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EFFECT OF HIGHWAY ALIGNMENT ON FLOODING -A CASE STUDY

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A heavy rainfall occurred in northwest Louisiana in May, 1989. The rainfall was particularly severe between May 17 and 18. As a result, an extensive area was flooded. Questions linking the presence of Interstate I-49 with the extent of flooding were raised. The Tapalcat Bayou watershed at a point east of the town of Allen was used for the analysis. The temporal rainfall distribution was defined and the effective rainfall (ER) hyetograph using the SCS-CN method was computed. The watershed was divided into small, uniform subareas. Unit hydrographs, based on Snyder's model, were defined for each subarea. By means of convolution of the unit hydrographs with the ER hyetograph, the runoff hydrograph was obtained. This hydrograph was routed through a pool-level reservoir to determine water levels. Scenarios with and without I-49 were considered. A sensitivity analysis was carried out to determine the influence of various parameters.

INTRODUCTION

Whenever a civil engineering project is built, chances are that the system that surrounds the project location will change, in one way or another, with respect to its original state. This is particularly true for linear projects such as highways. Prior to the construction phase, studies are normally carried out to estimate the impact of the project on the system behavior, and to devise measures to minimize such an impact. Uncertainties remain, however, as the only opportunity to actually test the effects of the project on the system is when the project has already been built and is in operation.

A large rainfall event occurred between May 16 and 18, 1989, over an extensive area in northwest Louisiana. The event caused significant flooding in many cases, which in turn caused extensive damage to crops and infrastructure. Interstate highway I-49, whose segment near the township of Allen had been recently completed, was considered by some local residents as responsible for the extent of the flooding. As it turned out, however, the effect of the presence of I-49 on flooding in the Allen area was marginal. This paper describes the procedure followed to reach this conclusion.

Based on available information, no reliable discharge or stage records were made in the area during the flood of May, 1989. The only records available were those of rainfall corresponding to several stations located in the vicinity. Furthermore, there seem to be no historical records other than rainfall in the region. Therefore, a methodology applicable to ungaged watersheds was followed. The entire watershed draining to Tapalcat Bayou was divided into smaller watersheds, and for each one of them a unit hydrograph (UH) was obtained using Snyder's method. This UH was then applied to the rainfall that took place between May 17 and May 18, 1989, in order to obtain an inflow hydrograph to the low area (Halls Brake and Berry Brake) that drains into Tapalcat Bayou. This hydrograph was routed through a pool-level reservoir to determine water levels, taking into consideration a stage-discharge relationship for the flow along Tapalcat Bayou downstream of Berry Brake. Scenarios with and without I-49 were considered. Finally, a sensitivity analysis was carried out to determine the influence of key parameters on peak discharge and water levels.

GENERAL DESCRIPTION OF THE AREA

Main Topographic Features

The Tapalcat Bayou watershed at a point just east of Allen, Louisiana, is delimited by hills on the south, west, and north (with maximum elevations close to 350 ft (106.7 m) above mean sea level); and by a slightly elevated terrain on the east (with maximum elevations around 125 ft (38.1 m)). Two low marshy zones called Halls Brake and Berry Brake (with elevations less than 120 ft (36.6 m)) receive most of the water that drains from these areas. The marshy zones lead to Tapalcat Bayou, some 2500 ft (762 m) north of Allen.

The slightly elevated terrain on the east tends to follow a northwest-southeast direction, i.e., the same overall flow direction as Rocks Bayou, Cow Bayou and Tapalcat Bayou (Figure 1). The distance between a hypothetical line representing the axis of the elevated terrain and the foot of the hills just north of Allen is of the order of 5000 to 6000 ft (1524 m to 1829 m).

Interstate highway I-49 crosses the area following a northwest southeast direction. The embankment has an elevation slightly higher than 130 ft (39.6 m) at the top. The highway has numerous structures (such as box culverts) that allow free flow passage from one side to the other side of the embankment. Near Allen, I-49 runs parallel to Tapalcat Bayou, and the distance between the highway and the foot of the hills decreases to approximately 600 ft (182.9 m).

