Multiple Flow Processes Accompanying a Dam-break Flood in a Small Upland Watershed, Centralia,Washington

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CONVERSION FACTORS AND VERTICAL DATUM

Multiply	By	To obtain
millimeter	0.039	inch
meter	3.28	foot
kilometer	0.62	mile
cubic meter	35.3	cubic foot
meter per second	3.28	foot per second
cubic meter per second	35.3	cubic foot per second

Sea level: In this report “sea level” refers to the National Geodetic Vertical Datum of 1929–a geodetic datum derived from a general adjustment of the first-order level nets of the United States and Canada, formerly called Sea Level Datum of 1929.

Multiple Flow Processes Accompanying a Dam-break Flood in a Small Upland Watershed, Centralia, Washington

By John E. Costa

Abstract

On October 5, 1991, following 35 consecutive days of dry weather, a 105-meter long, 37-meter wide, 5.2-meter deep concrete-lined water-supply reservoir on a hillside in the eastern edge of Centralia, Washington, suddenly failed, sending 13,250 cubic meters of water rushing down a small, steep tributary channel into the city. Two houses were destroyed, several others damaged, mud and debris were deposited in streets, on lawns, and in basements over four city blocks, and 400 people were evacuated. The cause of failure is believed to have been a sliding failure along a weak seam or joint in the siltstone bedrock beneath the reservoir, possibly triggered by increased seepage into the rock foundation through continued deterioration of concrete panel seams, and a slight rise (0.6 meters) in the pool elevation. A second adjacent reservoir containing 18,900 cubic meters of water also drained, but far more slowly, when a 41-cm diameter connecting pipe was broken by the landslide. The maximum discharge resulting from the dam-failure was about 71 cubic meters per second. A reconstructed hydrograph based on the known reservoir volume and calculated peak discharge indicates the flood duration was about 6.2 minutes. Sedimentologic evidence, high-water mark distribution, and landforms preserved in the valley floor indicate that the dam failure flood consisted of two flow phases: an initial debris flow that deposited coarse bouldery sediment along the slope-area reach as it lost volume, followed soon after by a water-flood that achieved a stage about one-half meter higher than the debris flow. The Centralia dam failure is one of three constructed dams destroyed by rapid foundation failure that defines the upper limits of an envelope curve of peak flood discharge as a function of potential energy for failed constructed dams worldwide.

INTRODUCTION

Centralia, Washington is located in the southern end of the Puget Trough about 135 km south of Seattle (fig. 1). At about 10:15 AM on October 5, 1991 the hillslope under the southwestern side of a concrete-lined water-supply reservoir used by the city of Centralia located on Seminary Hill (NE 1/4 SW 1/4, sec 9, T14N, R2W) suddenly failed. A roily mass of water, vegetation, and sediment flowed down a small, steep tributary into the eastern part of the city, destroying two houses, flooding scores more, and forcing the evacuation of 400 people. The dam failure occurred on a clear sunny morning after a prolonged period of dry weather. Temperatures were well-above normal in August and September, and no measurable rainfall had occurred for 35 days prior to the failure (NOAA, 1991).

The reservoir that failed was named "Reservoir Number 3." This reservoir is 105 m long, 37 m wide, 5.2 m deep, contained 13,250 m^3 of water, and was constructed in 1914. It was one of two adjacent reservoirs constructed of unreinforced concrete panels to store water from well fields for the water supply of the City of Centralia. The second reservoir ("Reservoir Number 4"), is 121 m long, 39 m wide, 6.1 m

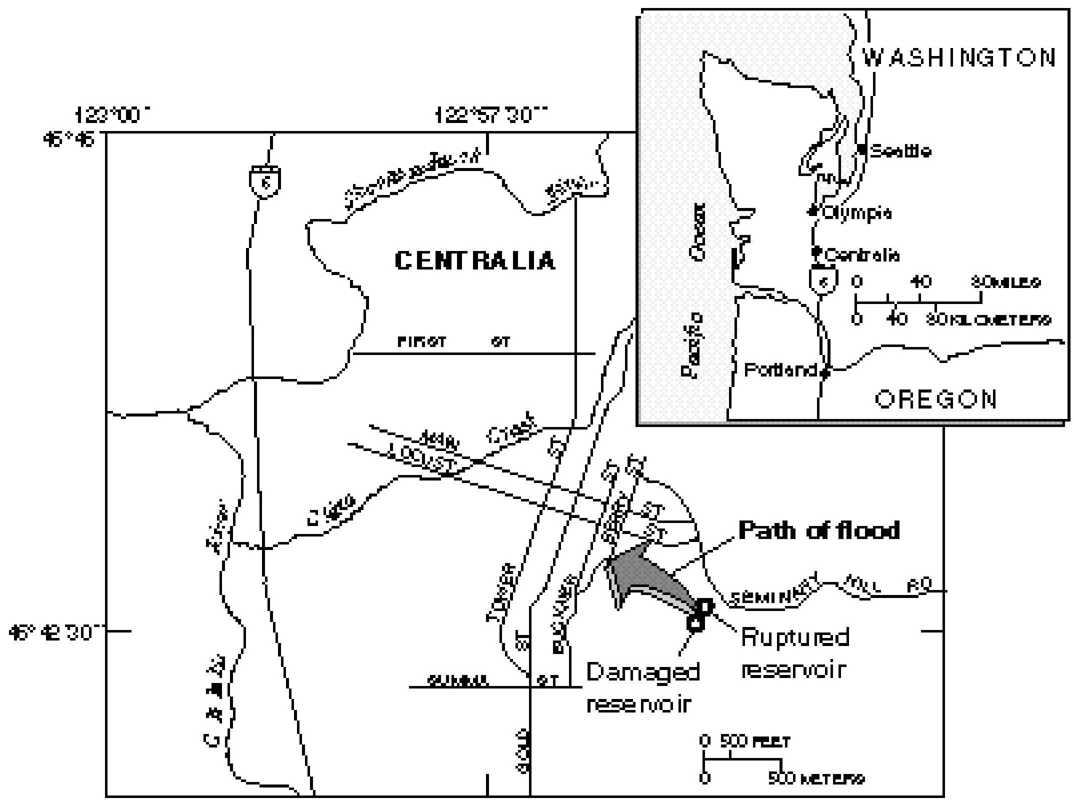

Figure 1. Location map of Centralia, Wash. and water-supply reservoirs that failed on Oct. 5, 1991.

deep, stored 18,900 m^3 of water and was constructed in 1926. Both reservoirs have 1:1 interior sideslopes (fig. 2). The reservoirs were excavated into bedrock below original ground level, and some of the excavated material was used as fill on the west side of the hillslope. The embankment failure under Reservoir Number 3 caused the service and drain pipes connected to the larger second reservoir (Reservoir Number 4) to break, allowing uncontrolled release of an additional 18,900 cubic meters of water through a 41-centimeter-diameter pipe over the next several hours.

