

## ANALYSIS OF SPILLWAY FAILURES BY UPLIFT PRESSURE

By Thomas E. Hepler<sup>1</sup>, Associate Member, ASCE, and  
Perry L. Johnson<sup>2</sup>, Member, ASCE

## ABSTRACT

Results of a model study to evaluate the development of uplift pressures beneath concrete chutes are presented. Two spillway failures are described to support the study results. Common design features to control or prevent uplift pressure development are provided.

## INTRODUCTION

High-velocity flow within open channels on steep slopes represents a significant source of energy which must be controlled to provide for the safe conveyance of water. Concrete-lined spillway chutes are commonly provided for dams to safely contain the flow until the energy is dissipated within a downstream stilling basin or plunge pool. Offsets may develop within the concrete linings at joints or cracks as a result of concrete shrinkage, differential settlement, ice pressures, or other loads. Depending upon the magnitude and configuration of the offsets, a portion of the flow may be deflected downward into the opening and produce a stagnation pressure beneath the concrete lining. If this pressure is sufficient to overcome the weight of the lining, the weight of the water within the chute, and any structural strength of the lining and anchorage, the lining will be displaced further and structural failure may result. The loss of a single floor slab on a steep slope may result in the successive loss of adjoining slabs upstream and eventual loss of the spillway crest and impounded reservoir, with potentially catastrophic results.

Although numerous design features have become standard practice for the control or prevention of uplift pressure development within spillways, older structures not provided with such features may represent a dam safety concern requiring modification. In the case of an existing spillway which has never operated, analysis methods are necessary to predict the potential for uplift pressure development and subsequent failure.

---

<sup>1</sup>Civil Engineer, Division of Dam and Waterway Design, U.S. Bureau of Reclamation, Denver, CO 80225

<sup>2</sup>Hydraulic Engineer, Division of Research and Laboratory Services, USBR, Denver, CO 80225

## HYDRAULIC MODEL STUDY RESULTS

A limited hydraulic model study [Johnson, 1976] was conducted in 1976 to develop uplift design criteria for unreinforced, concrete-lined canal laterals on steep slopes. Uplift pressures that result when flow passes over offsets (joints or cracks with displacement) in the lining were evaluated as a function of offset geometry and flow velocity. Offsets may be parallel to the direction of flow (horizontal offset) and/or may be into or away from the flow (vertical offset) as represented in Figure 1. A portion of the flow passing over the offset will be deflected downward into the joint, particularly for offsets into the flow. Depending on the offset geometry, part of the velocity head of the flow is lost to turbulence while the rest is converted to a stagnation pressure. If the lining-soil interface is sufficiently permeable to maintain high uplift pressures over a wide area, with limited or no pressure relief, an uplift pressure field that could exert sufficient force to lift the lining may develop.