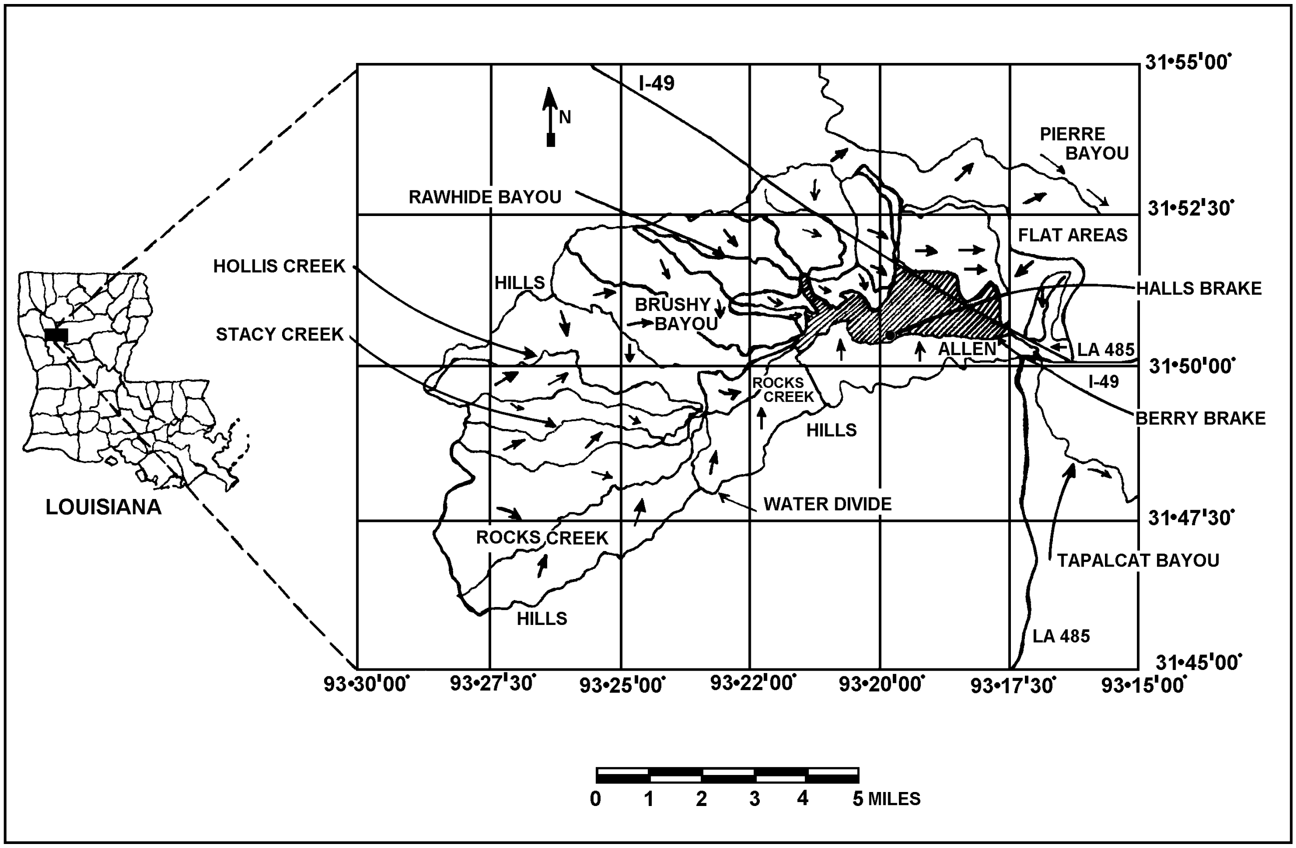

Figure 1. General location of the Tapalcat Bayou watershed and drainage pattern.

Drainage Pattern

The hills located on the south, west and north sides of the watershed are drained by a well defined network of creeks into Halls Brake. The major creeks are Rocks Creek, Brushy Bayou and Rawhide Bayou. Halls Brake drains into Berry Brake, which eventually feeds Tapalcat Bayou. In general, the flow tends to follow a southeast direction. Typical channel slopes are of the order of 0.2 to 0.4%.

The areas located on the northeast side of I-49 are drained by a poorly defined network of bayous into Cow Bayou which in turns drains into Tapalcat Bayou. Several minor creeks actually cross I-49 to drain into Halls Brake and Berry Brake, as well as into Tapalcat Bayou. In general, the flow tends to follow a south-southwest direction. However, the landscape on the northeast side of I-49 tends to be flat, which means that for high water levels the net flow direction may depend on the water levels on both Cow Bayou and Pierre Bayou.

RAINFALL ANALYSIS

Rainfall Distribution

Between May 16 and 18, 1989, heavy rainfall occurred on northwest Louisiana. The rainfall was particularly severe for a 24-hour period between May 17 and 18. As much as 10 inches (254 mm) of rain may have fallen over the area of interest during this 24-hour period, as derived from the isohyetal map shown in Figure 2. The hourly distribution of rainfall was computed using the data measured in Natchitoches, the only station among those surrounding Allen that provided hourly data. Table 1 shows the corresponding results.

