<u>Indirect Measurement Summary</u>

Little Missouri River at Albert Pike, Arkansas
Ouchita River Basin
Miscellaneous Site

Q= 40,100 ft^3/s

Flood of June 11, 2010

<u>Type of measurement:</u> Slope Area

<u>Location of site:</u> A miscellaneous site adjacent and through parts of Area C of the U.S. Forest Service Campground at Albert Pike Recreation Area. Lat N 34˚22’35”, Long W 93˚52’50” This site was selected because it was a straight converging reach. This reach runs almost due east and is downstream of the inflow from Brier Creek and upstream of the unnamed tributary coming in on the left bank where the river takes an abrubt curve to the southwest. A low water bridge crosses the river approximately 700 feet downstream of the the indirect measurement location. A severe constriction occurs 0.7 miles downstream of the indirect measurement site (just downstream of the abrupt bend in the river) as a highly resistant layer of novaculite protrudes into the channel on the left bank (outer bend of the river) and a constructed parking area protrudes on the right bank. Surveys were also conducted in the area of the constriction to check if critical depth computations for flow can be made in this constricted area.

17 of the 20 flood fatalities came from Area D of the campground which was 0.3 river miles upstream of the indirect measurement site. Area C of the campground was under construction at the time of the flood and had no occupants camping the morning of June 11, 2010.

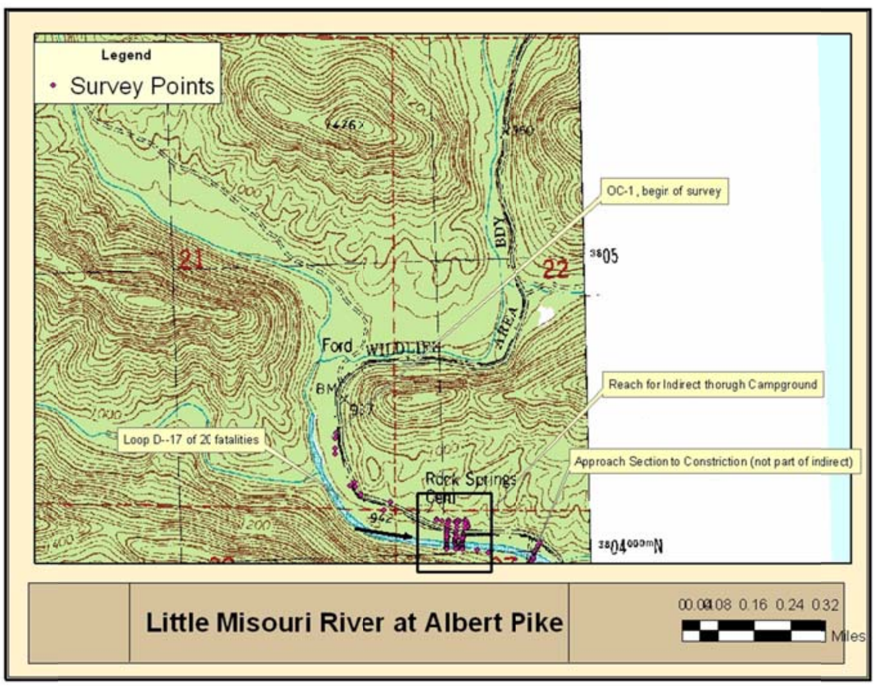
The site is Approximately 5.04 miles northwest of Langley, Arkansas, 12.55 miles southwest of Norman, Arkansas, 15 miles west-southwest of Caddo Gap, Arkansas, and 31 miles east of Cove, Arkansas. Arkansas Highway 369 comes from Langley north to Albert Pike Recreation Area. At the entrance to Albert Pike, AR Highway 369 ends and County Road 4 (AKA USFS Road 73) continues in the left bank floodplain of the Little Missouri River, which includes the indirect measurement reach Approximately 700 feet downstream of the downstream most cross section for the indirect, there is a low water bridge for County Road 106 and allows access to the right bank and Areas A and B of the Campground. County Road 220 (from specialty USGS TOPO for Forest Service) splits off to the south and proceeds through Loop B of the campground and follows the right bank downstream into the private Lowery’s Albert Pike RV Park. County Road 106 continues in a southwesterly direction and goes up hill and eventually ends running along Blaylock Creek.

<u>Survey of site:</u> Site was selected on Sunday June 13, 2010 by Robert Holmes during reconnaissance of the flood area. High water marks were flagged on the morning of June 14, 2010 by Robert Holmes. A survey commenced in the late afternoon of June 14, 2010 by Paul Rydlund, Larry Buschman, and Robert Holmes. Ferrell Killian joined the survey party on the morning of June 15, 2010. The initial occupation point (OC-1)was a pin driven into the asphalt road upstream of Area D near the area that served as the command center during the search and rescue operations. This location is approximately 0.36 river miles upstream of the indirect measurement site. An arbitrary Northing/Easting of 5000/5000 was assumed with an elevation of 100. The azimuth was established with a compass bearing of magnetic north. After the point was vacated by the total station survey, a Trimble GPS unit was setup to occupy the point for several hours to establish the true horizontal and vertical position of each survey point.   The UTM Zone 15 coordinates and NAVD88 Elevation for OC-1 is:

Elevation NAVD88	Northing (UTM Feet)	Easting (UTM Feet)
933.28	12482340.14	1373363.85

High water marks were surveyed along the left floodplain from OC-1 to the location of the upstream cross section of the indirect measurement. The survey was continued downstream of the indirect measurement site in order to capture additional high water marks along the river as well as the channel geometry of the constricted area.

The Survey of the indirect measurement site was made using a Nikon DTM-450 2-second total station, serial number 111304.



<u>Discharge and Gage Height:</u> 40,100 cfs, rated at fair. No Gage Height as this is a miscellaneous site.

<u>Drainage area:</u> 34.1 mi^2 as determined by Albert Rea of the USGS using NHD Plus. This value was verified by the USGS Arkansas Water Science Center.

