Numerical Modeling to Predict Canal Breach Outflow Hydrographs

Tony L. Wahl, Member[1], ASCE

ABSTRACT

Numerical unsteady-flow simulations were used to investigate the breach outflow hydrographs produced by hypothetical canal embankment failures, considering the interaction of canal hydrodynamic effects and the rate of breach development. The studies led to the development of appraisal-level procedures for estimating the peak outflow rate and other breach outflow hydrograph characteristics as a function of canal hydraulic properties, breach development rate, the length of the affected canal reach, and the location of the breach relative to upstream and downstream check structures. These procedures will help water managers identify canal reaches and specific breach locations that have potential to produce large peak outflow rates.

INTRODUCTION

The Bureau of Reclamation has constructed more than 8,000 miles of irrigation water delivery canals since 1902. Although typically reliable, canal failures have occurred on occasion throughout Reclamation's history. Threats to canals include animal burrows, tree roots, embankment and foundation issues, pipe penetrations, seismic events, internal erosion under static loading, hydrologic events, and operational incidents. Canal failures can have significant consequences, and potential consequences are increasing as urban development surrounds formerly rural canals.

To understand the risks associated with individual canals, modeling of potential failures is needed. Tools for predicting peak outflow from traditional embankment dams do not account for the upstream hydraulic boundary conditions imposed by a canal of finite cross section and volume. The ability of a canal to convey water to the site of a breach limits the potential peak outflow.

To gain a better understanding of canal breach processes and develop guidance and tools for evaluating flooding risks associated with potential canal breaches, Reclamation has carried out scale model canal breach tests in the hydraulics laboratory (Wahl and Lentz 2010). These tests demonstrated that breach development rates are related to measurable soil erodibility parameters. However, due to space limitations in the laboratory, the physical model tests could not evaluate the interaction between canal hydrodynamics and the rate of breach development. Numerical modeling was undertaken to examine this relationship and develop predictive tools for estimating peak canal breach outflow.

[1] Hydraulic Engineer, U.S. Bureau of Reclamation, Hydraulic Investigations and Laboratory Services Group, Denver, CO, twahl@usbr.gov

BREACH BOUNDARY CONDITIONS IMPOSED BY CANALS

The most important questions to answer regarding the potential failure of a canal embankment are the magnitude and duration of the peak outflow from the breach. In the hydraulics laboratory, Reclamation's testing simulated canal embankment breach development with a nearly constant upstream water level maintained in the canal. This is realistic if a breach develops rapidly within a long canal reach. However, in most real situations, the breach may be located close to check structures on the upstream or downstream legs of the canal that could be shut down in the event of breach, or might limit flow toward the breach even if they are not shut down by canal operators. Also, the canal cross section will limit the flow toward the breach once the breach becomes large enough to convey flow out of the canal at a rate that allows critical flow conditions to develop in the canals leading to the breach site.

Figure 1. — Canal breach test demonstrating breach outflow that is limited by critical-flow conditions in both legs of the canal leading to the breach. Flow in the canal prior to the breach was from right to left.

EXAMPLE CANAL

To evaluate the effects of nearby check structures and the conveyance capacity of the canal on peak breach outflow, a series of numerical modeling experiments was conducted for an arbitrary example canal, varying the breach development time and the reach length and location of breach along the canal length. The modeled canal had the following properties:

- Trapezoidal section, base width = 24 ft, side slopes = 1.5H:1V,
- Bed slope = 0.325 ft/mile,
- Design discharge, Q = 3,000 ft^3/s,
- Manning's n=0.014, normal flow depth y_n = 16.4 ft.

To make the results applicable to all canals regardless of scale, results were normalized with respect to the flow rate that would occur if the entire specific energy of the flow at normal depth could be instantaneously applied to creating critical flow conditions in each leg of the canal leading to the breach site. This is only a theoretical construct, since reversal of the flow in the downstream leg of the canal would be required and would consume some of the initial specific energy of the flow. In the upstream leg of the canal, energy losses and momentum changes would also be required to accelerate the flow from its normal-depth velocity to the new critical-depth velocity.