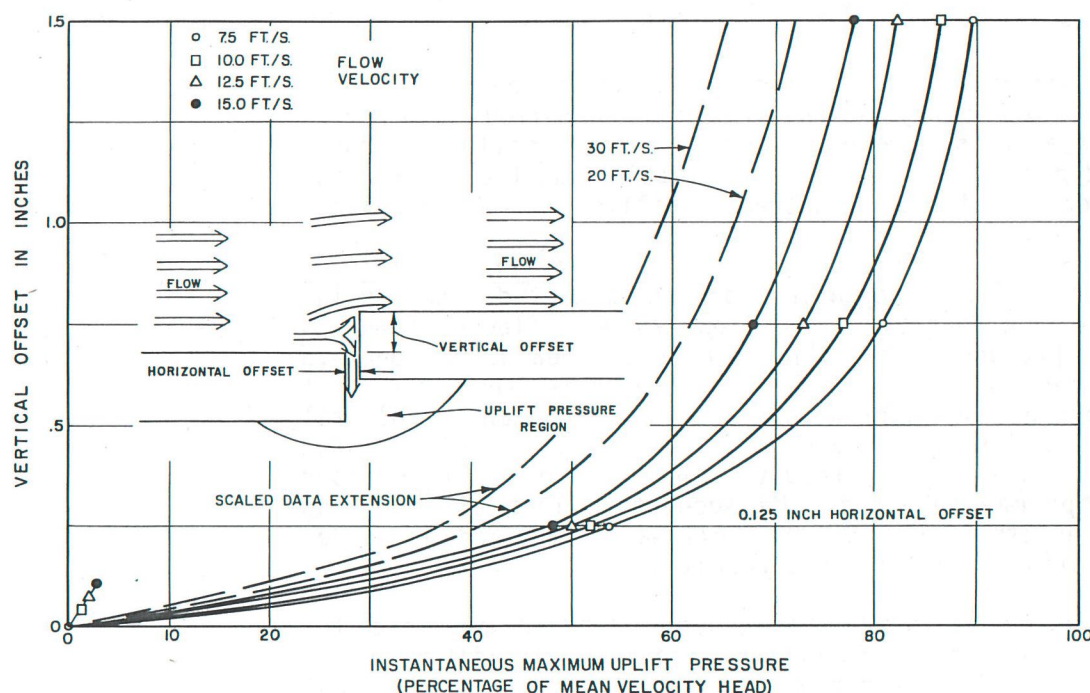


FIGURE 1 - TYPICAL MODEL STUDY FINDINGS

The study was conducted in a 6-inch (150 mm) wide flume with an adjustable floor which allowed the establishment of desired offsets. The adjustable flume floor was 2.5 inches (64 mm) thick to represent a 2.5-inch (64 mm) thick canal lining. The opening at the offset led to a watertight pocket beneath the flume floor. Dynamic pressure data were taken from this pocket to determine both average and instantaneous pressures.

The model included several simplifications which affect the data and

## SPILLWAY FAILURES ANALYSIS

859

their interpretation. First, the model was two-dimensional and thus represented only a unit width of a horizontal slab. In addition, only displacements along smooth joints in a plane normal to the flow were studied. In reality displacements could also occur along irregular cracks in planes at an infinite number of possible angles to the flow. Irregularities in offset alignment and surface will develop both increased and decreased uplift pressures. The results presented give general guidance to uplift pressure evaluation; however, field variations should be expected.

Various combinations of flow velocities ranging from 7.3 to 15.4 ft/s (2.2 to 4.7 m/s), horizontal offsets ranging from 0.125 to 1.5 inches (3.2 to 38 mm), and vertical offsets ranging from 0.125 to 1.5 inches (3.2 to 38 mm) were evaluated. These conditions adequately cover potential displacements and flow velocities that would occur in canal laterals. Froude number-based scaling laws have been used to extend the available data to higher velocities that are representative of spillway flows.

Typical study results are shown in Figure 1, with uplift pressures presented as a percentage of the mean velocity head for the flow over the offset. The hydrostatic component was subtracted from the measured pressures to obtain the dynamic uplift pressures shown. Thus, to define total hydraulic loading on a floor lining, hydrostatic loads should be added to both top and bottom surfaces. The dynamic uplift pressures presented in Figure 1 are maximum instantaneous values which yield short term maximum loadings and thus are critical with respect to lining stability.

Several conclusions can be made. Dynamic uplift pressures increase with increase in vertical offset. This is due to boundary layer influences which reduce flow velocities near the boundary and thus reduce the velocities intercepted by smaller offsets. Boundary velocity magnitude and boundary layer thickness are dependent on mean flow velocity, the extent of boundary layer development, flow surface configuration (the presence of vertical curves, contractions, expansions, etc.), and flow surface roughness. Since boundary layer thickness and velocity distribution may vary with location on the flow surface, what constitutes an acceptable offset may also vary with location. The findings indicate that with a 0.125-inch (3.2 mm) horizontal offset, 80 to 90 percent of the mean velocity head will be converted to instantaneous uplift pressure. Average uplift pressure will be less. The size of the horizontal offset also influences the resulting uplift pressure. The 0.125-inch (3.2 mm) horizontal offset (the smallest studied) yielded the largest uplift pressures. Increasing the horizontal offset to 0.25 inches (6.4 mm), 0.5 inches (13 mm), and 1.5 inches (38 mm) reduced the resulting uplift pressures by approximately 10, 20, and 40 percent, respectively. Reduced uplift pressure development appears to result because larger horizontal offsets allow more flow circulation which dissipates energy.

Finally, flow velocity over the offset affects the development of uplift pressure. In general, it appears that higher velocities convert a slightly smaller percentage of the mean velocity head into dynamic uplift pressure. However, because velocity heads increase as the



square of the velocity, the resulting dynamic uplift pressures increase substantially with velocity.