Purpose and Scope

This report presents documentation of the failure mechanism, peak discharge, geomorphology, and sedimentology of the failure of a constructed dam in a small upland watershed. Such floods are poorly documented compared with rainfall-runoff or snowmelt floods, and present unique hazards because dam failures and resultant flash flooding can occur at any time without warning, even during sunny weather.

Acknowledgements

Kurt Spicer, U.S. Geological Survey, led the field crew that surveyed the valley and channel downstream from the failed reservoir, and conducted the initial runs of the slope-area computer program. Dennis Saunders, U.S. Geological Survey, assisted with the field surveys and plotted the field data.

DAM-FAILURE CIRCUMSTANCES

The cause of this failure is not known with certainty, but increased seepage into the fractured bedrock foundation through continued deterioration of the concrete panel seams must have been a significant factor. Post-failure inspection of the seams between the concrete panels indicated that at least two kinds of caulking had been used in attempts to seal the gaps in the past. Maintenance and repair records for these reservoirs document they have had a history of excessive leakage for at least the last 33 years. In 1971 leakage rates were measured in Reservoir Number 3 when it was only 80 percent full. Over four percent of the capacity of the reservoir (545 cubic meters) was being

Figure 2. Aerial photograph of failed reservoir. East is at top of photo; ground slopes steeply to the west.

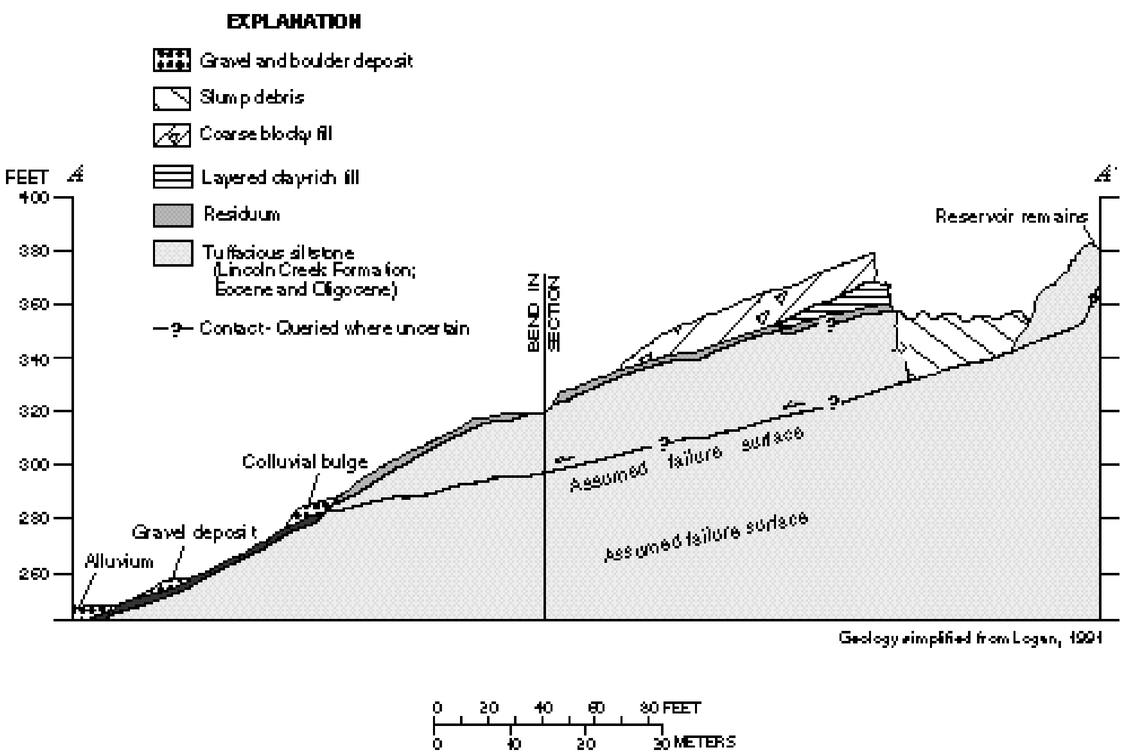

Figure 3. Geologic section of the area down-slope of Reservoir Number 3 (after Logan, 1991). Location of cross section A-A’ marked on figure 5.

Figure 4. Hillslope below Reservoir Number 3 that has been washed and eroded by overland flow. View is upslope.

lost per day, and water was seeping from the embankment downslope from the reservoir. Repairs were made to reduce the leakage (Dodd Pacific Engineering, Inc., 1992). A consultant's report estimated the leakage rate of the reservoir prior to the landslide failure in October, 1991, was between 284 and 568 cubic meters per day (Dodd Pacific Engineering, Inc., 1992).

Both reservoirs were drained, cleaned, and refilled four times a year. This pattern of cyclic draining and filling could have contributed to subsurface hydrostatic stress fluctuations that in turn may have led to decreased bedrock stability over time. A new well for water supply had come on line about a year before the failure, and the level of water in Reservoir Number 3 had been increased by 0.61 meters. Leakage into the bedrock foundation, and increased porepressure in the bedrock ground-water system as a consequence of raising the normal level of water in the reservoir, could also have contributed to the instability and failure.

The mass movement that caused the immediate failure of the reservoir was a relatively deep slide in bedrock that left a near-vertical headwall, and curved failure surface that extended to an estimated depth of 25 to 35 meters. Depth of the failure surface was estimated from the elevation difference between the reservoirs and toe of the slide bulging on the hillside below the reservoirs (fig. 3). The failed block of rock and fill moved downward about 9.1 m, and westward (outward) approximately 6.1 m, separating the concrete slabs that lined the floor of Reservoir Number 3 and opening a 40-meter gash in the southwest side of the reservoir. Post-failure slumping and sliding obscured the original morphology of the breach in the reservoir. Five days after the failure, the detached slump block had two breaches through which water discharged. The first breach was on the north end of the failure block. The depth of the breach was about 6 meters, and breach width was about 3-4 meters at the time of my visit, but the geometry had been obviously narrowed by post-failure bank collapse. A slightly smaller breach was located about 7 meters south of the first breach, but it too had been significantly modified by post-failure ground movements.

Below the breaches the ground was washed by sheetflow and there was substantial erosion of fill, colluvium, residuum, and some bedrock (fig. 4).