To demonstrate this concept using the data for the arbitrary canal, the specific energy of the canal at normal depth is $E=y_n+V^2/2g$ = 16.63 ft, where y_n is the normal depth of flow and g is the acceleration due to gravity. For critical flow the Froude number is $\mathsf{Fr} = V/(gD)^{1/2} = 1$, where D is the hydraulic depth A/T, A is the cross-sectional area of the channel, and T is the top width. Combining these equations we can solve iteratively to find that a critical depth of 12.33 ft has the same specific energy as the normal depth flow. The associated critical discharge is 8,721 ft^3/s. This is the flow rate that would occur in the upstream reach of the canal if there were an instantaneous failure of the canal bank that was large enough that the breach opening did not control the flow; the flow would be controlled instead by a critical section in the canal. If both the upstream and downstream canal legs delivered this flow to the breach site, the maximum peak breach outflow would be 17,442 ft^3/s. This quantity will be referred to as $Q_{c,\max}$ for the remainder of this paper.

HEC-RAS MODELING

To determine how canal and breach properties affect the dynamic response of a canal, a numerical model of a canal experiencing a breach failure was created using the HEC-RAS modeling suite (Hydrologic Engineering Center 2010). The model takes advantage of unsteady flow modeling and breach simulation capability to determine breach outflow hydrographs and the dynamic response of the canal reaches. HEC-RAS does not simulate actual embankment erosion processes, but does allow the simulation of different breach development rates through the selection of a total breach development time and geometric parameters describing the ultimate breach size and shape.

The basic configuration of the HEC-RAS models was a single HEC-RAS river reach varying from 2 to 100 miles in length. Within a short 400-ft reach of the canal located at the midpoint of the reach, one side of the canal was defined to be a lateral structure that would be breached at varying rates. HEC-RAS provides an option for the flow through this breached lateral structure to "leave" the model without the need for defining any other river channel reach to carry the flow away. Thus, the breach outflow is unaffected by any downstream tailwater that would be caused by the breach outflow. This is a worst-case scenario appropriate to the situation of the breach of a canal constructed as a fill section above the surrounding topography. The location of the breach and the lengths of the canal upstream and downstream from the breach site were varied to produce simulations with the following length characteristics:

- 1 mile of canal upstream and downstream from the breach site,
- 2 miles of canal upstream and downstream from the breach site,
- 2 miles of canal upstream and 50 miles of canal downstream, and
- 50 miles of canal upstream and downstream.

INITIAL AND BOUNDARY CONDITIONS

The modeled canal used the same cross section and channel slope as the example canal used in the previous discussion. Starting conditions for the simulations were normal-depth flow throughout the model at a flow rate of 3,000 ft^3/s. The boundary condition at the downstream end of the model was a normal-depth flow condition at a hydraulic gradient of 0.325 ft/mile. The boundary condition at the upstream end was a constant inflow of 3,000 ft^3/s, consistent with an assumption that canal operators are not able to immediately react to the breach as it is occurring (because the breach happens too quickly, there are no eyewitnesses and no remote indications of a breach, or remote indications are not immediately acted upon).

Neither boundary condition is perfectly realistic. The upstream and downstream canals would most likely terminate at gated check structures in real cases. At the downstream end, there is eventually a possibility for reverse-flow into the canal reach if the check structure is not closed down, and this reverse-flow would be controlled by an appropriate rating curve for the check structure and its gates. However, until the effects of the breach propagate down to the downstream boundary, the normal-depth outflow is a relatively accurate boundary condition. At the upstream boundary, a constant inflow is realistic until the water surface profile below the upstream check structure starts to drop. At this point, flow would increase through this check structure if it has not been closed down, and this increase would be controlled by the discharge rating curve of the check structure and its gates. Modeling either of the boundary conditions more accurately in HEC-RAS would require specification of details of the check structures and modeling of the adjacent canal pools.

