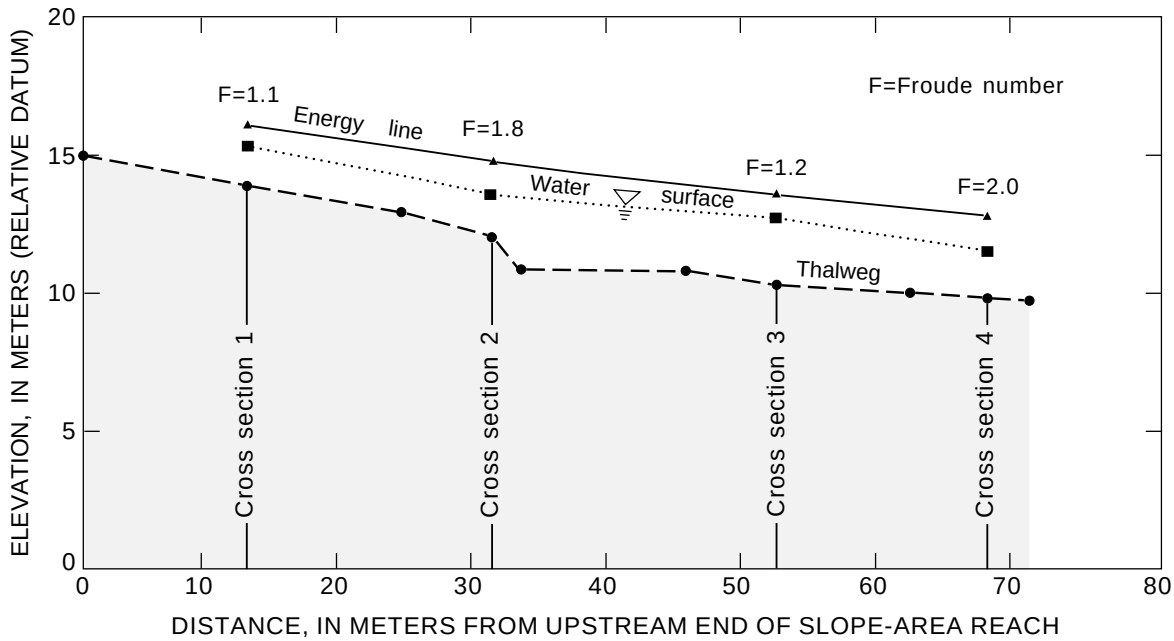

Figure 14. Total-energy diagram for the peak water-flood at Centralia, Wash.

Recent investigations have made some progress in understanding the magnitude of flow resistance of high-gradient channels (for slopes less than 0.04, see Jarrett, 1984; for slopes up to 0.16, see Marcus and others, 1992), and streams with large roughness elements (Hicks and Mason, 1991). These investigations all provide verified estimates of flow resistance in streams with fixed beds, low sediment transport, and relatively small discharges. During outstanding floods in steep channels where stream roughness elements are drowned out by high stages, channel-bed material is mobile, and large amounts of sediment are in transport. Roughness-verification studies conducted under relatively benign conditions may have little significance to large floods such as the Centralia dam-failure flood.

One approach in selecting roughness coefficients for indirect discharge estimates for extraordinary floods is to bracket the likely resistance coefficient by computing resistance with different equations developed to estimate particular kinds of resistance, and estimate a value based on knowledge of assumed processes occurring during a large flood. Total flow resistance in a river or stream is the sum of many kinds of roughness, including bed and bank resistance, spill resistance, and channel irregularities and curvature (Leopold and others, 1964). In steep streams during normal discharges, form or particle roughness can be represented by the ratio of flow depth to size of the roughness elements, know as relative roughness. The relation of Limerinos (1970) is a widely-recognized method to estimate particle resistance to flow, and as such provides a minimum value for flow resistance in the Centralia flood:

$$n = \frac{(0.1129)\,R^{1/6}}{1.16 + 2.0\log\left(\frac{R}{d_{84}}\right)}; \qquad (1)$$

where R = hydraulic radius = (0.76 m

d_{84} = particle size = (0.239 m)

S = channel slope = 0.09

n = Manning's roughness coefficient

Solving this relationship for n produces a value for of 0.050.

The study by Jarrett (1984) treats Manning's n as a "black-box" in which all the possible forms of flow resistance in high-gradient channels during normal flows are collected into a simple relation involving slope and hydraulic radius:

$$n = 0.32\,(S)^{0.38} R^{-0.16}; \qquad (2)$$

results in a computed n value of 0.13 for the Centralia flood.