Three distinct channels were eroded into the steep hillslope below Reservoir Number 3. The channel on the north edge of the slide block is the deepest (3 - 4.5 m). The second channel (1.5 - 3 m deep) lies about 15 m south of the first channel, and the third channel (about 1 - 1.5 m deep) is about 40 m south of the first channel, near the middle of the slide block (fig. 5).

The mass movements that led to the rapid draining of Reservoir Number 3 also broke the 41 cm drainage pipe beneath Reservoir Number 4, causing the 18,900 m^3 of water in this reservoir to drain through the pipe over the next several hours. This discharge severely eroded fill, colluvium, and friable siltstone bedrock on the hillslope below the pipe outfall, and eroded a 7-m deep channel in the hillside. Although Reservoir Number 4 contained over 40 percent more water than Reservoir Number 3, the slow release through a single pipe did not contribute significantly to the flood-peak discharge in the valley downstream.

DESCRIPTIONS OF AREA

Geology

Centralia lies about 18 km south of the glacial border at an altitude of 58 m on the flat valley floor of the Chehalis and Skookumchuck Rivers. These alluvial plains are underlain by thick glacial outwash debris. A small unnamed first-order tributary drains the hillslope below the Centralia water-supply reservoirs and flows into China Creek, a small creek that originates in the hills east of Centralia and flows underground through the middle of the city before joining the Chehalis River on the southwestern edge of the city just upstream of the juncture with the Skookumchuck River. Reservoirs Number 3 and 4 are located on the north side of Seminary Hill at an elevation of 128 m about 73 m above the city. South of the reservoirs, the land continues to rise and reaches a maximum altitude of 168 m. The reservoirs are underlain by sandy siltstone known as the Lincoln Creek Formation (late Eocene and Oligocene age) derived primarily from volcanic source materials (Snavely and others, 1958; Schasse, 1987). The hillslope is mantled by deeply weathered residuum and colluvium. These deposits are so weathered that most of the gravel clasts are altered to soft friable masses of sandy clay. On the west side of the reservoir the hillslope is covered by 3.0-3.5 m of coarse sandy gravel and boulder fill of highly weathered tuffaceous siltstone. This fill probably originated from the bedrock excavations for the reservoirs, and was severely eroded by floodwaters when Reservoir Number 3 drained (fig. 3).

The failure surface of the landslide that led to the reservoir collapse is in the tuffaceous siltstone of the Lincoln Creek Formation. Bedrock beneath the reservoir floor exposed by the landslide and flood is massive, spheroidally weathered and sparsely to densely jointed. Joints strike approximately east-west, and north-south, and have near-vertical dip. Along the failure surface, the siltstone was extensively broken and shattered into gravel and boulder-sized angular fragments. Well-developed slickensides were formed in the clay-rich fill and residuum and in the upper 40 cm of bedrock beneath the floor of the reservoir.

Valley and flood-deposit features

The valley below the reservoirs has a slope of about 21 percent for the first 200 meters, then gradually flattens to about 9 percent in the reach where the peak-flood discharge was calculated using the slope-area method, about 275 meters down-valley from the reservoirs. In the first 200 meters, unconfined flow eroded siltstone bedrock and surficial deposits of organic matter and soil, toppled trees, and locally became channelized, forming four channels deeper than one meter on the hillside. About 200 meters down valley, there is a significant break in slope and the valley widens. Here, the flows deposited a continuous thick layer of gravel and boulders (fig. 6). A pebble count using the Wolman (1954) method was made on the gravel deposit just upstream of the slope-area reach. The median size was 128 mm, the largest particle had an intermediate diameter of 560 mm, and the sorting (σ) was 0.97. Of 100 particles measured, 7 of the clasts larger than 16 mm were rounded masses of cohesive silt and clay that behaved as individual clasts during the flows. The most distinguishing characteristic of gravel and boulder deposits below the failed reservoir is the degree of weathering. Many siltstone gravel clasts in flow deposits are "soft" and have a low density. When struck with a hammer, the rocks produce a thud and break easily. About 250 meters downvalley, the gravel deposits become discontinuous and bars and splays of gravel are sepa-