<u>Unit Discharge:</u> 1,180 cfs/mi^2. Crippen and Bue (1977) envelope curve for this region (Region 8) is below, with this flood approximately plotted. You will note that this flood plots on the envelope.

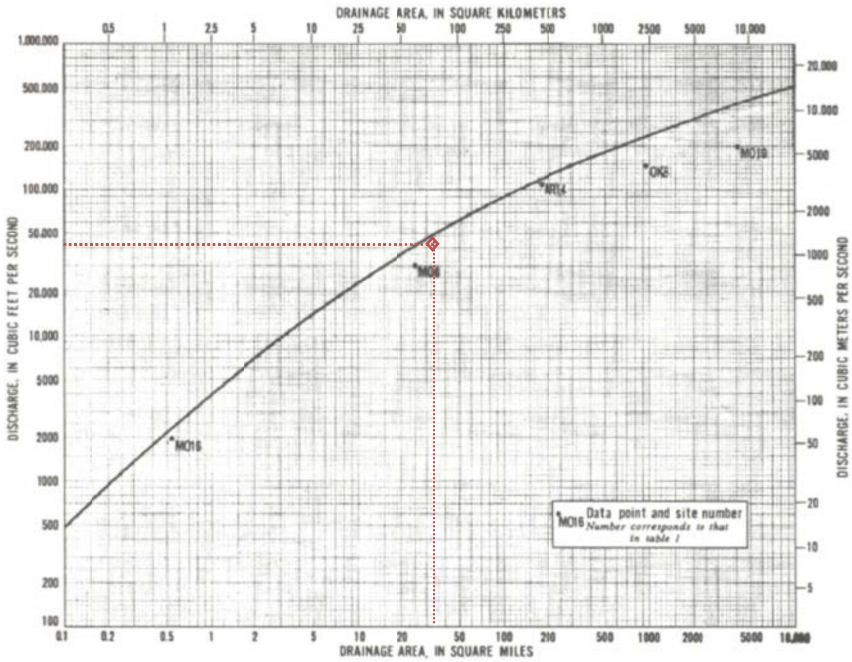


FIGURE 10.—Peak discharge versus drainage area, and envelope curve for region 8.

Nature of flood: This flood was a flash flood of extreme nature. As much as 7 inches of rain fell in a very short period of time starting just before or at midnight on June 10. Rates of rise at the USGS streamgage 10 miles downstream were as much as 8 ft/hour. Anecdotal accounts of the rates of rise from survivors in the Albert Pike Campground indicate as much as 3 feet in a few minutes. The Ouachita Mountains are a known "flood hotspot" in the United States due to their steep topography and proximity to the Gulf of Mexico (GOM) moisture source. Moisture-laden air masses travel north from the GOM over the Coastal plain with Orographic lifting occurring as the mass meets the Ouachita Mountains of southwestern Arkansas. The orographic lifting can produce intense rainfall. The intense rainfall on steep slopes results in large peak streamflows.

Field conditions: The indirect reach is fairly straight with nearly uniform boundary roughness through the reach. There was no evidence of scour or fill in this reach. Each of the three cross sections was subdivided into 2 subsections: 1) the main channel, which included the two bank areas with the trees, and 2) the left overbank, which is a panhandle subsection of the total cross section. XS1 is the most upstream cross section, XS2 is the middle cross section, and XS3 is the most downstream cross section.

The left overbank is loop C of the campground which was empty of campers the night of the flood. There are scattered trees in the overbank area along with a road that runs parallel to the stream channel. There are constructed picnic areas through this left overbank area with some treated lumber fencing that does not span wide distances. See 2 video clips: "Little_MO_LOB_Indirect_campground_1.MP4 " and "Little_MO_LOB_Indirect_campground_2.MP4 " for a good view of this left overbank area. A composite n value for the left overbank was estimated to be a 0.038 from engineering judgment, the modified Cowan method for floodplains outlined in Arcement and Schneider (1984), and

Petry and Bosmajian (1975) method outlined by Arcement and Schneider (1984). The decision to place it at 0.038 influenced by the desire to match the left overbank velocities near to 9.0 ft/s as estimated from stagnation pressure computations from the runup on the bathroom building in the left overbank 100 ft downstream of the indirect reach.

Looking from upstream diagonally to the XS2. Note location of high water marks as shown by hydrographer holding rod horizontally.

Looking from XS1 downstream through Left overbank area

The main channel has a rock bottom with very little gravel or other mobile material in the middle and downstream cross sections, but gravel just upstream of the upstream xsect. Upstream of the most upstream cross section Trees and brush were thick on the immediate left bank side before the thickness ended as one progressed into the left overbank which is shown above. The right bank side was completely wooded as it the right overbank was essentially the valley wall. There is very little sign of any channel instability, scour or fill having taken place during this flood. For a look at the main channel on video, see "Little_MO_Main_Indirect_campground_1.MP4 ". A composite n value of 0.053 was assumed based on engineering judgment, weighting of n values from channel width, examination of photos from Barnes (1967, http://pubs.usgs.gov/wsp/wsp_1849/pdf/wsp_1849.pdf), and use of theoretical equations. Bed material size data came from the gravel bar 700 feet below the reach and just below the low water bridge.

As noted, gravel was not as prominent in the indirect reach, but the assumption was made that the presence of gravel would serve to only increase the roughness value of n and the absence of gravel would indicate smoother n values. It is noted that gravel is transported through this reach and is deposited downstream below the low water crossing.

Looking from left bank upstream to XS1 where rodman is in middle of main channel. Note the boulders that are visible are not present in the main channel in the indirect reach

Looking at channel bottom at XS3 from left bank looking to right bank

From XS1 looking downstream, note some gravel on the bed of this xsect

From trail on right bank valley wall at XS1 looking to leftbank and main channel.

The high water marks were of generally fair to good condition. Because of high velocities, relatively few high water marks were found. Mudlines and/or washlines were found at locations on both banks. A few excellent high water marks were found on sign boards in the campground.