The verified flow resistance values in Jarrett's (1984) study were measured in channels with large, immobile roughness elements that produced high spill resistance and large *n*-values. Application of Jarrett's (1984) relation to channels steeper than 0.04 have indicated this method over-predicts flow resistance by an average 32 percent (Marcus and others, 1992). Thus a relation developed for non-mobile beds with large spill resistance is likely to produce *n*-values that are too large, and will define the highest likely values for flow resistance to use in this investigation.

Using the calculated *n* values generated above as upper and lower boundaries for main channel flow resistance ($0.050 < n < 0.13$), field-selected *n* values were used in the final determinations of discharge. Cross-sections are shown in figure 12. Field-selected resistance coefficients, constrained by values determined from empirical investigations, were used in the final discharge calculations (table 1). All cross-sections were subdivided using criteria defined by Benson and Dalrymple (1967). At cross-sections 1 and 2, main channel sections have a large part of the surface area covered by gravel and boulders. A field-selected *n* value of 0.075 was used in the main channel areas, and 0.045 selected for the small overbank areas consisting of bent grass and little or no coarse deposits. At cross-sections 3 and 4, main-channel *n* values were selected to be 0.055, and the small, washed sediment-free overbank areas at the edges of flow were assigned values of 0.045.

Data on ground and water-surface elevation, cross-section geometry, high-water marks, long profiles, and field-selected resistance coefficients were entered into the personal computer version of the U.S. Geological Survey C374 surface water program, following procedures described by Lara and Davidian (1970). The program computes the quantity

[$(1.486/n)\ AR^{2/3}$] (English units), known as conveyance, between each cross-section. Conveyance is converted to discharge by multiplying by $(S)^{1/2}$. Velocity-head is computed for each cross-section from the relation:

$$h_v = \frac{\alpha v^2}{2g} \qquad (3)$$

where α is a velocity-head coefficient that expresses the effect of cross-sectional nonuniformity in the kinetic energy flux, *v* is mean cross-sectional velocity, and *g* is gravitational acceleration. Relative errors in the final computed discharge are probably small because velocity heads do not exceed the water-surface fall, the flow field is not rapidly expanding, and velocity-head coefficients (α) are small (1.01 to 1.15) (Kirby, 1987).

The discharge estimates among different cross-sections are mutually consistent (spread is small), and a value of 71 m^3/s is a reasonable value for the peak water-flood discharge. This discharge estimate should be considered fair (a 15 per cent possible error). Scour and deposition, steep slope, and difficulties in estimating roughness coefficients all contribute to some uncertainty in the final discharge estimate. Froude numbers at all cross-sections are greater than 1.0, indicating supercritical flow. A total energy diagram for the flood is shown in figure 14. The channel thalweg has an irregular profile because of scour and a headcut that developed during the flood. The high-water profile is more regular, and the total energy grade line is quite smooth. True energy slope from this profile is 0.075, compared to a channel slope of 0.09, and a water-surface slope of 0.089. Flow entering the slope-area reach is supercritical (Froude number of 1.1), and remains supercritical through the reach. This result conflicts with the conclusion that supercritical flow may occur over only short distances (less than 8 m) in high-gradient channels, and then is forced to change back to subcritical flow because of extreme energy dissipation (Trieste, 1992).

The estimated peak discharge can be checked against the simplified slope-area method developed by Riggs (1976), and from reports of the draining time of the failed reservoirs. Using data from flow-resistance verification studies, Riggs (1976) found that there is a strong relation between water-surface and flow resistance. If slope can be a surrogate for flow resistance, *n*, and cross-sectional area is closely related to hydraulic radius, Riggs (1976) developed the relationship (English units):

$$\log Q = 0.366 + 1.33 \log A + 0.05 \log S - 0.056\,(\log S)^2;$$

$$Q = 2400\ ft^3/s \text{ or } 68\ m^3/s \qquad (4)$$

where A is cross-sectional area, and S is water-surface slope. This value is similar to the slope-area discharge of 71 m^3/s. An official of the City of Centralia, responsible for the operation of the reservoirs, reported that Reservoir Number 3 drained "in three to five minutes". At a constant discharge rate of 71 m^3/s (the reconstructed flood peak discharge), it would