In order to give an indication of the severity of this rainfall, the return periods for durations between 1 and 24 hours were computed. The TP-40 National Weather Service methodology was used for this

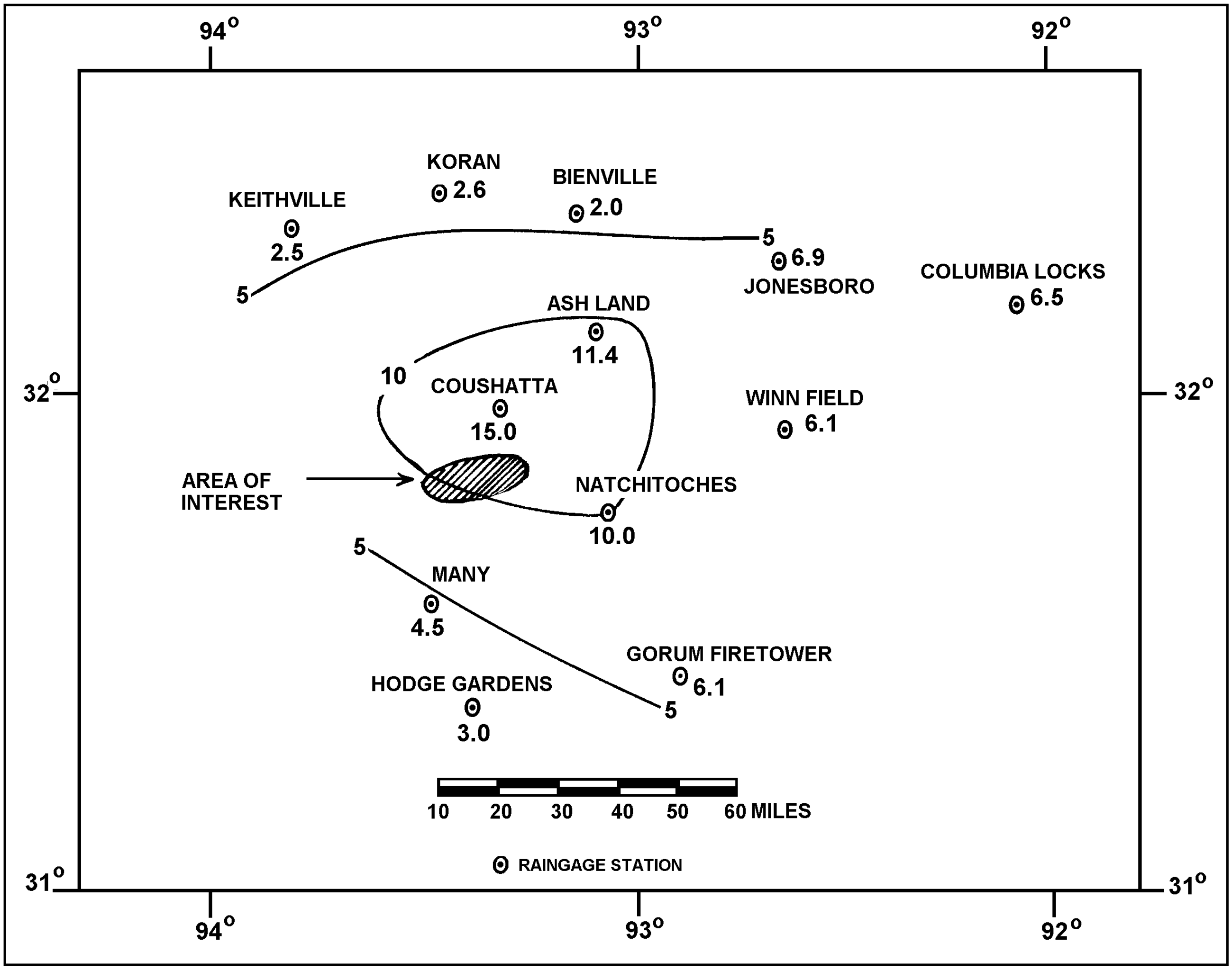

Figure 2. Isohyetal map for the rainfall of May 17-18, 1989.

purpose (Hershfield, 1961). As noted in Table 1, return periods varied between 5 and 10 years for the 1-hour duration, and between 50 and 100 years for the 24-hour duration. The bulk of the rainfall occurred during the first 12 hours, and for this duration, the return period turned out to be 100 years. By any account, the rainfall event was severe, even more so if one considers that in the days before the big storm, significant rainfall was reported in the area: nearly 6 inches (152 mm) in Natchitoches and Many, and 4 inches (102 mm) in Coushatta, between May 12 and 16.

Effective Rainfall (ER)

The SCS-Curve Number (SCS-CN) method was used for this purpose. An antecedent moisture condition AMC III was adopted, based on the amount of rainfall that occurred in the days before the event. A soil group C was assumed for the entire watershed, as most of the area is composed of moderately drained soils with a loamy surface layer and a clayey and loamy subsoil (SCS, 1985). The low, flat areas are composed of clayey soil throughout. The area is mostly woods and agricultural. As a result, a Curve Number CN of 86 was adopted.

The initial abstraction was assumed to be equal to 20% of the potential maximum retention. As a result, the effective rainfall ER turned out to be equal to 8.28 in. (210 mm). At points in time less than 24 h, the corresponding ER was computed using the cumulative total precipitation up to that point. Results are summarized in Table 1. The rainfall hyetograph and losses are shown in Figure 3.