BREACH DEFINITION

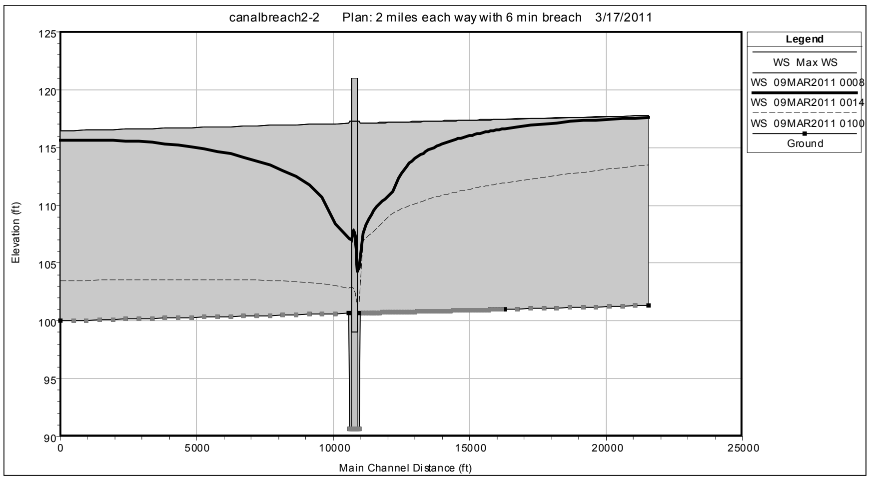
The lateral structure in the model was defined to be 10 ft deeper than the elevation of the canal invert, in an attempt to create geometry that would allow the complete draining of the canal. This is also similar to many field observations of canal breaches in which the breach invert is lower than the original canal invert, again consistent with the situation of a canal elevated above its surroundings. This was only partially effective, as it was found that breaches defined to extend to much deeper elevations captured too much of the canal flow and caused cross sections of the model in the immediate vicinity of the breach to go dry, producing model instability. Ultimately, for stable model behavior it was determined that breach openings could extend only about 1 to 2 ft lower than the canal invert. This sustained a small upstream canal flow past the breach into the downstream canal reach and prevented the model from going dry at any cross section. More sophisticated 2D modeling tools might handle this situation in a better way, but the HEC-RAS model was sufficient for the purposes of this study.

The breach initiation mechanism was selected to be a piping failure growing linearly from zero to 200 ft wide. The pipe was assumed to form initially at a point near the canal invert elevation and enlarge vertically to the top of the embankment and down to the determined minimum elevation, 1 to 2 ft lower than the canal invert as described previously. The width of the breach was selected to be large enough that the breach at its full width would capture the great majority of the flow from the upstream canal and would not be the hydraulic control on the outflow. Critical-flow calculations through a rectangular breach opening suggested that control would shift from the breach opening to the canals when the breach width exceeded about 85 ft.

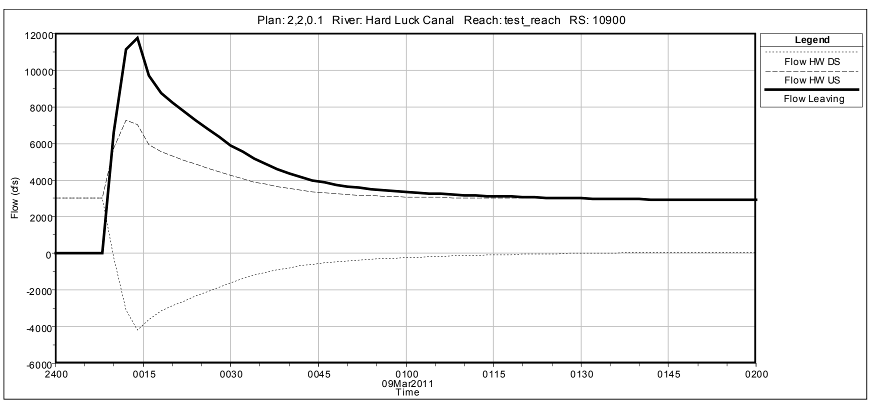
The wider 200 ft width was selected to ensure that outflows would not be sensitive to the breach size, and this was confirmed with subsequent testing of the HEC-RAS model. Breach development times varying from 15 seconds up to 6 hours were used, and breaches were initiated after about 8 minutes of steady, normal-depth flow simulation. The fastest of these breach development times are clearly unrealistic for most canals, but they were selected to allow for the best possible definition of a relationship between peak breach outflow and breach development time over a wide range of conditions.