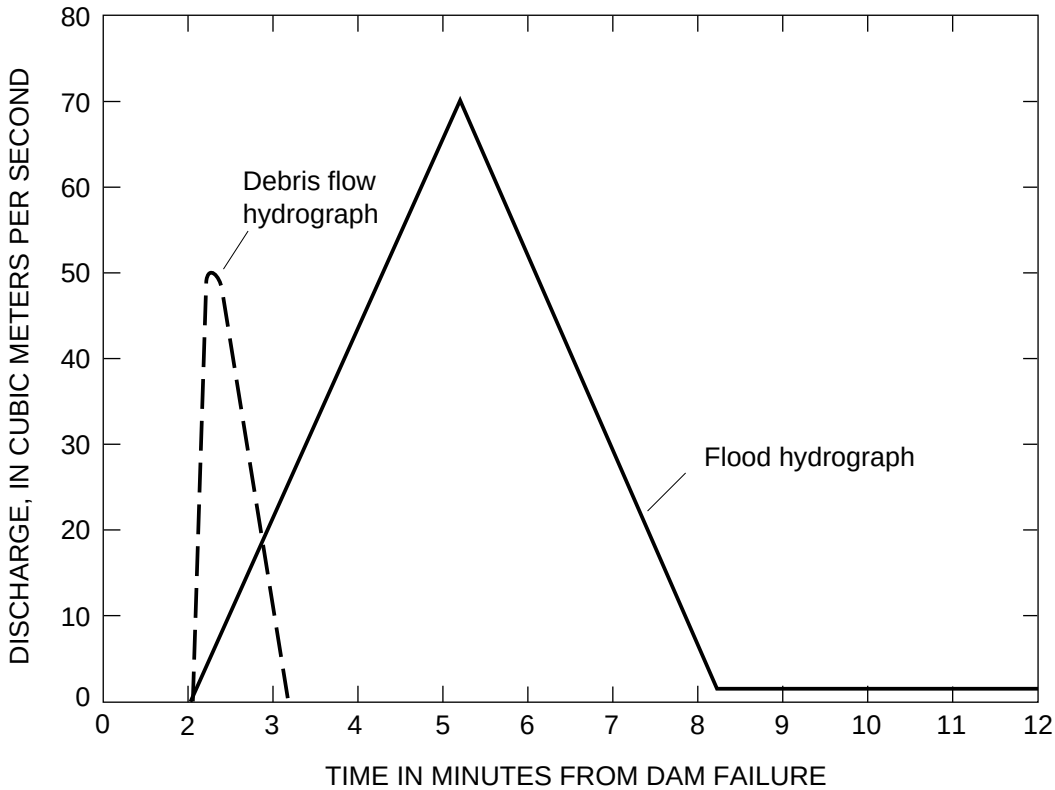

Figure 15. Reconstructed hydrographs of the Centralia debris flow and water-flood.

take 3.1 minutes to drain 13,250 m^3 of water from the reservoir. The reported draining rate of the reservoir is also consistent with a peak-discharge estimate of 71 m^3/s.

Flood and Debris-flow Hydrographs

Several pieces of data about the dam-failure and resulting flood, such as reservoir volume, reports of drainage time, peak discharge calculations, and average velocity of the flood, allow construction of a flood hydrograph, and a speculative reconstructed hydrograph of the debris flow (fig. 15). The peak discharge of the water flow was 71 m^3/s, and the volume of water in the reservoir was 13,250 m^3. Using the average velocity of the flood through the slope area reach (4.2 m/s), it would take 1.1 minutes for the flood to travel 275 meters from the reservoir to the measurement site. If a triangular-shaped hydrograph is assumed, the area under the curve is the reservoir volume, and the base of the hydrograph, or duration of the flood past the slope-area site, would be 6.22 minutes. After 7.3 minutes from the time of the reservoir failure, the flood peak had passed the indirect-discharge measurement site, and moved into the city. The 41-cm pipe in Reservoir Number 4 that was broken during the landslide would have contributed a small "base-flow" to the flood hydrograph, and probably continued for several hours after the flood wave had passed. If the velocity through the pipe is assumed to have been between 10 and 15 m/s, it would have taken between 2.8 and 4.2 hours for the 18,900 m^3 of water to drain from the second reservoir. This is consistent with witness reports that Reservoir Number 4 drained "over several hours".