Right Bank High Water Marks

Mark Name	Station	NAVD Elevations
RH-1	262.6	924.68
RH-2-F-WSH	401.6	923.56
RH-3-E	425.0	923.75
RH-4-P	469.0	924.02
RH-5-F	470.1	923.32
RH-6-F	644.1	922.93
RH-7-E	788.0	922.3
RH-8-F	1129.9	919.78

Left Bank High Water Marks

Mark Name	Station	NAVD Elevations
LH-7-E	185.0	924.42
LH-8-F	187.2	925.36
LH-9-F	252.4	925.15
LH-10-F	409.3	925.03
LH-11-P	461.8	924.71
LH-12-F	539.8	922.69
LH-13-G	554.5	922.42
LH-14-E	556.1	922.68
LH-15-F	559.4	922.71

Computations:

SAM 2.1 was used to process the Total Station Survey data and ready it for input into the SAC program (Slope Area Computation). The discharge computed for this measurement was 40,100 cfs. The reach was contracting from X1 to X2 and slightly expanding from X2 to X3. Following are the output diagnostics from SAC.

Reach	dH,fall (ft)	length (ft)	Discharge (cfs)	Spread (%)	HF (ft)	CX	RC	RX	ER
X1 - X2	0.65	105.	38124.	0	0.378	1.000	0.720	0.000	#
X2 - X3	0.50	100.	43163.	3	0.518	0.982	0.000	-0.070	#
X1 - X3	1.15	205.	40088.	1	0.865	0.993	0.348	-0.036	#

Definitions:

Spread: the percent difference between discharge computed with no expansion loss (k=0) and discharge computed with full expansion loss (k=1.0), divided by the discharge computed with full expansion loss

HF: friction head which is the sum of Q*Q*L/(K1*K2) over subreaches

L: reach length; K1, upstream section conveyance;

K2: downstream section conveyance

CX: the computed discharge divided by the discharge computed with no expansion loss (k=0)

RC: velocity head change in contracting section divided by friction head

RX: velocity head change in expanding section divided by friction head

Sensitivity Analysis:

Sensitivity analysis was conducted on the n values and water surface slope. The n values were varied +/- 10% to see the impact on the final flow result. A +/- 10% variation in n values resulted in -6.5% to +7.7% difference from the accepted flow value of 40,100 ft^3/s. The accepted fall value of 1.15 ft was lowered to1.00 ft (-13% change) which resulted in a -4.0 % change in the accepted flow value of 40,100 ft^3/s.

Evaluation:

Use 40,100 cfs and consider it fair reliability. The indirect is downgraded from good to fair based on the low number of high water marks, short reach lengths, and only 3 cross sections. Additional factors supporting the quality of this measurement include the following:

1. The high water marks were flagged and surveyed within 4 days of the flood. A good wash line was evident on the valley walls and a few excellent high water marks were found in the display cases.
2. Although there was slight expansion from X2 to X3, the diagnostics (Spread near 0, CX approaching 1, and small values of RX) indicate that expansion is not an issue. The velocity head change from XS1 to XS3 is about 33% of the friction head in this reach. The lower this ratio the more accurate the measurement per Kirby (1987).
3. There is little evidence that the reach cross section main channel has changed much during the flood. The channel is remarkably stable. The left overbank had minor erosion/deposition of sands, gravels, and fines from the ground being bare from construction. The impact to any cross sectional change is negligible.
4. There is no evidence that this flood was a debris flow based on evidence left behind such as scouring and deposition.
5. An estimate of velocity at the bathroom house 100 feet downstream of XS3 in the left overbank was made from measuring the difference in the water surface elevation on the front of the building and inside the electrical room. The difference was 1.25 ft. Equating that to a stagnation point, the 1.25 would be equal to the velocity head at that point. Computing the

velocity from $v^2/2g = 1.25\,ft$ provided and estimate of v = 9.0 ft/s. As the SAC sensitivity analysis results were evaluated, some weight was giving to those combination of parameters that allowed for a velocity in the overbank that matched closest to 9.0 ft/s.

6. As a check, it was thought that the constriction 1660 ft downstream of XS1 would induce critical flow depth. As such the survey team collected cross section data at the constriction and an approach section. Using HWMs at the constriction, the mean hydraulic depth (D=A/T) computes as 12.02 with a cross sectional area of 2,874 ft2. From critical flow equation ($V = \sqrt{gD}$, the flow for the measured depth was 56,541 cfs, which way overestimated the flow given from SAC. However, the extra cross sections were used to run a second independent SAC computation for XS3, XS4, and XS5 (XS5 is the constriction and XS4 is the approach). The SAC computation for these three section, although dealing with some expansion issues, yielded a discharge estimate of 40,600 cfs, which is within 1.2% of the 40,100 estimated for this measurement.
7. Superelevation was noted in the curved reach which is 1500 feet downstream of the indirect reach. The superelevation was used to estimate the mean velocity (Chow, 1959) at 7.22 ft/s. Multiplying this value by the XS4 area estimates the discharge at 38,100 cfs, which is within 5% of the 41,100 cfs estimated for this measurement.

<u>Previous computations:</u>
None

<u>Remarks:</u>
Riggs (1976) method estimates the Q at 51,000 ft^3/s, although its use its validity is questionable when water surface elevations far exceed bank-full magnitudes.

Response to selected review comments of Rodney Southard. Southard comments as numbered item, Holmes response in *italics*:

1. GPS equipment was used to establish vertical and horizontal control at the point of origin for the indirect survey. Suggest documenting the coordinates of this point in the indirect report and water surface elevations used in the analyses. It would be beneficial to translate all points to real world coordinates for future reference.

   I have noted the UTM Zone 15 NAD83 horizontal location and NAVD88 vertical elevation of OC-1 in the summary. All points were already translated to real world coordinates in the file "Little_MO_Campground_RydlundIndirects_post alignment.xlsx"

2. Thalweg slope is 28.3 ft/mile estimated water surface slope used is 29.6 ft/mile. Also, the main channel is eccentric with respect to the floodplain. There were also large movable obstructions on the left floodplain such as trucks and RV's that might give misleading HWM's resulting in the inconsistent marks recovered. The water surface profile on the right bank is more consistent and resulted in a slope of 28.3 ft/mile per high water marks found. Suggest using 28.3 ft/mile or 1.1 ft/mile for the fall for the reach. From the high water mark profile plotted the following water surface elevations would be: X1 = 91.15, X2 = 90.6, and X3 = 90.05.