Table 1. Hourly Total Precipitation (P) and Effective Rainfall (ER) over the Area of Interest

Day	Hour ending	Time (h)	P (in.)	Maximum Precipitation (in.) Duration 1-h	3-h	6-h	12-h	24-h	ER (in.)
17	1500	0			↑	↑	↑	↑	
	1600	1	2.0	2.7	4.7				0.849
	1700	2	2.7		↓	4.9			2.339
	1800	3							0
	1900	4							0
	2000	5							0
	2100	6	0.2			↓	9.0		0.186
	2200	7	1.9						1.800
	2300	8	1.7						1.643
	2400	9	0.2						0.195
18	0100	10	0.1						0.097
	0200	11	0.1						0.097
	0300	12	0.1				↓	10.0	0.097
	0400	13	0.1						0.098
	0500	14	0.1						0.098
	0600	15							0
	0700	16	0.2						0.195
	0800	17	0.1						0.098
	0900	18							0
	1000	19	0.1						0.098
	1100	20							0
	1200	21	0.2						0.196
	1300	22	0.1						0.097
	1400	23							0
	1500	24	0.1					↓	0.098
		Total =	10.0						8.28
				Return period (years)	5-10	25	5-10	100	50-100
				Maximum intensity (in./h)	2.70	1.57	0.82	0.75	0.42

Note: 1 in. = 254 mm

UNIT HYDROGRAPH DERIVATION

The entire area was divided into smaller, more uniform units, as shown in Figure 4. For each subarea, the following geometric characteristics were measured: Area (A); main channel length (L); distance from the watershed centroid to the outlet (L_c); difference in altitude between the most upstream point in the watershed and the outlet (H); average channel slope (S_m); time of concentration (t_c): and lag time (t_L).

S_m was computed by dividing the main channel of the subarea into several reaches, by computing the slope of each reach, and by taking the weighted average of all the slopes. t_c was computed using Kirpich's formula (Singh, 1992), and t_L was taken as $0.6t_c$. The watershed geometric characteristics are summarized in Table 2. For subareas XIII-a to XIII-d, the accuracy of the maps available (USGS 1:24000 maps) precluded a reasonable estimation of channel average slopes.

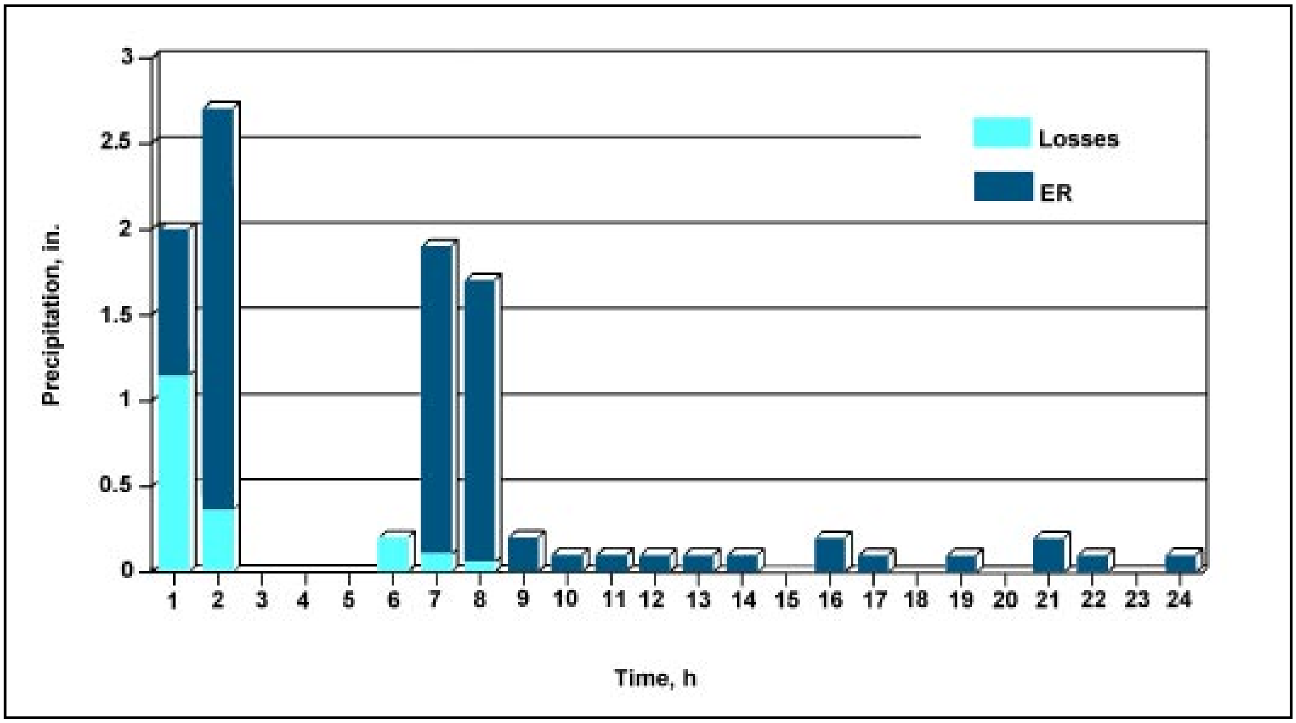

Figure 3. Rainfall and losses hyetograph. Effective rainfall (ER) by the SCS-CN method.