RESULTS

Figure 2 shows typical results of a model run, with the breach occurring halfway between check structures located 4 miles apart. The breach development time for this case is 6 minutes, a relatively fast breach. The three water surface profiles in Figure 2(a) show the start of the simulation, the peak outflow from the breach at t=14 min, and t=60 min, respectively. Figure 2(b) shows the outflow hydrograph (Flow



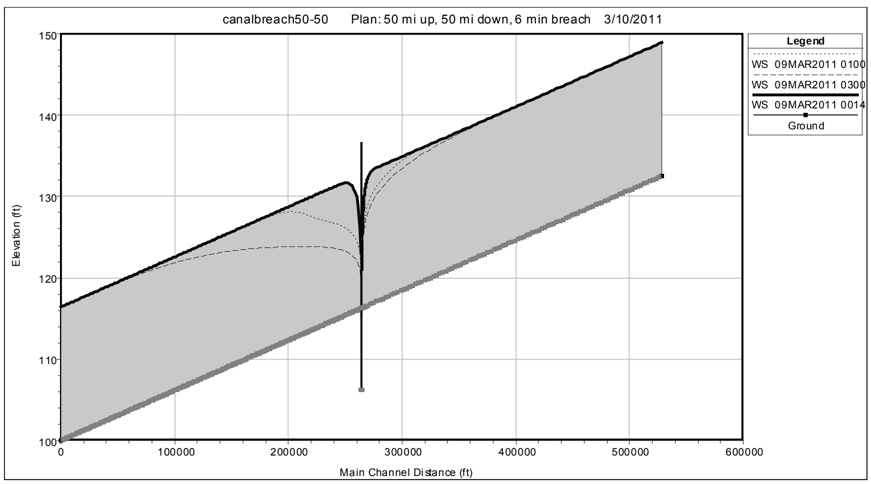
(a)



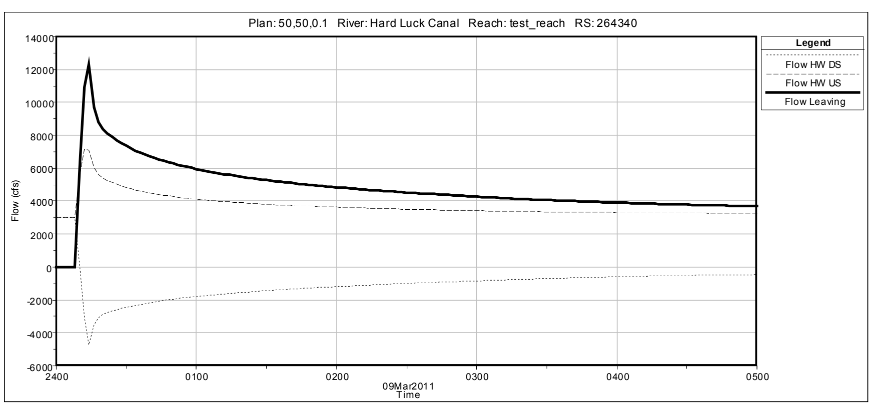
(b)

Figure 2. —HEC-RAS simulation of 6 min breach of example canal embankment with 2 miles distance to nearest upstream and downstream check structures. In (a), initial canal flow is from right to left. In (b), FLOW HW US and FLOW HW DS are the flow rates at the cross sections immediately upstream and downstream from the lateral structure that contains the breach.

Leaving), and the flow hydrographs at the canal cross sections immediately upstream and downstream from the breach. The peak breach outflow is about 11,800 ft^3/s, but reduces back to about 6,000 ft^3/s at time 30 min. Reverse flow exceeding 4,000 ft^3/s occurs in the downstream canal, and the peak flow rate in the upstream canal is about 7,300 ft^3/s. Notable features of this simulation are the transition from the very steep initial water surface profile in the upstream canal toward the 3,000 ft^3/s M2 profile after an extended period of time, and the significant draining of the downstream canal reach. This draining would be dependent on actions being taken at the downstream check structure to prevent reverse flow. The downstream boundary condition of normal depth flow is not fully realistic. A more realistic boundary condition would be a rating curve for the check structure, which would include the possibility for reverse flow if the canal water level declined rapidly enough to create a reverse head differential at the check structure.