   Determination of the true channel slope is difficult whether it be from a thalweg field survey, picking points from DRG, or cutting a profile in ARCGIS from the DEM. The difference between the water surface slope (fall) in the original computation (29.6 ft/mile) is well within the error in estimating the local channel slope. As such, I have chosen to stay with the original channel slope of 29.6 ft/mile.

3. Subarea break between the left overbank and main channel is at 165.3 stationing for cross section 1 (X1). Geometry data for this cross section and comparison to X2 and X3 the subarea break would be more representative at station 137.9. Perhaps a subarea break even at station 156.5 would be suitable depending on slope area output results.

   Subarea break point was located to coincide with a boundary roughness change. Mark Smith re-computed the indirect with the same suggested subdivision of X1, with the result that it only changed the discharge by less than -0.8%. No changes to the subarea break point was made.

4. Stagnation point computations at the bathroom below X3 to estimate the velocity of the floodwater on the left overbank may be suitable if equilibrium was reached at the peak stage. Were there any constrictions to flow for floodwater to enter the bathroom? Considering the rapid change in stage in this reach was the equilibrium reached for the very peak stage? Did the HWM elevation at the bathroom compare well (+/- 0.1 ft) to other HWM's in the vicinity?

   According to eyewitnesses, the peak stage lasted a pretty long time. As such, I believe it long enough to reach equilibrium. As to whether, the marks inside the power room agreed with other marks in the vicinity, we did not survey the marks inside the restroom, nor the runup mark on the front of the building. Instead, we used a hand level to transfer the marks from the electrical room to the concrete masonry block on the outside of the building in order to get an elevation difference between the runup and the value inside.

5. There is a streamgage downstream of the indirect site on the Little Missouri River (07360200). What was the peak flow at the streamgage? May want to compare peak values/unit runoff for consistency.

   I have been comparing all the indirects for unit discharge to evaluate consistency, including the Little Missouri River near Langley, Arkansas (07360200). Final discharge for the Langley gage has not been agreed too as of this writing.

6. In one of the pictures taken in the field, a U.S. Forest Service sign designated locations as areas "A" or "B" etc., but the indirect write-up references loops? Are they representing the same locations? If so may want to be consistent with U.S. Forest Service terminology.

   Forest Service has been very inconsistent with use of terminology. Referring to the individual campgrounds as "Loops" in most of their correspondence with me, but indeed the reference sign map refers to "Area". I have revised the language to "Area" in the summary.

Response to selected review comments of Jaysson Funkhouser. Funkhouser comments as numbered item, Holmes response in *italics*:

1. Overall Manning's values are slightly "atypical" to the values that have been used with other indirects in Arkansas. Specifically:

2. Left overbank for XS1-3 of 0.038 seems low. Especially with the obstacles (vehicles, etc.) that the water flowed around. There was only 10 – 12' of water flowing over the floodplain (fairly shallow considering the depth of the channel);

   Given the low tree density and the "swept clean" look of the ground, I consider an n value of 0.038 to be reasonable. In addition, 10 to 12 feet of flow would not be considered shallow when comparing it to many of the n value verification studies in the literature. As a last refining check, I used the runup on the bathhouse as a check of the velocities in the overbank. The runup indicated velocities in the 9 ft/s range. As noted in the SAC output, use of the overbank composite n = 0.038 yielded overbank velocities in the 9 ft/s range.

3. The Manning's values seem high for the main channel. There were many boulders present and the channel meanders, but there was 25' of water flowing through the channel.

   The main channel n value is a composite n value that includes the impact of the vegetation on both banks. The n value for the main channel was evaluated through a variety of means as noted in the summary and computations. The influence of the vegetation on both banks played a large role in *increasing the roughness value. The overall main channel width was 148 feet, of which 75 feet (50%) was trees and dense vegetation. I feel that 0.056 is a reasonable value for the n value.*

4. The floodplain is around 300' wide and the cross sections for the indirect were about 100' apart. "Reach too short" error was generated from SAC. Because of this situation and the numerous obstacles the water flowed around and the steep slope, consider rating this indirect down from "good" to "fair to poor".

   A "reach too short" is more of a warning than an error. It is automatically given to any reach that is less than 75 times the mean depth per page 4 of TWRI Book 3 Chapter A2. As will be noted, 75 times mean depth is one of three possible criteria to meet. As noted in the TWRI, "one or more of the three criteria should be met when possible". It is clear in this reading that all three requirements are not necessary to be met, and as such are not be the sole basis to downgrade the measurement. However, the measurement was downgraded to "fair" from "good" based on small number of available high water marks, only three cross sections, and short reach lengths.

References:

Arcement, Jr., G. J. and Schneider, V.R., 1989, Guide for selecting Manning's roughness coefficients for natural channels and flood plains, U.S. Geological Survey Water Supply Paper 2339,

Barnes, Jr. H. H., 1967, Roughness characteristics of natural channels, U.S. Geological Survey Water Supply Paper 1849, 213 p

Kirby, W.R., 1987, Linear error analysis of slope-area discharge determinations, Journal of Hydrology, Volume 96, pp125-138

Riggs, H.C., 1976, A simplified slope-area method for estimating flood discharges in natural channels, U.S. Geological Survey Journal of Research, Volume 4, Number 3, pp 285-290

Computed By: Robert R. Holmes, Jr. PhD, P.E., D.WRE
National Flood Specialist
Date: July 14, 2010

Reviewed By: Rodney E. Southard
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Date: August 10, 2010