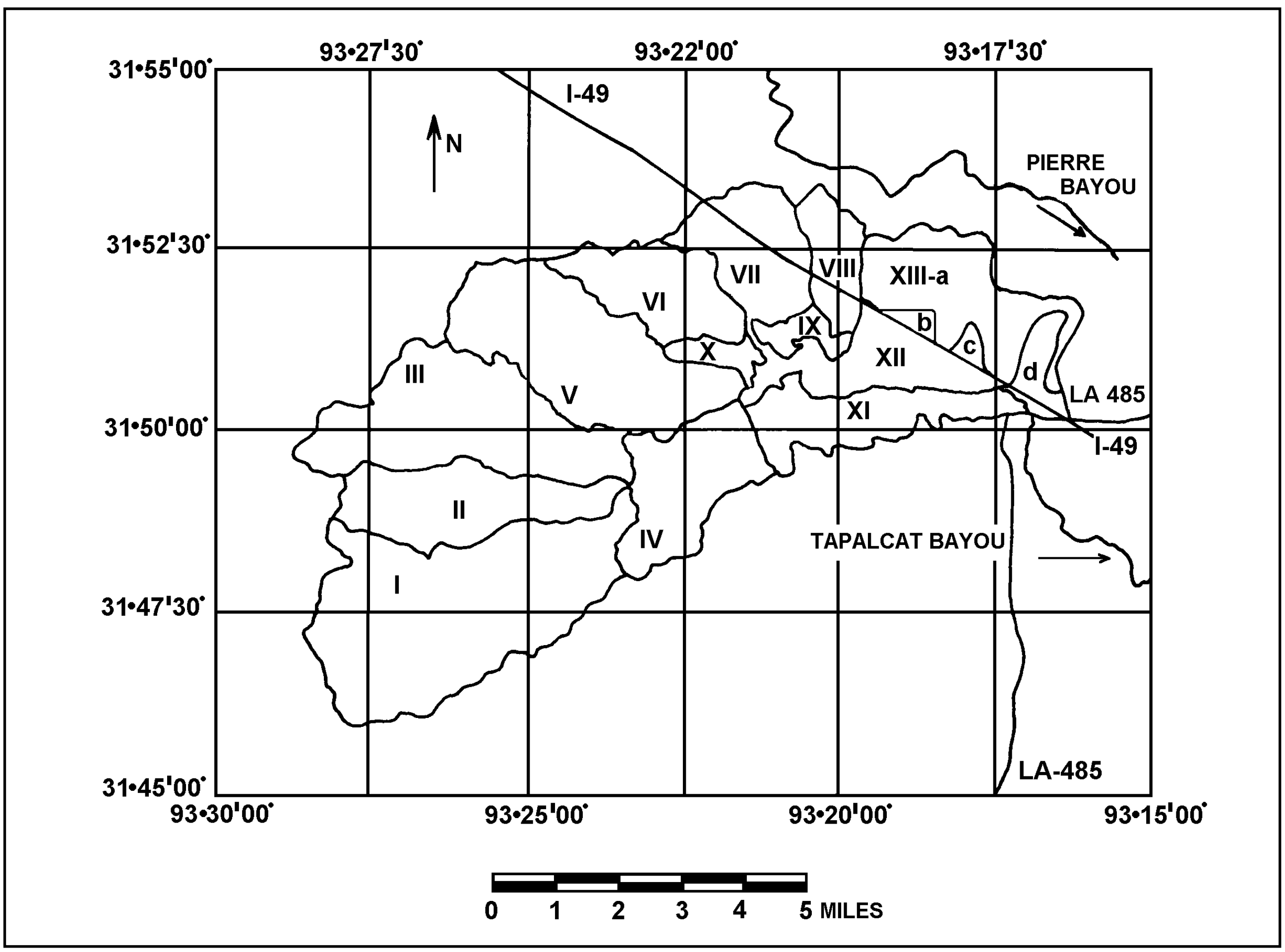

Figure 4. Subareas considered for unit hydrograph derivation.