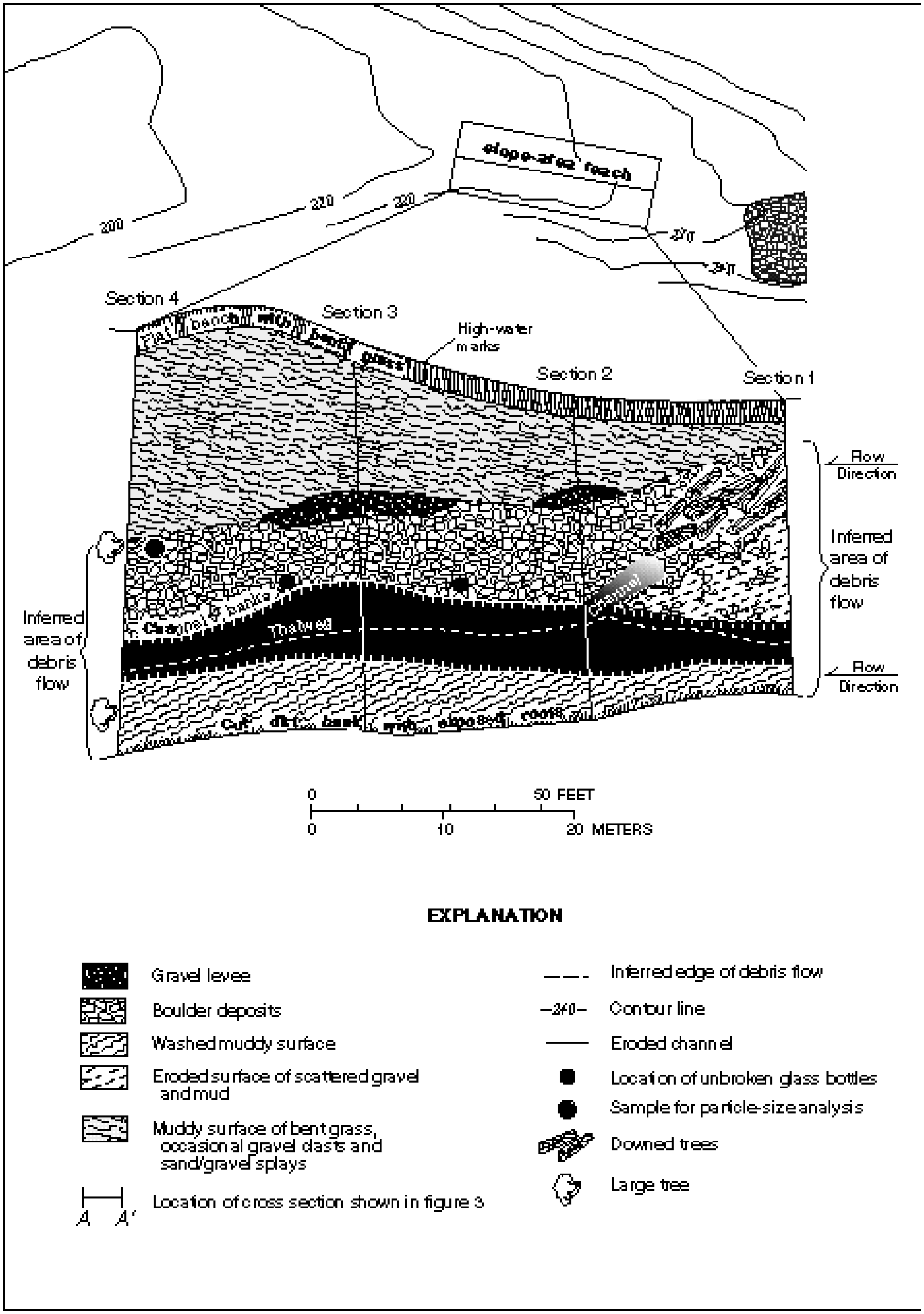

Figure 5. Topographic map of the slide block and slope-area reach below Reservoir Number 3. Topographic survey done in English units (feet).

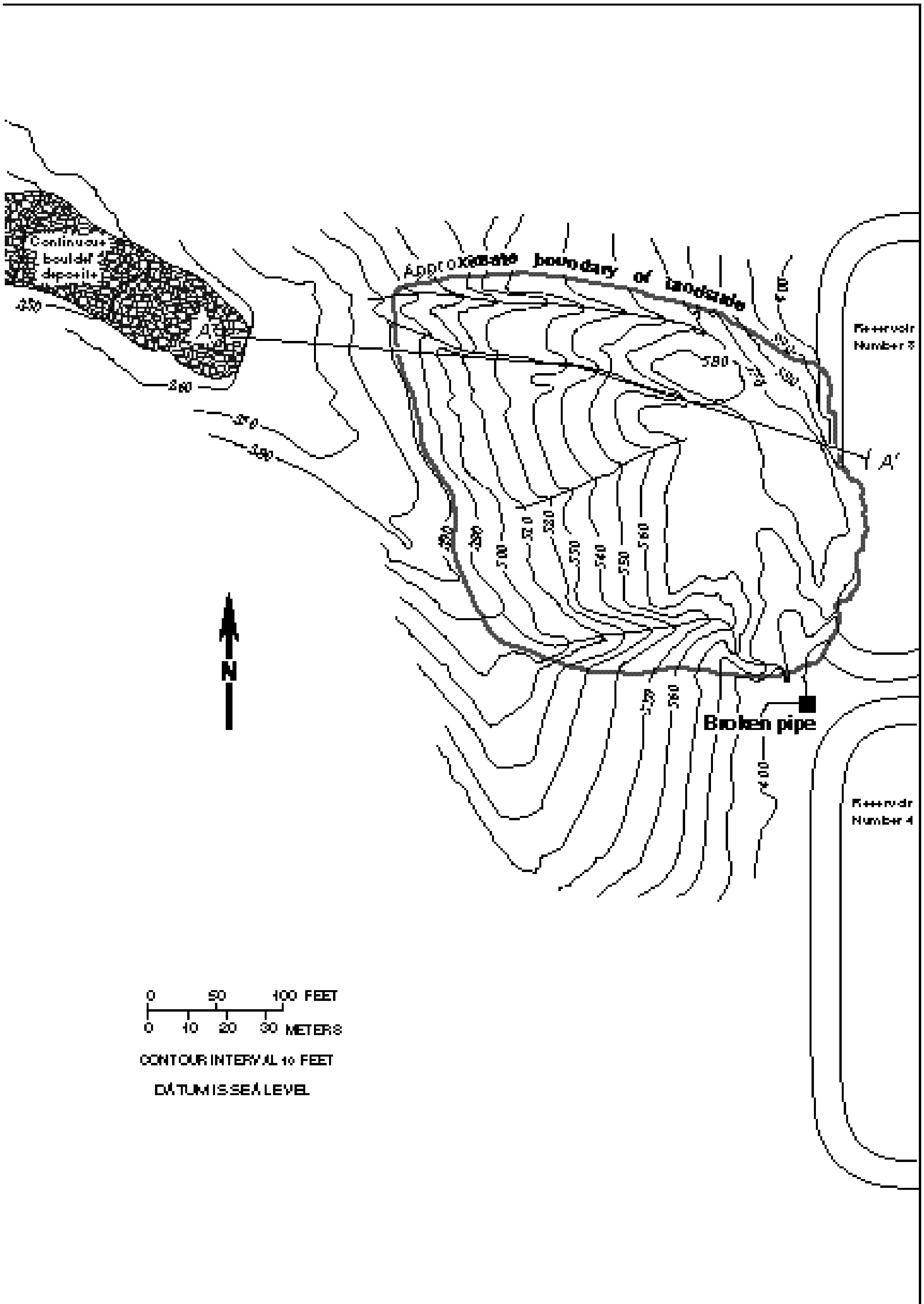
Continuous boulder deposits
A
A'
Approximate boundary of landslide
Reservoir Number 3
Reservoir Number 4
Broken pipe
N
0 50 100 FEET
0 10 20 30 METERS
CONTOUR INTERVAL 10 FEET
DATUM IS SEA LEVEL

Figure 6. Massive gravel deposit upstream from the slope-area reach. View is downstream.

rated by washed chutes and channels. Further down-valley, the flow width expanded rapidly, resulting in the deposition of a fan at the edge of town.

EVIDENCE OF MULTIPLE FLOW PROCESSES

Morphology and Sedimentology of Deposits

Multiple flow processes may be common phenomenona during large flows in steep channels, but sufficient evidence to accurately reconstruct the occurrence of multiple flows with different rheologies, and their chronology, is rarely preserved (Broscoe and Thomson, 1969; Johnson and Rahn, 1970). Field investigations after the dam failure indicated that the initial flow down the valley was a debris flow, which was followed quickly by a water flood that had a stage about one-half meter higher. Evidence for this conclusion includes the texture and sedimentology of deposits in the valley, the presence of washed debris-flow levees along part of the channel, the transport and preservation of glass bottles in gravel and boulder deposits, and the characteristics of high-water marks on the valley flats.