Check By: Jaysson Funkhouser, P.E.
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Reviewed By: Mark Smith
Regional Surface Water Specialist
Date: September 3, 2010
Approval: Mark E. Smith
Surface Water Specialist,
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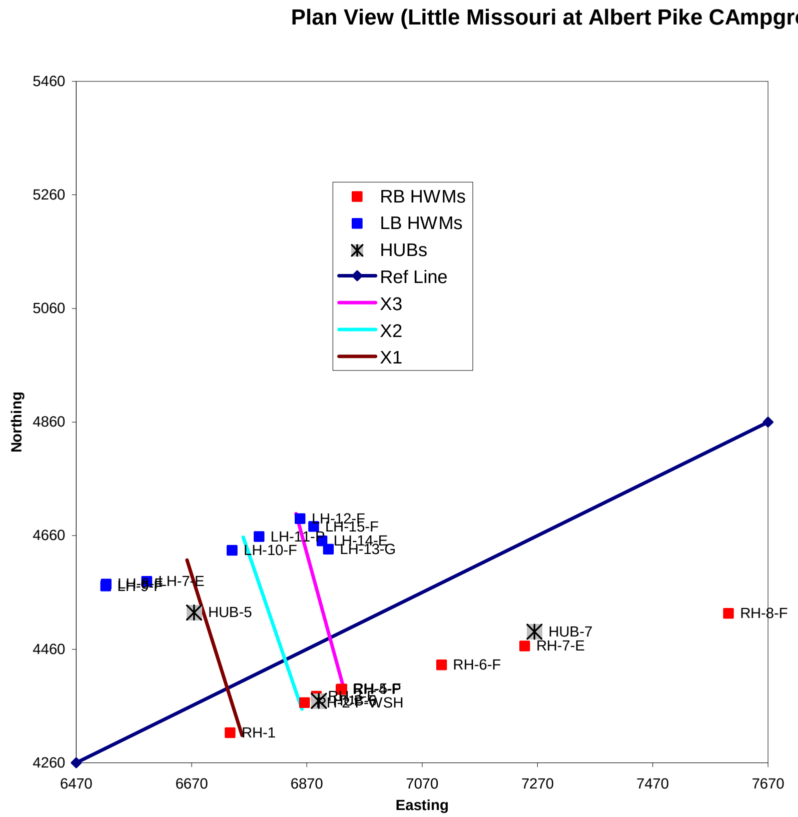
Date: January 21, 2011

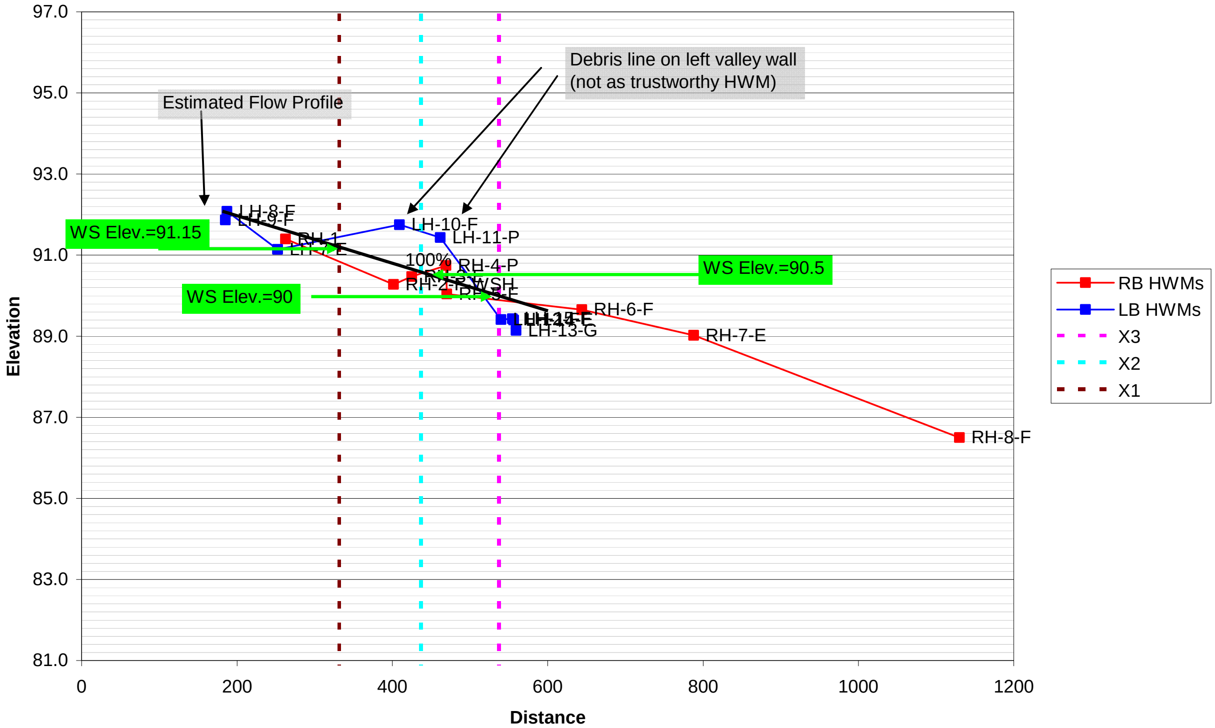
Appendices

Little Missouri River at Albert Pike, Arkansas
Miscellaneous Site
Flood of June 11, 2010