Table 2. Geometric Characteristics of the Subareas of Figure 5

Subarea	Main Current	Area (acres)	L (ft)	L_c (ft)	H (ft)	S_m (%)	t_c (h)	t_L (h)
I	Rocks Creek	6697	44000	18000	213	0.336	3.70	2.22
II	Stacy Creek	2626	34000	14000	213	0.466	2.75	1.65
III	Hollis Creek	4342	41200	15000	213	0.400	3.43	2.06
IV	Rocks Creek	2381	17000	9000	19	0.112	3.11	1.87
I-IV	Rocks Creek	16046	61000	24000	232	0.261	5.22	3.13
V	Brushy Bayou	4434	38000	13500	162	0.302	3.47	2.08
VI	Rawhide Creek	1980	21600	8000	162	0.607	1.81	1.09
VII	(No name)	2068	23000	6000	102	0.370	2.32	1.39
VIII	(No name)	1098	14600	6500	92	0.426	1.43	0.86
IX	(No name)	410	9600	4000	112	0.954	0.82	0.49
X	(Several)	450	2200	1000	52	0.236	0.20	0.12
XI	(Several)	1972	5400	2000	82	1.49	0.48	0.29
XII	Rocks Bayou	1908						
XIII-a	Cow Bayou	2683	22500	8000	10		5.48	3.29
XIII-b	(Several)	156	2000	700	5		0.44	0.26
XIII-c	(Several)	174	4000	1200	7		0.86	0.52
XIII-d	(Several)	475	8600	3400	9		2.08	1.24
	Total =	33854						

Note: 1 acre = 0.4047 ha; 1 ft = 0.3048 m

These geometric characteristics were used to derive the Snyder unit hydrograph associated with each subarea. For computational purposes, the HEC-1 model (US Army Corps of Engineers, 1990) was utilized. The time interval chosen for modeling was 1 hour, which implied that, for each subarea, the unit rainfall duration was computed and then adjusted to 1 hour to make it equal to the effective rainfall time interval. Table 3 shows the results of the computational procedure. Areas I to IV were lumped into a single unit for hydrograph computations due to their similarity in geometric properties and drainage patterns, as shown in Table 2.

The use of the HEC-1 model imposed some limitations on the modeling approach. For instance, the HEC-1 model can handle only up to 300 hydrograph ordinates and uses a fixed time interval,

Table 3. 1-h Snyder's UH Parameters Using the HEC-1 Model

Subarea	t_p (h)	t_b (h)	C_p (h)	h_p (1/h)	q_p (cfs/mi^2)	Q_p (cfs)
IV	3.15	19	0.60	0.186	119.7	3002
V	2.10	13	0.60	0.268	172.6	1196
VI	1.08	6	0.60	0.469	300.3	929
VII	1.39	9	0.60	0.380	245.1	792
VIII	0.86	4	0.53	0.502	323.5	555
IX	0.79	3	0.50	0.502	323.1	207
X	0.79	3	0.50	0.498	321.4	226
XI	0.79	3	0.50	0.500	322.6	994
XIII-a	3.29	21	0.60	0.177	114.0	478
XIII-b	0.79	3	0.50	0.502	324.1	79
XIII-c	0.79	3	0.60	0.502	323.7	88
XIII-d	1.24	7	0.60	0.418	269.5	200

Note: 1 cfs/mi^2 = 1.094 (m^3/s)/ha; 1 cfs = 2.832x10^{-2} m^3/s which means that, in some cases, a trade off between total simulation time and time resolution may be required. In the case of the Tapalcat Bayou, one of the early objectives of the project was to determine flood duration times. As a result, the trade off would have meant the inability to detect the effect of some small subareas given their short unit hydrograph t_b and t_p values. To evaluate this situation, an alternative approach for defining the Snyder unit hydrograph associated with each subarea was considered. As before, the unit rainfall duration for each subarea was computed and then adjusted to 1 hour to make it equal to the effective rainfall time interval. The basic unit rain fall duration (D_0), in hours, was computed as

$$D_0 = \frac{t_L}{5.5} \tag{1}$$

Assuming t_p to be approximately the same as t_L, the corrected time to peak t^*_p in hours became

$$t_p^* = t_p + \frac{1 - D_0}{4} \tag{2}$$

Considering a peak coefficient C_p of 0.6, the peak ordinates of the l-h unit hydrograph were computed as

$$h_p(1/\mathrm{h}) = \frac{0.6}{t_p^*} \tag{3}$$

$$q_p\left(\mathrm{cfs}/\mathrm{mi}^2\right) = \frac{384}{t_p^*} \tag{4}$$

$$Q_p(\mathrm{cfs}) = A\, q_p \tag{5}$$

where A is the watershed area in mi^2. The time base was computed as 4.5 to 5.0 times t_p. The width in hours of the unit hydrograph at values of 50% (W_{50}) and 75% (W_{75}) of the unit hydrograph peak (q_p) was estimated as (Singh, 1992)

$$W_{50} = \frac{830}{q_p^{1.1}} \tag{6}$$

$$W_{75} = \frac{470}{q_p^{1.1}} \tag{7}$$

These parameters served as guides to define the shape of the unit hydrograph. In order to ensure that the area under the hydrograph was equal to one, several hydrographs were plotted and the area under the curve was computed; the procedure was repeated until the computed area was not significantly different from one. For the smallest subareas (X, XI, XIII-b and XIII-c), it was necessary to compute the corresponding S-curves and offset them by 1 hour to obtain their unit hydrographs. Table 4 shows the results of the computational procedure.