## CASE STUDIES - SPILLWAY FAILURES AT BUREAU OF RECLAMATION DAMS

### Dickinson Dam

Dickinson Dam is located on the Heart River near Dickinson, North Dakota, and was constructed for the Bureau of Reclamation from 1949 to 1950. The service spillway is located on the right abutment of the dam, and consists of a 200-foot (61 m) long overflow crest discharging into a rectangular chute and stilling basin. The spillway operated satisfactorily in each of the first three years after construction, discharging a maximum of 3,200 ft<sup>3</sup>/s (90 m<sup>3</sup>/s) in 1952.

In April 1954, the spillway failed while discharging a maximum 4,000 ft<sup>3</sup>/s (110 m<sup>3</sup>/s). Within 24 hours, four of the six concrete slabs immediately downstream from the spillway crest cracked near the middle along a transverse cutoff wall and rose above the flow at the upstream end, using the longitudinal reinforcement in the top face as a hinge. The flow beneath the failed slabs eroded the soft sandstone-shale foundation until the transverse cutoff wall was released and the slabs floated downstream. Subsequent inspection of the damage revealed foundation erosion to maximum depths ranging from 6 to 12 feet (1.8 to 3.6 m). Portions of the gravel filter blanket beneath the intact slabs were found to be frozen solid.

The chute slabs were 15 inches (380 mm) thick and underlain by a 12-inch (300 mm) sand-gravel filter blanket containing clay pipe drains. Shallow concrete cutoffs were provided along longitudinal and transverse contraction joints. Foundation grouting, anchor bars, and waterstops were not provided. Subfreezing temperatures and inadequate insulation resulted in loss of the underdrain system by freezing. An offset into the flow along the transverse joint between the crest and the chute could have resulted from ice pressures beneath the chute slab and/or by differential settlement of the heavier crest. Open joints could have resulted from shrinkage of the concrete within both the crest and chute slabs. The absence of waterstops would have allowed spillway flows to enter the open joints, with resistance to uplift pressures provided only by overlying dead loads. Based on the estimated loads and structural capacity of the slab, an added uplift pressure head equal to approximately one-third of the mean velocity head would have been required to initiate the observed failure. With an average flow velocity of 21 ft/s (6.4 m/s) and an assumed horizontal offset of 0.125 inches (3.2 mm), a vertical offset into the flow of only 0.2 inches (5 mm) would have been sufficient to produce the additional uplift required, using the results shown in Figure 1.

### Big Sandy Dam

Big Sandy Dam is located on the Big Sandy River, approximately 45 miles (72 km) north of Rock Springs in southwestern Wyoming. The spillway, located on the right abutment, consisted of a 170-foot (52 m) long side-channel overflow crest discharging into a trapezoidal, concrete-

## SPILLWAY FAILURES ANALYSIS

861

lined chute and stilling basin. Final design drawings for the spillway were prepared by the Bureau of Reclamation in 1950, and construction was completed in 1952. The designs and construction generally followed the acceptable practice at that time, and included pressure grouting along the crest, anchor bars, and underdrains.

Soon after construction, reservoir seepage and extremely low winter temperatures began to take their toll on the spillway structure. By 1958, serious cracking had developed in the concrete side walls of the stilling basin and adjoining portions of the chute, where pipe drains had not been provided. To improve drainage in these areas and to minimize further damage, weep holes were drilled through the concrete in areas of distress. By 1960, the concrete side walls immediately below the massive overflow crest had rotated outward, producing significant concrete spalling along the contact with the crest. As was the case in the stilling basin and chute, freezing seepage and resulting ice pressures were blamed.

Despite the extensive damage, the spillway operated without incident between 1957 and 1983. Spillway discharges never exceeded  $500 \text{ ft}^3/\text{s}$  ( $14 \text{ m}^3/\text{s}$ ), far below the rated capacity of  $7,350 \text{ ft}^3/\text{s}$  ( $208 \text{ m}^3/\text{s}$ ). During spillway operation, the single outlet for the chute drains above station  $4 + 66$  would flow full, and water jets would appear from weep holes drilled through the chute floor slab between stations  $4 + 66$  and  $4 + 85$ . This indicated relatively free access of water to the foundation, through joints in the floor, during spillway discharges [USBR, 1981]. The slab finally failed by uplift pressure in June 1983, while discharging  $400 \text{ ft}^3/\text{s}$  ( $11 \text{ m}^3/\text{s}$ ).