(a)

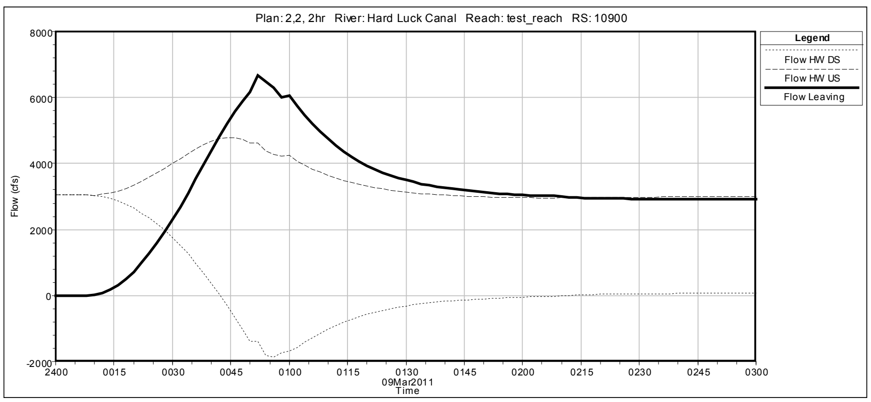


(b)

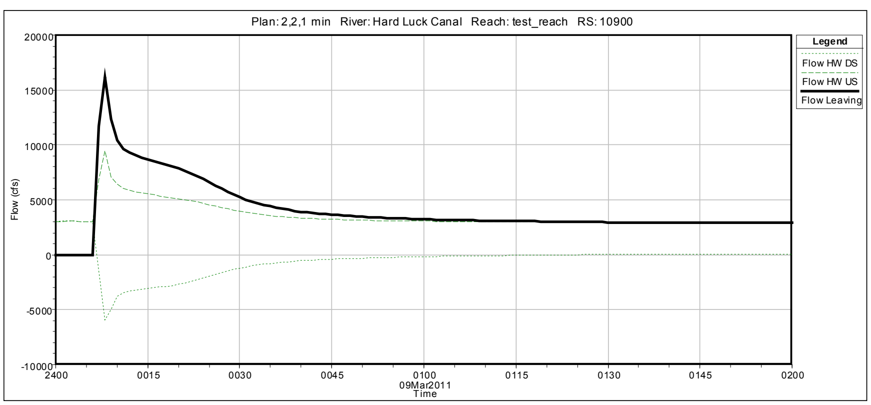
Figure 3. —HEC-RAS simulation of 6 min breach of example canal embankment with 50 miles distance to nearest upstream and downstream check structures. In (a), initial canal flow is from right to left.

Figure 3 shows the behavior with much longer 50-mile canal reaches upstream and downstream from the breach site. The breach development time is still 6 min, and the peak outflow rate is about the same, but the larger volume of water available in the longer canal reaches leads to a much slower decline of the breach outflow hydrograph. Behavior of the upstream canal is very similar in this case to the previous scenario. The downstream canal behavior is practically unaffected by the downstream boundary condition, since the 50 mile distance to the downstream check structure is so great that the effects of the breach on the water surface profile have not yet reached the downstream end of the model after a time of 1 hr.

Figure 4 demonstrates the effects of faster and slower breach development times with the breach located halfway between two check structures that are 4 miles apart (similar canal reach lengths as Figure 2). With a breach development time of 2 hr the peak breach outflow is reduced to about 6,700 ft^3/s occurring about 44 min after the start of the breach process (before the breach is fully formed), while a 1 min breach produces a peak outflow of 16,200 ft^3/s. This latter value is approaching the theoretical upper limit that we previously calculated to be $Q_{c,\max}$=17,442 ft^3/s.



(a)



(b)

Figure 4. —Differences in simulated breach outflow for breach development times of (a) 2 hr and (b) 1 min. The breaches are located at the midpoint of a 4-mile long canal reach.