The debris-flow hydrograph is more speculative. It would take only a short time for the water flowing across the fill, residuum, and bedrock to incorporate enough sediment to become a debris flow (typically 60 percent sediment or more by volume). Average velocities of debris flows in small, steep, vegetated basins are similar to flash-flood average velocities (Costa, 1984; 1987). Flood high-water marks are about 0.25-0.35 meter higher than the tops of presumed debris-flow landforms and deposits. Washed and strongly imbricated gravels and boulders lie on the floodplain in the area where they were originally

deposited by the debris flow, and the landforms and deposits of the debris flow are cut by water-eroded channels and chutes. Thus the peak discharge of the debris flow was less than that of the water- flood, arrived before the water flood peak, and receded before the water-flood wave passed.

I assume that the debris flow peak discharge was 50 m^3/s, or about 70 percent of the water-flood peak discharge. Debris-flow deposits reached about 80 percent of the height of the water-flood high-water marks, and inundated about 2/3 of the area swept by the water flood. Flow velocities are assumed to have been similar. Using the above information, a tentative debris-flow hydrograph is plotted as the dashed line in figure 15. About 1/3 of the area under the debris-flow hydrograph overlaps the water-flood hydrograph, and the remaining 2/3 of the debris-flow hydrograph, not included in the water-flood hydrograph, is sediment. This suggests the volume of the debris flow was about 1,800 m^3.

CONSTRUCTED-DAM FAILURES

A dam failure is a complex hydrologic, hydraulic, and geologic phenomenon whose resulting flood characteristics are controlled primarily by the failure mechanism and characteristic and properties of the dam. Models that use simple and readily available geometry of the dam and reservoir can provide reasonable reproductions (and thus predictions) of peak discharge from dam failures (Costa, 1988, p. 442-448). One such simple measure is the product of water volume, height of dam, and specific weight of water, or potential energy of water behind a dam.

The collapse of the southwestern side of Reservoir Number 3 opened a large breach in the side of the reservoir and allowed 13,250 m^3 of water to escape rapidly. The resulting flood peak-discharge a short distance downvalley ranks as one of the largest floods documented from the failure of a constructed dam for the available potential energy of water in the dam prior to failure (fig. 16). The data in figure 16 include earthen and rigid concrete dams that have failed by a variety of processes, and for which reasonable estimates of peak flood discharge exist (Costa, 1988). Three dam failures define the limiting line for data of potential energy as a function of peak flood discharge: Buffalo Creek, W.Va.; Malpasset Dam, France, and Reservoir Number 3, Centralia, Wash.

The distinguishing characteristic of these three dams is that a rapid foundation failure and subsequent instantaneous release of water led to an extraordinary large flood peak-discharge for the size of the reservoir. The dam at Buffalo Creek, W.Va. was a coal spoil pile that failed in February, 1972 by rapid slumping and sliding of the liquified face of the dam accompanying a heavy rainstorm (Davies and others, 1972). Malpasset Dam, France, was a 61-m high concrete thin-arch dam that collapsed in a catastrophic bedrock foundation failure in December, 1959 (Jansen, 1980). Reservoir Number 3 in Centralia, Wash. failed rapidly when part of the bedrock foundation under the southwest corner of the reservoir slid out and downward, emptying the reservoir in a matter of minutes.

Failure mechanisms for many of the other dams plotted in figure 16, primarily overtopping, did not lead to an instantaneous release of water. Breaches that formed during overtopping grew gradually, and other kinds of foundation failures were not so catastrophic as the rapid mass movements that caused the dam failures that determine the location of the envelope curve. The Centralia, Wash. reservoir failure defines the location of the envelope curve at the low end of the available data, and thus represents an important hydrologic event for identifying the limit of the size of floods from dams that differ in size and in failure mechanism.

CONCLUSIONS

The failure of Reservoir Number 3 on Oct. 5, 1991 in Centralia, Wash. from a deep-seated bedrock foundation slide is more than a curiosity. Rapid release of 13,250 m^3 of water eroded hillslope deposits, fill, and bedrock. The flood quickly bulked into a debris flow with an estimated volume of 1,800 m^3 that swept into the eastern edge of the city of Centralia. Geomorphic and sedimentologic evidence can be used to document that the dam-break flood had at least two phases - initially a debris flow that was quickly followed by a water flood whose maximum stage was about one-half meter higher than the debris flow. Flood peak discharge is calculated to have been 71 m^3/s using the slope-area method in which roughness coefficients were field-selected after being bracketed by calculations that determine only grain roughness (a minimum value), and total roughness in steep, fixed-bed channels (a maximum value, because