Photographs of the concrete slab prior to failure reveal a partially spalled and open joint at the upstream end (station  $4 + 66$ ) and the presence of seepage flow beneath the slab exiting at the downstream end (station  $4 + 85$ ). One month prior to failure, a partial offset into the flow could be seen at the upstream joint, as evidenced by a noticeable disturbance within the minor flow of  $10 \text{ ft}^3/\text{s}$  ( $0.3 \text{ m}^3/\text{s}$ ). During failure, the chute drain outlet ceased to flow, indicating loss of the clay pipe at station  $4 + 66$ . A spectacular "rooster-tail" developed at the upstream end, sending spray high above the level of the side walls. After the spillway flows ended, the stilling basin was unwatered for inspection of the damage. The concrete slab between stations  $4 + 66$  and  $4 + 85$ , with an average thickness of 15 inches (380 mm), had lifted up to 2 feet (0.6 m) off the foundation and was supported by debris at the downstream end. The 1-inch (25 mm) anchor bars, spaced on 5-foot (1.5 m) centers and 4 feet (1.2 m) long, had been pulled out of the soft sandstone foundation, with little evidence of the original grout encasement. Total erosion of the slab foundation was estimated at 10 to  $20 \text{ yd}^3$  (8 to  $15 \text{ m}^3$ ), with depths below grade up to 1 foot (0.3 m).

Based on the estimated loads and the ultimate capacity of the anchor bars, assumed at 10,000 pounds (44,000 N) each, an added uplift pressure head equal to 87 percent of the mean velocity head would have been required to cause the observed failure. Using the model study results, such an uplift would not be expected for the average flow velocity of  $31 \text{ ft/s}$  ( $9.4 \text{ m/s}$ ). Assuming the anchor bars were only 50 percent



effective due to deterioration of the grout-foundation contact, an added uplift pressure head of 49 percent would have been necessary, corresponding to an assumed horizontal offset of 0.125 inches (3.2 mm) and a vertical offset of approximately 0.5 inches (13 mm). Such a combination of offsets is considered reasonable for the slab that failed.

#### UPLIFT CONTROL FEATURES RECOMMENDED FOR DESIGN

In both cases of uplift failure at Bureau spillways, freezing action in the foundation was a major contributing factor, and foundation erosion continued after failure of the structure. The design of spillways today must include features which will prevent these damaging effects of water. Foundation seepage may be minimized by pressure grouting, and controlled by an underdrain system consisting of perforated pipes within a graded sand and gravel filter. In cold climates, rigid foam insulation should be provided between the concrete slab and the underdrains to prevent freezing. Embedded waterstops in floor joints will prevent the flow of water through the joints to the foundation. Longitudinal reinforcement and transverse cutoffs at joints will prevent relative vertical displacement. Finally, the structure must be designed to resist the maximum uplift pressure anticipated after consideration of all other design features. A design uplift head of 10 feet (3.0 m) for chutes on rock foundations has been recommended [Austin, et al, 1971] unless higher values are anticipated. Foundation anchors are sized to provide the additional uplift resistance necessary after the effective weights of the concrete, backfill, and contained water are considered.

#### CONCLUSIONS

A relationship has been developed between flow velocity, offset geometry, and the resulting uplift pressure in spillway chutes and canal laterals, based on the results obtained from a two-dimensional hydraulic model. Maximum uplift pressures of up to 90 percent of the average flow velocity head (adjusted for the flow depth in the channel) have been recorded. Spillway failures at Dickinson and Big Sandy Dams caused by uplift pressure demonstrate the need for adequate design features to control or prevent the development of these pressures.

#### APPENDIX - REFERENCES

- Austin, G. H., Ruffatti, M. J., and Jones, C. W. (1971), "Value Engineering Study of Uplift Control on Spillways for Dams," USBR, Denver, Colorado.
- Johnson, P. L. (1976), "Research into Uplift on Steep Chute Lateral Linings," USBR, Denver, Colorado.
- "SEED (Safety Evaluation of Existing Dams) Report for Big Sandy Dam, Eden Project, Wyoming," (1981), USBR, Denver, Colorado.

# HYDRAULIC ENGINEERING

Proceedings of the 1988 National Conference  
sponsored by the  
Hydraulics Division of the American Society  
of Civil Engineers

Hosted by the Colorado Section, ASCE

Clarion Hotel and Conference Center  
Colorado Springs, Colorado  
August 8-12, 1988

Edited by Steven R. Abt and Johannes Gessler



Published by the  
American Society of Civil Engineers  
345 East 47th Street  
New York, New York 10017-2398

THE UNIVERSITY OF CHICAGO  
LIBRARY

1964

1

1964

1964

1964

1964

1964

1964

1964

1964

1964

1964

1964

1964

1964

1964