Table 1 shows results of all HEC-RAS simulations for the example canal. The times shown from the peak outflow condition to 50% recession are the elapsed times from the peak outflow to the time at which the flow rate drops halfway back to the long-term equilibrium flow in the canal. For example, if the normal discharge in the canal was 3,000 ft^3/s and the peak breach outflow was 15,000 ft^3/s (a 12,000 ft^3/s increase), the 50% recession point would be reached when the breach outflow receded back by 6,000 ft^3/s to a flow rate of 9,000 ft^3/s. The table shows that peak outflow rates are not affected by the length of the upstream canal and are also relatively insensitive to the length of the downstream canal. Peak outflow is very sensitive to changes in the breach development time. The hydrograph recession time exhibits a more complex behavior. It is clearly sensitive to the breach development time with fast breaches exhibiting a rapid drop in outflow after the peak breach flow occurs. Sensitivity to the upstream and downstream reach length seems to be dependent on the breach development time. Breaches that develop slowly have outflow hydrographs whose recession limb duration increases with reach length, while breaches that develop quickly recede quickly even when the reach length is large. These observations of the effect of canal reach length can also be extended to the effects of operational responses at check structures, since the closing of a check structure effectively truncates the canal reach length. The intuitive result is that the value of an operational response is greatest when a breach develops slowly and the reach length is short.

Table 1. — Characteristics of breach outflow hydrographs for HEC-RAS simulations of the breach of the example canal.

Reach Length		Breach development time	Peak outflow Q_{peak}	Time from peak to 50% recession
upstream	Downstream			
Mi	*mi*	*min*	*ft³/s*	*min*
1	1	6	11,100	4
1	1	120	5,500	12
2	2	0.25	16,100	4
2	2	1	16,200	4
2	2	6	11,800	10
2	2	120	6,700	18
2	2	240	5,200	18
50	2	6	11,800	10
2	50	6	12,300	10
50	50	1	17,400	3
50	50	6	12,300	12
50	50	120	7,900	54
50	50	360	6,200	54

To facilitate development of predictive relations for estimating breach outflow hydrograph parameters, additional HEC-RAS simulations were carried out on a smaller hypothetical canal consisting of a trapezoidal channel with 14 ft bottom width, 3:1 side slopes, design flow rate of 700 ft^3/s, channel slopes varying from 1 to 2 ft per mile, and Manning roughness coefficients of n=0.014 and n=0.024. This range of values provided scenarios with varying normal flow depths and canal Froude numbers at normal-depth conditions varying from 0.19 to 0.44. Five HEC-RAS breach simulations were carried out on the varying forms of this hypothetical canal with upstream and downstream reach lengths also varied from one-eighth mile to 2 miles. The results were consistent with those in Table 1 and helped to fill data gaps, but did not materially change the dimensionless relations that were subsequently developed.

To develop relations applicable to canals of varying sizes and other properties, the following dimensionless parameters were computed for each scenario:

- Dimensionless upstream and downstream canal reach lengths, $L^*_{us}=L_{us}/R_h$ and $L^*_{ds}=L_{ds}/R_h$, where L is the canal reach length, "us" and "ds" indicate upstream and downstream, respectively, and R_h is the hydraulic radius of the canal for normal-depth flow conditions;
- $t^*_f=t_f/t_{ref}$, where t_f is the breach development time and t_{ref} is a reference time scale computed as the hydraulic depth of the canal at normal depth conditions, $D=A/T$, divided by the wave celerity, $(gD)^{0.5}$. This simplifies to $t_{ref} = (D/g)^{0.5}$;
- $Q^*_{peak}=Q_{peak}/Q_{c,max}$, where Q_{peak} is the maximum breach outflow and $Q_{c,max}$ is the sum of the maximum theoretical discharges through the upstream and downstream canal sections when critical flow occurs with a specific energy equal to the specific energy in the canal at normal-depth flow conditions; and
- $t^*_{recession}=t_{recession}/t_f$, where $t_{recession}$ is the recession time defined previously and shown in Table 1.

It should be noted that when calculating the dimensionless times, values of the breach development time, t_f, were adjusted from the values shown in Table 1 to the time that would have been required for the breach width to reach just the point at which hydraulic control shifts from the breach opening to the supplying canals.