In cut banks of newly-deposited gravel in the slope-area reach, the washed surface of clean gravels and boulders was immediately underlain by a sandy gravel deposit in which many clasts were separated from others by a matrix of sand and silt (fig. 7). A mechanical analysis of one gravel sample is shown in figure 8. The median particle size is about 14 mm, and the shape of the frequency curve and sorting (σ = 2.1) is characteristic of debris flow deposits (Scott, 1988). A mechanical analysis of coarse, gravel fill collected from the hillslope just below a breach in the down-dropped section of bedrock under the reservoir is also plotted in figure 8. This coarse fill is presumably the source material for most of the downstream gravel deposits. Median particle size is about 8 mm, and curve shape and sorting (σ = 2.2) is very similar to the down-valley inferred debris-flow material. The derivative down-valley deposit is depleted in fines and coarser in texture than the source material, a consequence of being transported about 275 meters in the dam-break flow.

Figure 7. Debris-flow sediments deposited on the right side of the floodplain near cross section 4. Note the coarse boulder lag on the surface, the matrix-supported gravels and cobbles, the occasional megaclasts, and the non-erosive deposition on floodplain grasses. Shovel for scale.

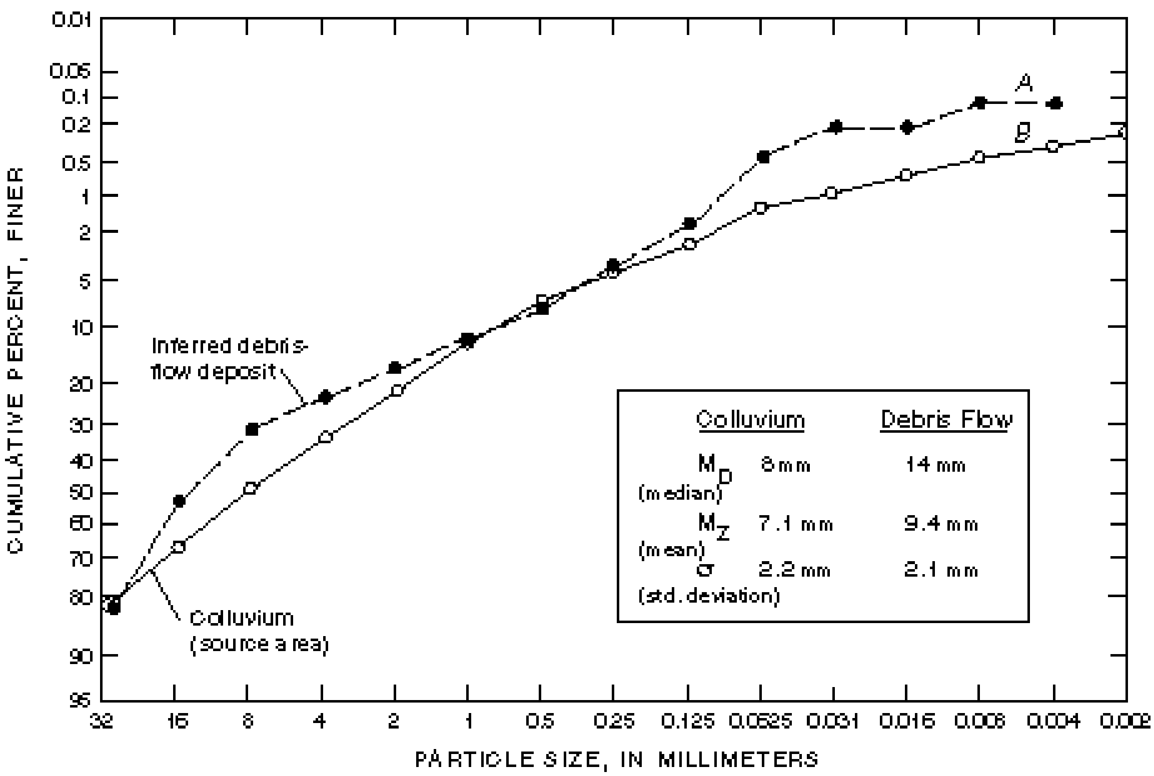

Figure 8. Mechanical analysis of debris-flow deposits shown in figure 7(A), and source-area gravel fill (B).

Figure 9. Boulder levee washed of fines along the left valley wall between cross sections 2 and 3. High water mark is about one meter above the levee, hand shovel for scale.

In several locations through the slope-area reach, paired linear ridges of clean gravel and boulders on the surface with matrix-supported gravel in the subsurface are found along both sides of the thalweg of the channel. The location, morphology, texture, and sedimentology of these boulder ridges lead to the interpretation that they are washed debris-flow levees that were deposited in the early part of the dam-break flow, then washed by a water flood a short time after emplacement (fig. 9).

In two locations in the slope-area reach, unbroken glass beer bottles were found in a washed gravel deposit with imbricate structure (fig. 10). The bottles had obviously been transported in a flow that was moving coarse boulders and gravel, and it is unlikely a glass bottle could survive such transport unless the flow was a near-laminar rigid visco-plastic fluid with finite shear strength, as in some debris flows. Preservation of fragile clasts and objects such as brittle shale fragments, blocks of unconsolidated colluvium, and chunks of soil source materials is characteristic of some laminar, non-deforming debris flows (Johnson, 1970; Enos, 1977).

The right edge of the floodway in the slope-area reach is broad and flat, and high-water marks were well-defined. In this flat overflow region, the area

Table 1. Hydraulic data for Centralia, Washington dam-failure flood, Oct. 5, 1991

Cross section number	Area (m^2)	Width (m)	Hydraulic Radius (m)	Froude Number	Velocity Head (m)	Manning n	Debris-flow area (m^2)
1	19.1	18.9	0.98	1.10	0.65	0.045 0.075	13.7
2	15.2	18.0	0.79	1.76	1.17	0.045 0.075	9.2
3	19.5	24.4	0.76	1.22	0.78	0.045 0.055	14.6
4	13.2	26.8	0.49	2.01	1.26	0.045 0.045 0.055	8.1

nearest the valley side had tall grasses bent in the flow direction, and little or no sediment deposited on the surface. Except for scattered gravel clasts and pieces of woody debris, the bent grass was the only indication of the flood. A 5-cm diameter alder sapling was scarred, but not bent or stripped of bark or leaves (fig. 11). This area had been swept by a short-duration water flood. Closer to the channel, gravel bars and deposits up to 0.5 meter thick appear. Trees are battered and scarred, and the original overbank surface is greatly modified by sediment deposition and some minor scour. Along the left valley wall, leaves above the high-water marks were splattered with mud. This part of the valley floor had been swept by a sediment-rich flow (fig. 11).