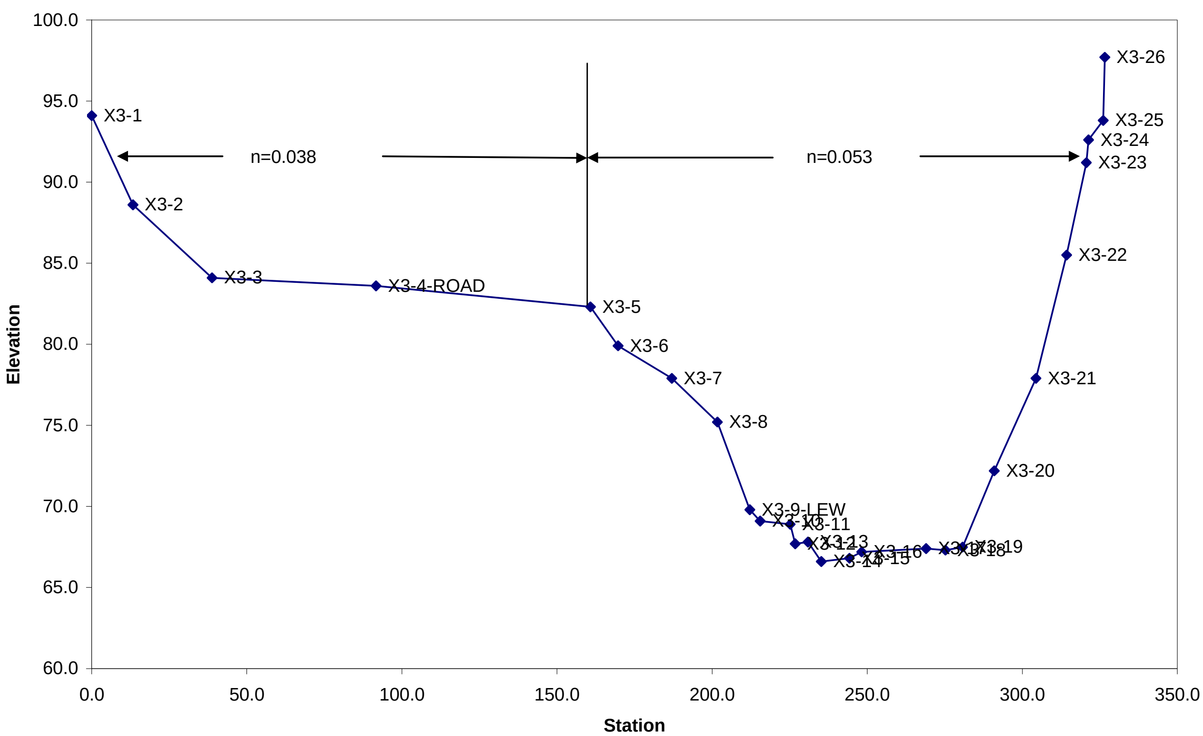
Graphs from SAM



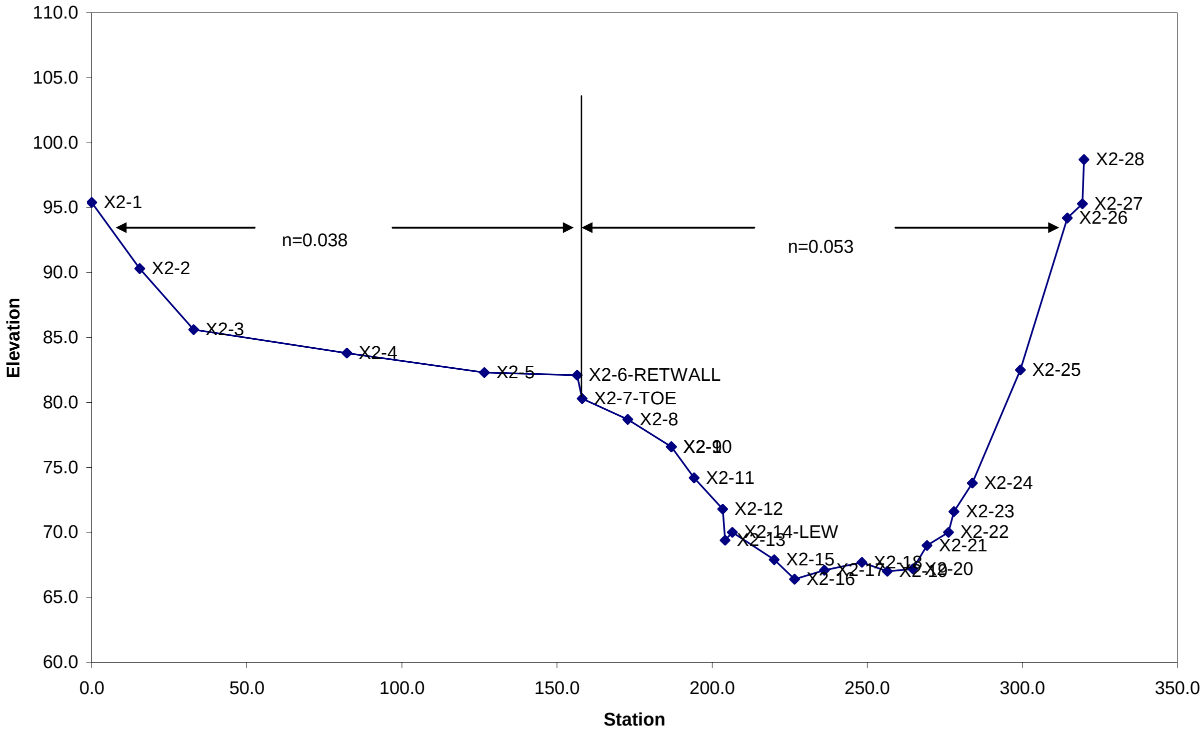
High Water Marks Profile (Little Missouri at Albert Pike CAmpground/9991)