Figure 5 shows the relation between dimensionless peak discharge and the dimensionless breach development time. The proposed upper envelope curve indicates the highest peak outflow likely to be obtained for a given breach development time. Data points lying closest to the upper envelope curve are generally those for the cases with very long canal reaches downstream and upstream from the breach site. Data points lying well below the envelope curve are associated with shorter canal reaches.

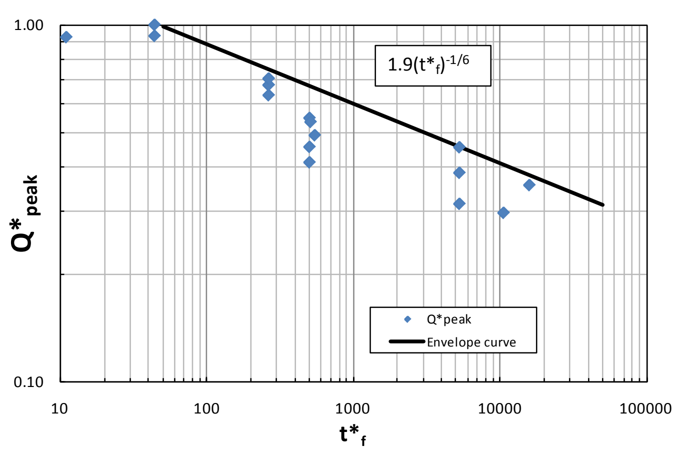


Figure 5. — Dimensionless peak discharge as a function of dimensionless breach development time.

The most important factor affecting the proximity to the envelope curve seems to be the downstream canal reach length, as Table 1 previously showed that the peak outflow is more sensitive to the downstream reach length than the upstream reach length. This is likely due to the fact that until the upstream canal is shut down, it provides water to the breach site at a rate that is not dependent on the reach length (critical flow prevails). In contrast, the downstream canal immediately begins to drain and although critical flow also prevails from that direction, the energy available to drive the critical flow condition diminishes as the downstream reach drains. To quantify the effect of the downstream reach length, for each case the ratio $Q^*_{peak}/Q^*_{envelope}$ was computed, where $Q^*_{envelope}=1.9(t^*_f)^{-1/6}$. Figure 6 shows how this ratio varies as a function of the dimensionless downstream-canal reach length. Again, an upper envelope curve is shown that will allow one to make conservative estimates of the percentage of the peak flow that could be developed in a canal with a specific downstream reach length. The curve envelops most of the simulations carried out for this study and has the desirable properties of tending toward 1.0 for long canal reaches and toward zero when the canal reach length is very short. Clearly there is some scatter in the data, indicating that other factors have some influence, but the canal reach length appears to be a useful predictive parameter. Combining these two relations, the predictive equation for dimensionless peak discharge is

$$Q^*_{\text{peak}} = \frac{1.9}{\left(t^*_{\text{f}}\right)^{1/6}}\left[1 - \frac{1}{\left(L^*_{\text{ds}}\right)^{1/3}}\right] \tag{1}$$