The elevation of preserved levee deposits, and the edges of gravel deposits were used to reconstruct the approximate stage of the debris flow at each of four cross sections. These stages are plotted on the cross-sections (fig. 12). No estimate of debris-flow velocity could be made from any remaining evidence such as super-elevation marks (Costa, 1984). The cross-sectional area of the debris flow decreases in the downstream direction from 13.7 m^2 at section no. 1, to 8.1 m^2 at section no. 4, 180 m downstream. Post-debris flow scour by floodwater at cross-section 3 probably accounts for the local increase in measured debris-flow area. In addition to changes in velocity, the decline in debris-flow cross-sectional area is at least partly explained by the fact that the debris flow was depositing sediment along the floodplain in the reach of the slope-area discharge measurement (table 1).

The debris flow was followed by a water flood that set high-water marks about one-half meter above the maximum stage of the debris flow. The slope-area reach is in a location where valley slope is decreasing. The initial debris flow deposited part of its volume in this reach, but this material was partially eroded by the water-flood that followed. These debris-flow deposits helped to protect the valley floor from the high velocity, shear stress, and stream power of the subsequent water-flood. There is little field evidence for erosion of original floodplain topography other than some local scour on the floodplain surface, and a 2-m deep headcut in the channel between cross section 2 and 3 (fig. 13).

Discharge Estimates

A four-section slope-area indirect discharge estimate was made on October 10, 1991, five days after the dam failure. The slope-area method is frequently used to compute peak discharge after the passage of a flood using high-water marks, channel cross-sections, and estimates of flow resistance (Dalrymple and Benson, 1967). Estimates of peak discharge of large water floods from indirect evidence in steep basins are subject to many uncertainties (Kirby, 1987; Jarrett, 1987). Recognized problems include misidentification of debris flows as water floods (Costa and Jarrett, 1981), improper site selection and hydraulic assumptions (Costa, 1987; Jarrett, 1987), and field selection of roughness coefficients ("Manning's n").

Figure 10. Unbroken glass beer bottle deposited with debris-flow sediments. Location plotted on fig. 5. Glove on rock above bottle in center of photo for scale.

Figure 11. Flood plain in the slope-area reach. View is downstream. Debris flow inundated the floodplain on the left side of the channel. Willow in the center of photo was scarred by the water flood, but not bent or stripped of bark or leaves.

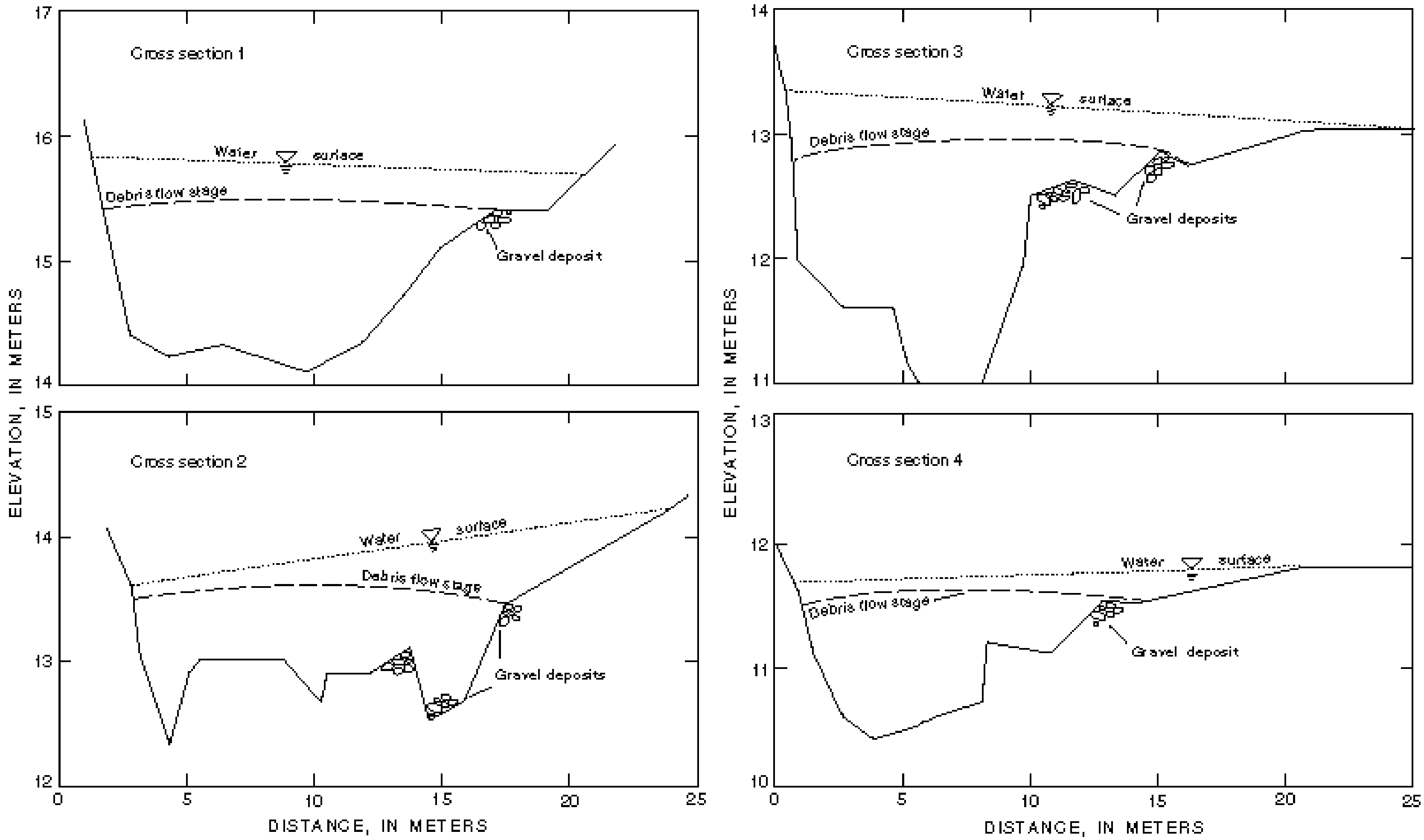

Figure 12a-d. Cross sections used in determination of peak discharge. Cross section 1 is upstream-most cross section.

Figure 13a-b. Cross sections 2 and 4 used in the slope-area analysis. Both views are upstream.