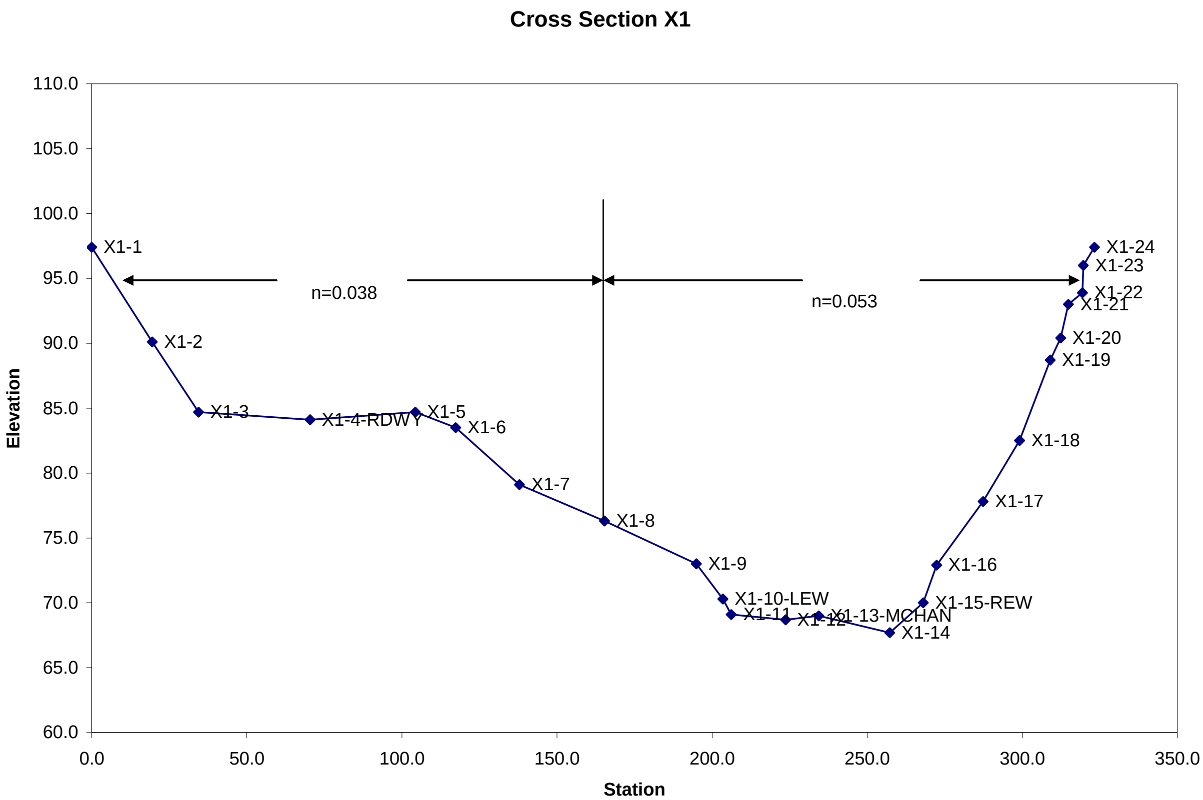


Cross Section X3



Cross Section X2





Roughness Estimates

Left Overbank

Method 1—Modified Cowan method for floodplains (see below for computation) method 1

Arcement and Schneider (1984) pp 3-17

$n = (n_b + n_1 + n_2 + n_3 + n_4)\, m$ ← modified method for floodplains

$n_b = 0.020$

$n_1 = 0.001$

$n_2 = 0$

$n_3 = 0.005$ ← obstruction

$n_4 = 0.035$ ← vegetation

$m = 1$

$n = (0.02 + 0.001 + 0 + 0.005 + 0.035)\, 1$

$= 0.041$

Method 2---Petry and Bosmajian (1975) $n = n_0\sqrt{1+\left(\frac{C_*\sum A_i}{2gAL}\right)\left(\frac{1.49}{n_0}\right)^2 R^{4/3}}$ (see below for computation)

method 2

Arcement + Schneider (1984) pg 17

$n = n_0\sqrt{1+\left(\frac{C_*\,\Sigma A_i}{2gAL}\right)\left(\frac{1.49}{n_0}\right)^2 R^{4/3}}$

$n = 0.026\sqrt{1+\left(\frac{(4)(65)}{(2)(32.2)(1000)65}\right)\left(\frac{1.49}{0.026}\right)^2 6.5^{4/3}}$

$n = 0.0484$

$R = 6.5'$

$n_0 = 0.026$

$g = 32.2\ ft/s^2$

$A = 1000\ ft^2$

$C_* = 4$

$L = 205'$

$\Sigma A_i = 65\ ft^2$ ← estimate 10 trees 1 ft wide for depth 6.5'

Decision was made to use 0.038 for the left overbank based on reviewing the sensitivity analysis, the above two quantitative estimates, pictures from Arcement and Schneider, and the fact that a velocity in the left overbank of 9 ft/s was estimated from the 1.25 ft of runup on the LOB bathroom. This uses the stagnation pressure estimate of $v^2/2g = 1.25\,ft$.

Main Channel

Main channel was estimated by doing a weighted (by width) estimate of the main channel composite n value. The main channel had brush and trees on the left shore (33 ft in width), a rock and gravel bed (gravel was minor) low water channel (73 ft in width) and a densely treed right shore up a steep bank into the valley wall (42 ft in width).
The left shore was estimated to be n= 0.080.
The right shore was estimated to be n = 0.080.
The low water channel was estimated to be 0.031. This was based on published n values in Chow (1959), engineering judgetment, and several theoretical equations based on bed material. The Bed material size data came from the gravel bar 700 feet below the reach and just below the low water bridge. As stated, the gravel was not present in large quantities, but gravel would be in transit during the peak flows as deposits are predominant downstream. The data for the theoretical equations are as follows:

D90= 0.15 meters
D84=0.38 ft
D50 = 0.15 ft; 0.048 m
S (friction slope)=0.0043
R = 16 ft; 4.9 meters
T=148 ft,

Following are computations:

Meyer-Peter Muller(1948) equation for gravel bed streams $n = \frac{1}{26} d_{90}^{1/6}$ (metric units) ---n = 0.028,

Jarretts (1984) equation for mountain streams $n = 0.47 S^{0.38} R^{-0.16}$ (English units) --- n = 0.038;

Limerinos (1970) for gravel streams in California $n = \frac{0.0926 R^{1/6}}{1.16 + 2.00 \log \frac{R}{d_{84}}}$ (English units) ---n =0.033;

Froelich's (1978) equation for gravel and cobble streams $n = 0.245 R^{0.14} \times \left(\frac{R}{d_{50}}\right)^{-0.44} \left(\frac{T}{R}\right)^{-0.3}$ (English units)----- n = 0.024;

Griffiths (1981) equation for gravel rivers $n = \frac{0.113 R^{1/6}}{0.76 + 1.98 \log \frac{R}{d_{50}}}$ (all SI units) ---- n = 0.031.

A weighted value of n for the main channel is 0.056.