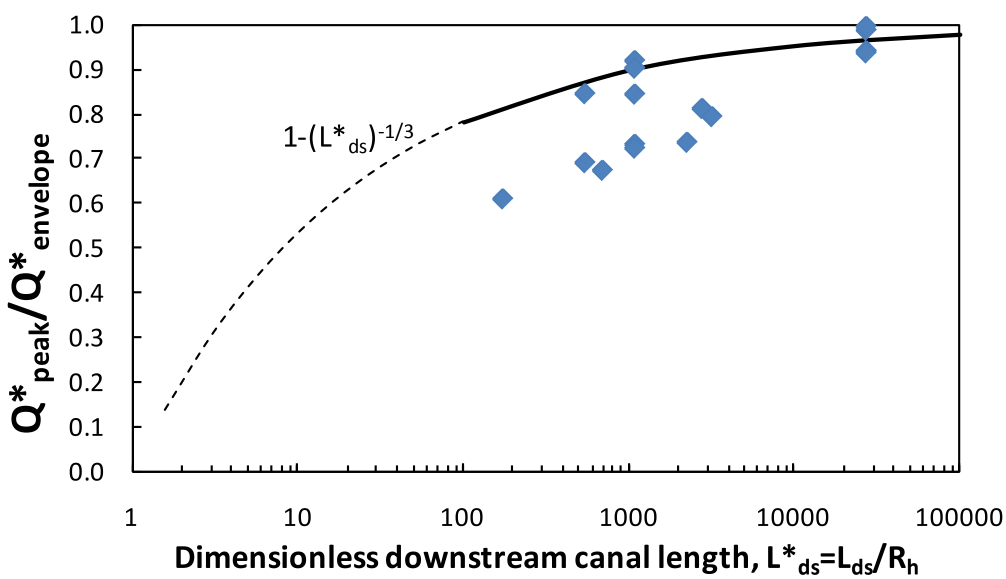


Figure 6. — Effect of downstream canal reach length on peak breach outflow.

The final aspect of a canal breach outflow hydrograph that is of significant interest is the duration of high flows following the peak outflow. Assuming no operational response that shortens the duration, the most useful predictive relation that could be determined from these simulations is shown in Figure 7. The dimensionless time required for the flow to recede 50% of the way back to the normal canal flow rate is inversely related to the dimensionless breach development time. Again, the figure shows that other factors also affect the recession time, but the relation to the breach development time should be useful for predictive purposes. Note that although the figure shows the dimensionless recession time increasing for rapid failures (small values of t^*_f), since the time reference for the recession time is the breach time itself, the net result is that rapid failures still experience more rapid recessions than do slow failures, which is consistent with the HEC-RAS results shown in Table 1.

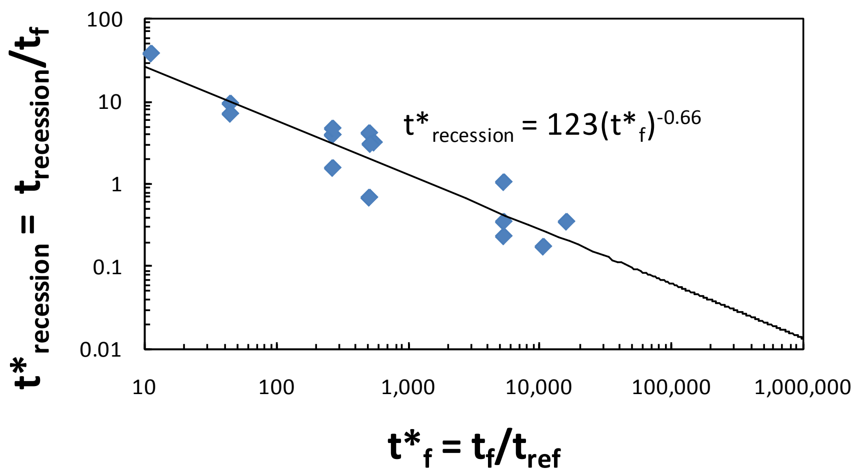


Figure 7. — Hydrograph recession time as a function of breach development time.

CONCLUSIONS

A series of numerical simulations of unsteady flow in an example irrigation canal following breach of the canal embankment were used to develop essential dimensionless relations between peak breach outflow, breach development time, and canal reach length. Utilizing information from physical model tests that relates breach development rates to embankment material erodibility parameters, these relations have the potential to be used for making appraisal-level estimates of peak breach outflow for hypothetical canal breach scenarios.

REFERENCES

Hydrologic Engineering Center, 2010. CPD-68. HEC-RAS River Analysis System, User’s Manual, Version 4.1, January 2010. U.S. Army Corps of Engineers, Davis, CA.

Hydrologic Engineering Center, 2010. CPD-69. HEC-RAS River Analysis System, Hydraulic Reference Manual, Version 4.1, January 2010. U.S. Army Corps of Engineers, Davis, CA.

Wahl, Tony L., and Lentz, Dale J., 2011 . *Physical hydraulic modeling of canal breaches*. Hydraulic Laboratory Report HL-2011-09, U.S. Dept. of the Interior, Bureau of Reclamation, Denver, Colorado, 56 pp.