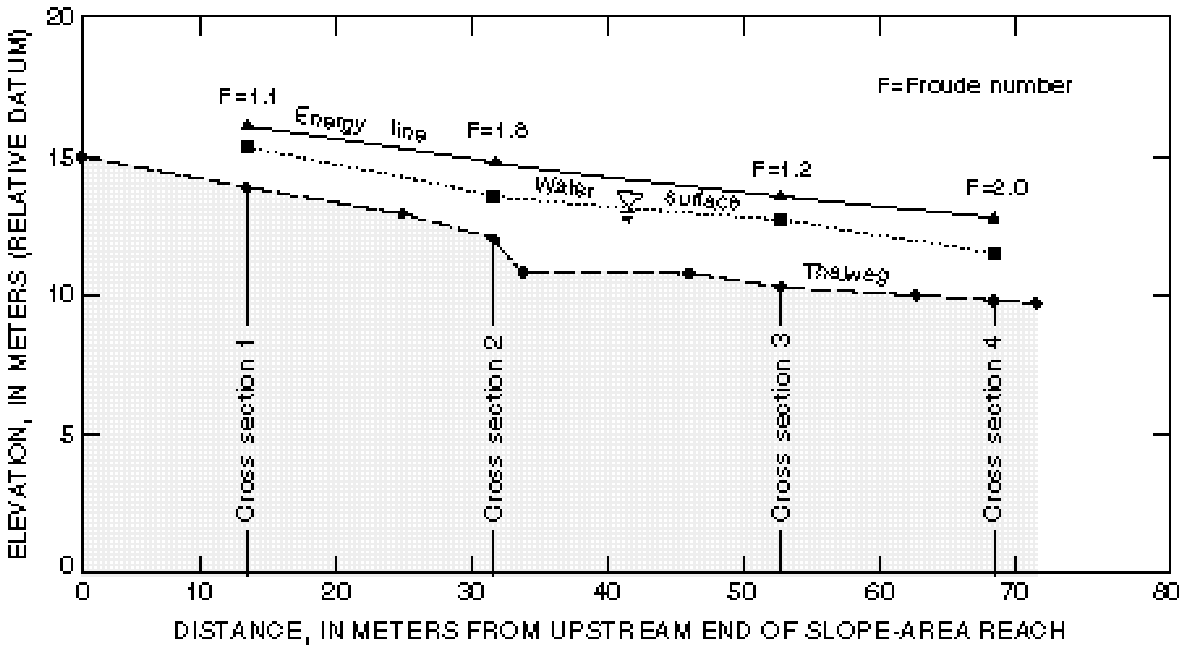

Figure 14. Total-energy diagram for the peak water-flood at Centralia, Wash.

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):

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

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

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

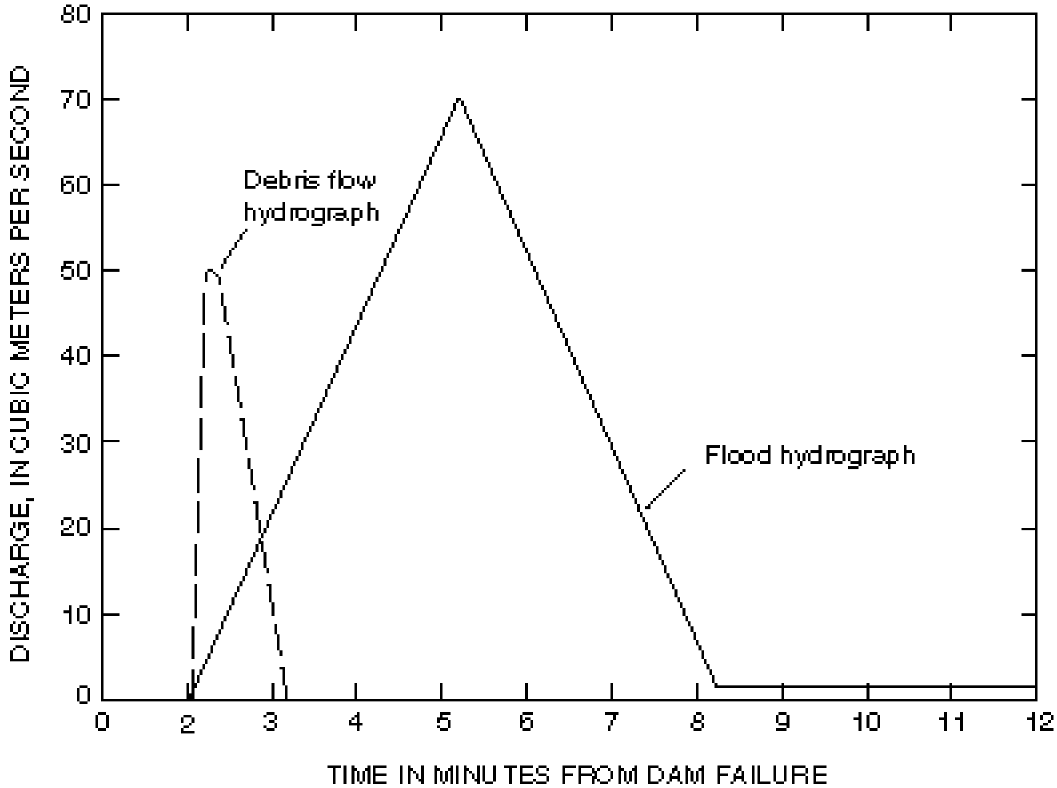

Figure 15. Reconstructed hydrographs of the Centralia debris flow and water-flood.