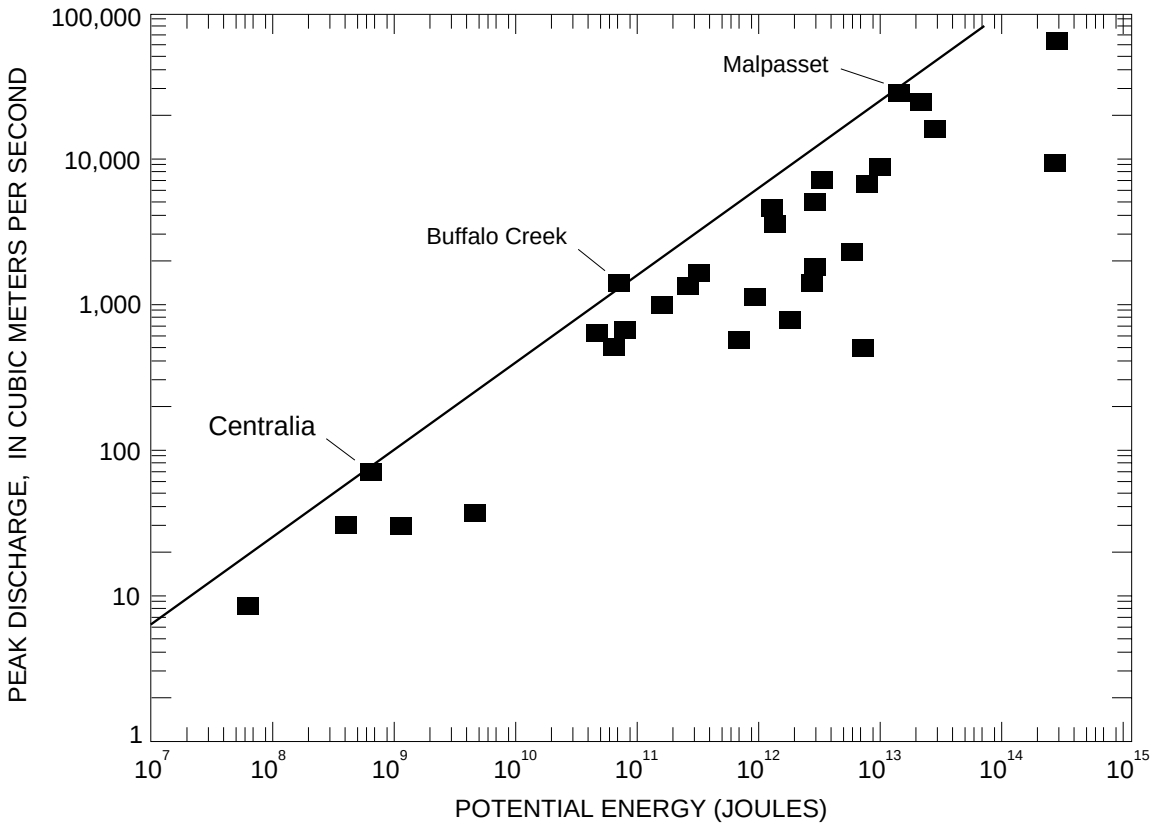

Figure 16 . Potential energy versus peak discharge for constructed dams. Envelope curve is defined by the French Malpasset Dam failure (Jansen, 1980), the Buffalo Creek, W. Va. coal-spoil dam failure (Davies and others, 1972), and the Centralia, Wash. reservoir failure. Data from Costa (1988).

the stream-bed material here was entirely mobile). The resulting discharge of 71 m^3/s is consistent with estimates derived by considering the rate of emptying of the reservoir, and a simplified slope-area relation that substitutes slope for flow resistance. The flood peak was in the supercritical flow regime for at least 200 m through the slope-area reach. A reconstructed hydrograph of the flood indicates the duration of flow past the slope-area site located 275 m downstream of the reservoir was 6.2 minutes and the entire flood was over within about 7.3 minutes.

The foundation failure of Reservoir Number 3 resulted in a rapid draining of water. The resulting flood a short distance downvalley was large considering the potential energy of the water prior to the failure when compared with other historic constructed dam failures. Plotted in this manner, the Centralia flood, along with two other rapid-foundation failure dam break floods, defines the empirical limit for flood peak discharge associated with the failure of constructed dams. These results reaffirm that dam failure floods, while rare, are important hydrologic events that need to be carefully documented because such floods are relatively rare compared with rainfall-runoff or snowmelt floods, and can occur during sunny, pleasant weather without any precursory indications. Floods from the failure of dams in small upland basins present unique challenges and considerations for public safety.

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