A comparison between the results of Tables 3 and 4 made differences between the two approaches evident, especially in regard to small subareas. In general, as the size of the subarea decreased, i.e., as its t_c and t_L values decreased, the more detailed approach made it more likely to pinpoint higher peak discharges. As will be shown later, however, this effect was largely absorbed by the small size of the small subareas in comparison with the larger ones and, as a result, the effect of considering a

Table 4. 1-h UH Parameters Using Alternative Snyder's Method Approach

Subarea	t_p (h)	t_b (h)	C_p (h)	h_p (1/h)	q_p (cfs/mi^2)	Q_p (cfs)
IV	3.24	14.58	0.60	0.185	118.5	2971
V	2.24	10.08	0.60	0.268	171.5	1188
VI	1.29	5.80	0.60	0.466	298.0	922
VII	1.58	7.11	0.60	0.380	242.9	785
VIII	1.07	5.35	0.53	0.560	358.6	615
IX	0.72	3.60	0.50	0.833	533.1	342
X	0.70	1.70	0.50	1.000	632.0	442
XI	1.05	2.60	0.50	0.916	586.6	1807
XIII-a	3.39	15.24	0.60	0.177	113.4	475
XIII-b	1.05	2.50	0.50	0.938	610.0	146
XIII-c	1.25	3.60	0.60	0.739	476.3	129
XIII-d	1.44	7.19	0.60	0.417	266.9	198

Note: 1 cfs/mi^2 = 1.094 (m^3/s)/ha; 1 cfs = 2.832x10^{-2} m^3/s

more detailed approach on the overall behavior of the entire watershed turned out to be marginal. Therefore, using a unique, and somewhat coarse, value of time interval did not have a significant influence on the final results.

FLOOD HYDROGRAPH

The unit hydrographs, as defined before, were applied to the effective rainfall hyetograph, by means of convolution, to obtain the hydrograph for Halls and Berry Brakes. The rainfall over this area (represented by subarea XII) was also added. As shown in Figure 4, all the subareas, with the possible exception of subareas XIII-a and XIII-d, contributed to water storage on the southwest side of I-49. Further, the drainage pattern, as depicted by the subareas chosen, means that all the hydrographs began at the same time and, therefore, a simple addition process was sufficient to obtain the total hydrograph that flowed into Halls and Berry Brakes. For simplicity, Halls Brake and Berry Brakes were assumed to constitute a single storage unit.