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

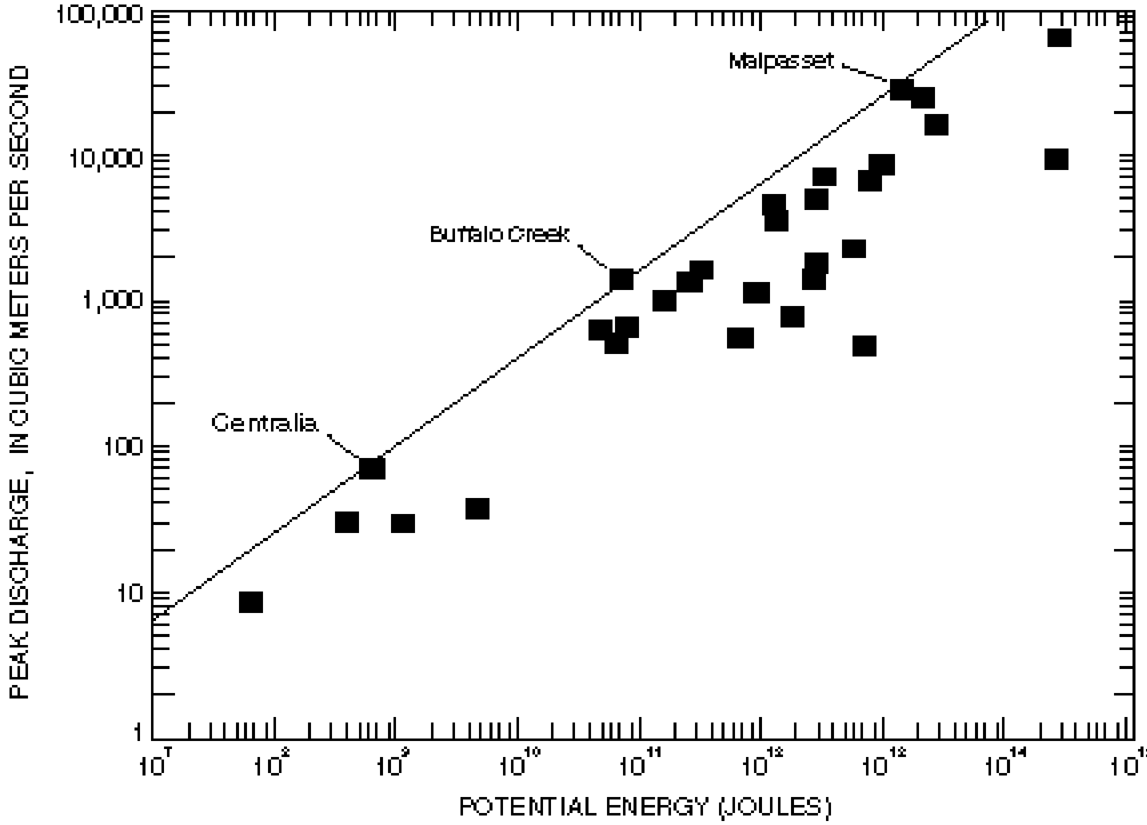

Figure 16 . Potential energy versus peak discharge for constructed dams. Envelope curve is defined by the French Malpasset Dam failure (Jansen, 1980), the Buffalo Creek, W. Va. coal-spoil dam failure (Davies and others, 1972), and the Centralia, Wash. reservoir failure. Data from Costa (1988).

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.

REFERENCES CITED

Benson, M.A. and Dalrymple, T., 1967, General field and office procedures for indirect discharge measurements: Techniques of Water-Resources Investigations, U.S. Geological Survey, Book 3, Chap. A1, 30 p.

Broscoe, A.J. and Thomson, S., 1969, Observations on an alpine mudflow, Steel Creek, Yukon: Canadian Journal of Earth Science, v. 6, p. 219-229.

Costa, J.E., 1984, The physical geomorphology of debris flows: in Costa, J.E. and Fleisher, P.J. (Eds), Developments and Applications of Geomorphology, Springer-Verlag, New York, p. 268-317.

______ 1987, A comparison of the largest rainfall-runoff floods in the United States, with those of the Peoples Republic of China: Journal of Hydrology, v. 96, p. 101-115.

Costa, J.E. and Jarrett, R.D., 1981, Debris flows in small mountain stream channels of Colorado and their hydrologic implications: Association of Engineering Geologists Bulletin, v. 18, p. 309-322.

Dalrymple, T. and Benson, M.A., 1967, Measurement of peak discharge by the slope-area method: Techniques of Water-Resources Investigations, U.S. Geological Survey, Book 3, Chap. A2, 12 p.

Davies, W.E., Bailey, J.F., and Kelly, D.B., 1972, West Virginia's Buffalo Creek flood - a study of the hydrology and engineering geology: U.S. Geological Survey Circular 667, 32 p.

Dodd Pacific Engineering, Inc., 1992, Site observations and review of reservoir failure, Water Utility Reservoirs No. 3 and No. 4, Centralia, WA: consultant's report, Dodd Pacific Engineering, Inc., Seattle, WA, 10 p.

Enos, P., 1977, Flow regimes in debris flows: Sedimentology, v. 24, p. 133-142.

Hicks, D.M. and Mason, P.D., 1991, Roughness characteristics of New Zealand Rivers: Water Resources Survey, New Zealand, 329 p.

Jansen, R.B., 1980, Dams and Public Safety: Bureau of Reclamation, Denver, CO, 332 p.

Jarrett, R.D., 1984, Hydraulics of high-gradient streams: Journal of Hydraulic Engineering, v. 110, no. 11, p. 1519-1539.

______1987, Errors in slope-area computations of peak discharges in mountain streams: Journal of Hydrology, v. 96, p. 53-67.

Johnson, A.M., 1970, Physical Processes in Geology: Freeman and Cooper, San Francisco, 577 p.

Johnson, A.M. and Rahn, P.H., 1970, Mobilization of debris flows: Zeitschrift fur Geomorphology, v. 9, p. 168-186.

Kirby, W.H., 1987, Linear error analysis of slope-area discharge determinations: Journal of Hydrology, v. 96, p. 125-138.

Lara, O.G. and Davidian, J., 1970, Preparation of input data for automatic computation of discharge by slope-area method: Techniques of Water-Resources Investigations, Chapter C2, Book 7, 14 p.

Leopold, L.B., Wolman, M.G., and Miller, J.P., 1964, Fluvial Processes in Geomorphology: W.H. Freeman, San Francisco, 522 p.

Limerinos, J.T., 1970, Determination of the Manning coefficient from measured bed roughness in natural channels: U.S. Geological Survey Water Supply Paper 1898-B, 47 p.

Logan, R.L., 1991, letter to Administrator, City of Centralia, dated Dec. 10, 1991, Washington State Department of Natural Resources, 4 p. plus geologic cross-section.

Marcus, W.A., Roberts, K., Harvey, L. and Tackman, G., 1992, An evaluation of methods for estimating Manning's n in small mountain streams: Mountain Research and Development, v. 12, p. 227-239.

National Oceanographic and Atmospheric Administration, 1991, Climatological Data, Washington: v. 95, no. 8-10.

Riggs, H.C., 1976, A simplified slope-area method for estimating flood discharges in natural channels: U.S. Geological Survey Journal of Research, v. 4, p. 285-291.

Schasse, H.C., 1987, Geologic map of the Centralia Quadrangle, Washington: Washington Division of Geology and Earth Resources, Open File Report 87-11, Scale 1:100,000, Olympia, WA

Scott, K.M., 1988, Origins, behavior, and sedimentology of lahars and lahar-runout flows in the Toutle-Cowlitz River system: U.S. Geological Survey Professional Paper 1447-A, 74 p.

Snavely, P.D., Brown, R.D., Roberts, A.E., and Rau, W.W., 1958, Geology and coal resources of the Centralia-Chehalis District, Washington: U.S. Geological Survey Bulletin 1053, 159 p.

Trieste, D.J., 1992, Evaluation of supercritical/subcritical flows in high-gradient channels: Journal of Hydraulic Engineering, v. 118, p. 1107-1118.

Wolman, M.G., 1954, A method of sampling coarse river-bed material: Transactions, American Geophysical Union, v. 35, p. 951-956.