Width(W)	n	W*n
33'	0.080	2.616
73	0.031	2.263
42'	0.080	3.36
147.7		8.239

$$n_c = 8.239/147.7 = 0.056$$

For this indirect, a n value of 0.053 was used for the main channel. This was based on engineering judgment, weighting of n values from channel width (above), examination of photos from

Barnes (1967, http://pubs.usgs.gov/wsp/wsp_1849/pdf/wsp_1849.pdf), and review of the sensitivity analysis data.

SAC INPUT DATA

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T1      SAC/WSPRO Input for Little Missouri at Albert Pike Campground  (9991)
T2
Q
WS      89.7
XS   X3   537
GR      0,94.1 13.3,88.6 38.8,84.1 91.7,83.6 160.8,82.3 169.7,79.9 187,77.9
GR      201.7,75.2 212.2,69.8 215.5,69.1 225.2,68.9 226.8,67.7 230.9,67.8
GR      235.2,66.6 244.3,66.8 248.2,67.2 269,67.4 275.2,67.3 280.7,67.5
GR      291,72.2 304.4,77.9 314.3,85.5 320.7,91.2 321.4,92.6 326.1,93.8
GR      326.6,97.7
N       0.038   0.053
SA      160.8
HP 4 X3   90.0
XS   X2   437
GR      0,95.4 15.5,90.3 32.9,85.6 82.3,83.8 126.6,82.3 156.5,82.1
GR      158.1,80.3 172.8,78.7 186.9,76.6 186.9,76.6 194.2,74.2 203.4,71.8
GR      204.2,69.4 206.5,70 220,67.9 226.6,66.4 236.2,67.1 248.3,67.7
GR      256.5,67 264.7,67.2 269.3,69 276.2,70 278,71.6 283.9,73.8 299.4,82.5
GR      314.5,94.2 319.4,95.3 319.9,98.7
N       0.038   0.053
SA      158.1
HP 4 X2   90.5
XS   X1   332
GR      0,97.4 19.5,90.1 34.4,84.7 70.4,84.1 104.3,84.7 117.3,83.5
GR      137.9,79.1 165.3,76.3 194.9,73 203.5,70.3 206.2,69.1 223.7,68.7
GR      234.4,69 257.3,67.7 268.1,70 272.4,72.9 287.4,77.8 299.1,82.5
GR      309,88.7 312.4,90.4 314.9,93 319.4,93.9 319.7,96 323.3,97.4
N       0.038   0.053
SA      165.3
HP 4 X1   91.15
```

SAC OUTPUT

DISCHARGE COMPUTATIONS

Reach	dH,fall (ft)	length (ft)	Discharge (cfs)	Spread (%)	HF (ft)	CX	RC	RX	ER
X1 - X2	0.65	105.	38124.	0	0.378	1.000	0.720	0.000	#
X2 - X3	0.50	100.	43163.	3	0.518	0.982	0.000	-0.070	#
X1 - X3	1.15	205.	40088.	1	0.865	0.993	0.348	-0.036	#

Definitions:

Spread, the percent difference between discharge computed with no expansion loss (k=0) and discharge computed with full expansion loss (k=1.0), divided by the discharge computed with full expansion loss

HF, friction head- HF = sum of Q*Q*L/(K1*K2) over subreaches; Q, discharge; L, reach length; K1, upstream section conveyance; K2, downstream section conveyance

CX, the computed discharge divided by the discharge computed with no expansion loss (k=0)

RC, velocity head change in contracting section divided by friction head

RX, velocity head change in expanding section divided by friction head

ER, warnings, *-fall <' 0.5ft, @-conveyance ratio exceeded, #-reach too short error, 1-negative or 0 fall

******, terms that can not be computed because' of strong expansion in reach

CROSS SECTION PROPERTIES

I.D. X3 Velocity head 2.02ft Discharge 40088.cfs

Ref.distance 537.ft Q/K 0.0045 Alpha 1.048

Sub area no.	Water surface el.(ft)	n	Area (sq.ft)	Top width (ft)	Wetted perimeter (ft)	Hydraulic radius (ft)	Conveyance x 0.001 (cfs)	%	Vel. (fps)	F
1	90.00	0.038	907.9	150.9	151.6	5.99	117.422	20.	8.7	0.62
2	90.00	0.053	2688.2	158.6	167.7	16.03	480.499	80.	12.0	0.51
Total	90.00	---	3596.	309.	319.	11.26	597.921	100.	11.1	0.58

I.D. X2 Velocity head 2.05ft Discharge 40088.cfs

Ref.distance 437.ft Q/K 0.0044 Alpha 1.041

Sub area no.	Water surface el.(ft)	n	Area (sq.ft)	Top width (ft)	Wetted perimeter (ft)	Hydraulic radius (ft)	Conveyance x 0.001 (cfs)	%	Vel. (fps)	F
1	90.50	0.038	924.0	143.2	144.7	6.38	124.693	21.	9.0	0.62
2	90.50	0.053	2633.3	151.6	161.2	16.34	476.612	79.	12.1	0.51
Total	90.50	---	3557.	295.	306.	11.63	601.305	100.	11.3	0.57

I.D. X1 Velocity head 1.75ft Discharge 40088.cfs

Ref.distance 332.ft Q/K 0.0036 Alpha 1.018

Sub area no.	Water surface el.(ft)	n	Area (sq.ft)	Top width (ft)	Wetted perimeter (ft)	Hydraulic radius (ft)	Conveyance x 0.001 (cfs)	%	Vel. (fps)	F
1	91.15	0.038	1192.3	148.6	150.4	7.93	185.849	28.	9.3	0.58
2	91.15	0.053	2616.0	147.8	154.0	16.98	485.917	72.	11.1	0.46
Total	91.15	---	3808.	296.	304.	12.51	671.765	100.	10.5	0.52

Definitions:

n, Manning's coefficient of roughness   Q/K = discharge/conveyance

F, Froude number F = Ki*Q/(K*A sqrt(g*(Ai/TWi)); Q, discharge; A, total cross-section area; g, acceleration of gravity; Ai, sub-section area; TWi, sub-section top width