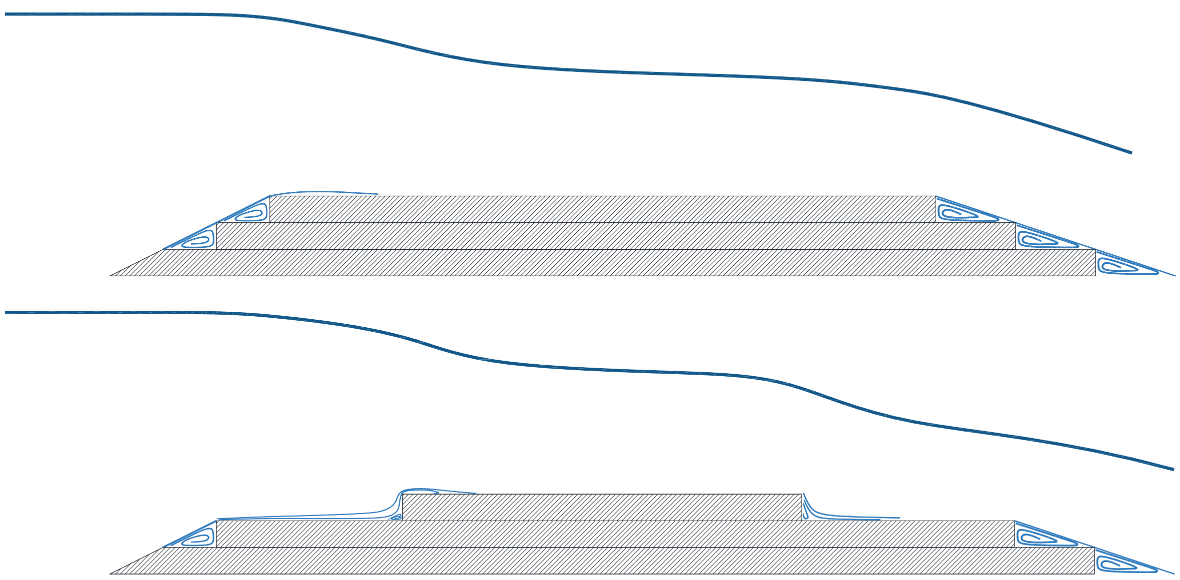


Figure 13: Illustration of changes in flow structure when crest cap lifts are aligned (top) or not aligned (bottom) with the upstream and downstream dam face slopes.

Limited testing (4 repetitions of a configuration with w/t = 7.64) was also performed to evaluate the failure threshold for a single lift of RCC on the crest of the dam, similar to the condition that would exist following failure and removal of the top lift. These tests showed that the threshold head for sliding failure is reduced about 25-30% below the trend line shown in Figure 10. This is believed to be due to the closer proximity of the smooth (non-stepped) upstream face of the dam, which would cause the approach flow to exhibit less boundary layer turbulence and higher near-bed velocity in the approach flow, as compared to the upper of two lifts. The increased relative width of the lower lift compared to the top lift would help to offset this difference so that the threshold reservoir elevation for sliding of the bottom lift might still exceed that of the top lift. This is especially possible when the dam crest is relatively narrow, so that there is a large percentage difference in the widths of the top lift and second lift. It should be noted that immediate sequential failure of both lifts was not observed in any of the laboratory tests that included two lifts.

5.3 Application to Umbarger Dam

The crest-width to lift-thickness ratio for Umbarger Dam is w/t = 25. Figure 10 suggests that an overtopping head of about 30 ft is needed to produce sliding of an intact top lift of the dam.

This is a significantly higher failure threshold than predicted by previous analyses. As discussed, this is probably due to small differences between pressure distributions above and below the slab in the physical model vs. those assumed by the previous analyses. The great value of the physical model is that it overcomes the need to estimate these pressure distributions. If the construction of the model produces similar field conditions as the real-world structure, the model should reliably represent the real-world behavior.

The model tests demonstrated that the greatest threat to stability of the crest cap lifts is the development of uplift pressure within the seam between lifts, without drainage of that pressure at the downstream edge. The loose stacking of blocks in the model is believed to represent the unbonded condition that has been observed in the field, presumably caused by a combination of poor initial bonding and the long-term effects of freeze-thaw cycles as water penetrates the dam crest during winter months. As long as no special effort was made to seal the downstream edge during original construction, it seems likely that water can enter the upstream side of the seam between lifts as readily as it can drain through the downstream edge. This should produce a pressure distribution similar to that occurring in the model tests.

The failure of a lift that is compromised by longitudinal cracks needs to be considered. It should be kept in mind that the longitudinal cracks simulated here were extremely regular in form compared to the real-world situation.

Testing showed that sealing of the downstream edge of the seam between lifts would greatly reduce the head required to reach the threshold of failure. In contrast, sealing the upstream edge could be an effective way to improve and ensure the stability of the structure.

6 CONCLUSIONS

The physical model tests presented in this report have helped to illustrate the mechanics of potential failure of RCC overlay crest caps. The physical model naturally creates the correct pressure distributions on the top and bottom edges of the RCC slabs, avoiding the need to estimate these pressure distributions when doing an analytical free-body analysis. The use of high-speed video allowed failure modes to be confirmed visually.

The integrity of the RCC crest caps is an important consideration. Potential failure modes are more complex and failure thresholds are lower when the slab is not continuous.

Tests with sealed downstream edges of the RCC crest caps exhibited greatly reduced failure thresholds. An effective treatment for preventing failure of crest caps may be to seal the upstream edge and any cracks on the crest (i.e., sealing against water intrusion like one would maintain a pavement). Additionally, the downstream seam between lifts should be kept open and free draining to prevent the buildup of uplift pressure below the crest cap.

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