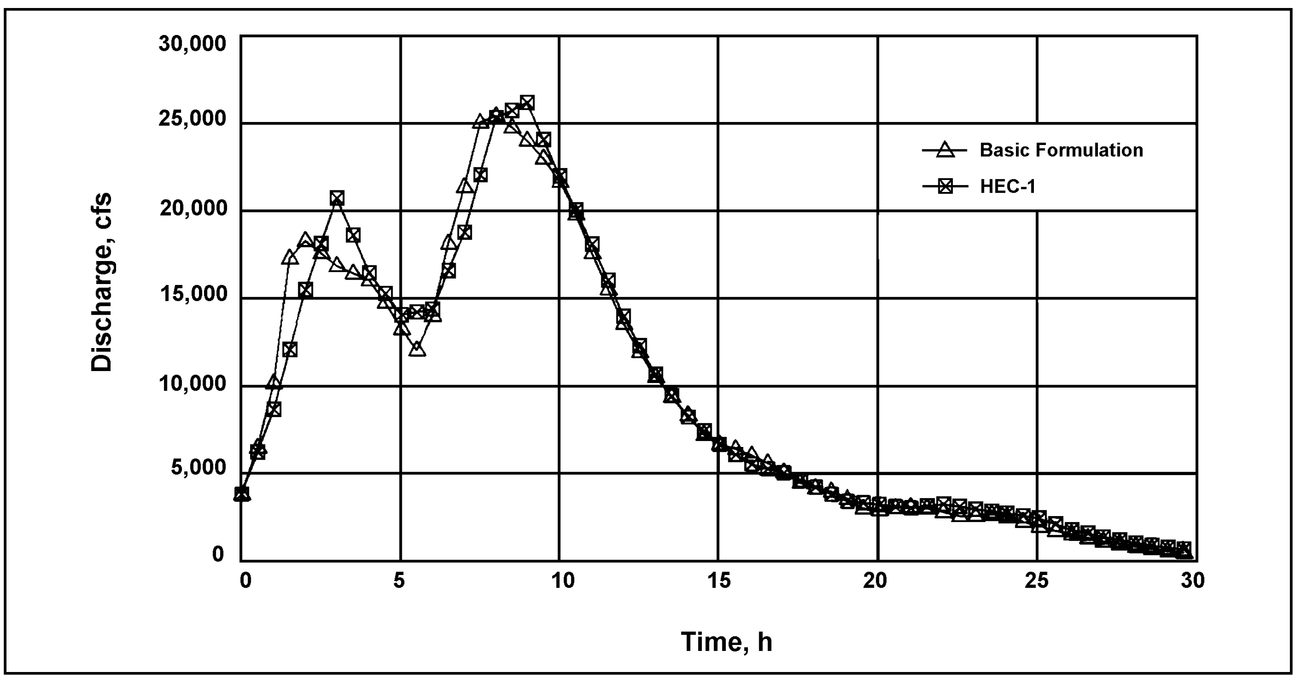

Figure 5. Hydrograph at Halls and Berry Brakes.

In the case of subareas XIII-a and XIII-d, it was decided to assume that they also contributed to the storage, even though they drain into Tapalcat Bayou. On the one hand, computations were made simpler. On the other hand, results would be on the safe side, i.e., water levels on the southwest side of I-49 would be slightly overestimated. The resulting hydrographs at Halls and Berry Brakes are shown in Figure 5. As seen, the HEC-1 model and the alternative formulation produced almost identical hydrographs, despite their differences regarding UH shape, peak discharges and unit rainfall duration.

HYDROGRAPH ROUTING

In order to estimate water levels on the southwest side of I-49, the hydrograph obtained before was routed through a pool-level reservoir. The HEC-1 model was used for this purpose. The general formulation of the routing scheme used can be written as

$$\frac{dS}{dt} = I - O \tag{8}$$

where S is storage, I is inflow, and O is outflow. To complete the model, a storage-outflow relationship is needed. This was accomplished by specifying the storage-stage curve and the outflow-stage curve, as described below.

Stage-Storage Relationship

This relationship was defined by measuring volumes between contour lines on the topographic map. Scenarios with and without I-49 were considered. For the with I-49 scenario, only the southwest part of I-49 was included, assuming that the storage on the other side was negligible. For the without I-49 scenario, the storage was assumed to be limited by the slightly elevated terrain located east of Halls and Berry Brakes. Table 5 shows the storage-stage data for both scenarios.

Table 5. Storage-Stage Relationship

	With I-49 Scenario			Without I-49 Scenario		
Stage (ft)	Area (acres)	Storage (acre-ft)	Cum. Storage (acre-ft)	Area (acres)	Storage (acre-ft)	Cum. Storage (acre-ft)
117	0		0	0		0
120	1610	1610	1610	1656	1656	1656
125	2433	10108	11718	3305	12403	14059
130	3256	14223	25941	4128	18583	32642

Note: 1 ft = 0.3048 m; 1 acre = 0.4047 ha; 1 acre-ft = 1.234×10^3 m^3

Stage-Outflow Relationship

I-49 runs parallel to Tapalcat Bayou for approximately 4500 ft (1372 m) between the end of the reservoir area and the LA 485 bridge over Tapalcat Bayou near Allen. As no watermark records on the bridge were available and, therefore, no information regarding how effectively the bridge controlled the flow upstream from it, the outflow-stage relationship for the reservoir involved the computation of flow profiles along Tapalcat Bayou for various discharge values. The HEC-2 model was used for this purpose.

Several runs varying the assumed initial flow conditions were made. Critical flow conditions on the bridge, as well as three subcritical initial energy slopes at the bridge (1.5×10^{-3} ft/ft, 5.0×10^{-4} ft/ft, and 2.5×10^{-4} ft/ft), were considered. This way, the possibility of having the bridge completely covered with water was also simulated. Two sets of runs were made to include both with and without I-49 scenarios.

1. <u>With I-49 Scenario</u>. In all cases, regardless of the initial flow condition assumed at the bridge, the differences in water surface elevations near the bridge section were the largest, and the corresponding differences near Berry Brake were the lowest. This clearly indicates that the flow along the upstream reaches of Tapalcat Bayou tends to be controlled by the slope of the channel rather than by the conditions at the bridge. As a result, the drainage of Halls and Berry Brakes depends mainly on the conveyance of the cross section of Tapalcat Bayou, and a stage-discharge relationship for the reservoir can be defined with confidence using the results for the most upstream cross section on Tapalcat Bayou. This stage-discharge relationship is shown in Figure 6. An additional observation is that the conditions at the bridge do not play a significant role in the drainage of Halls and Berry Brakes.

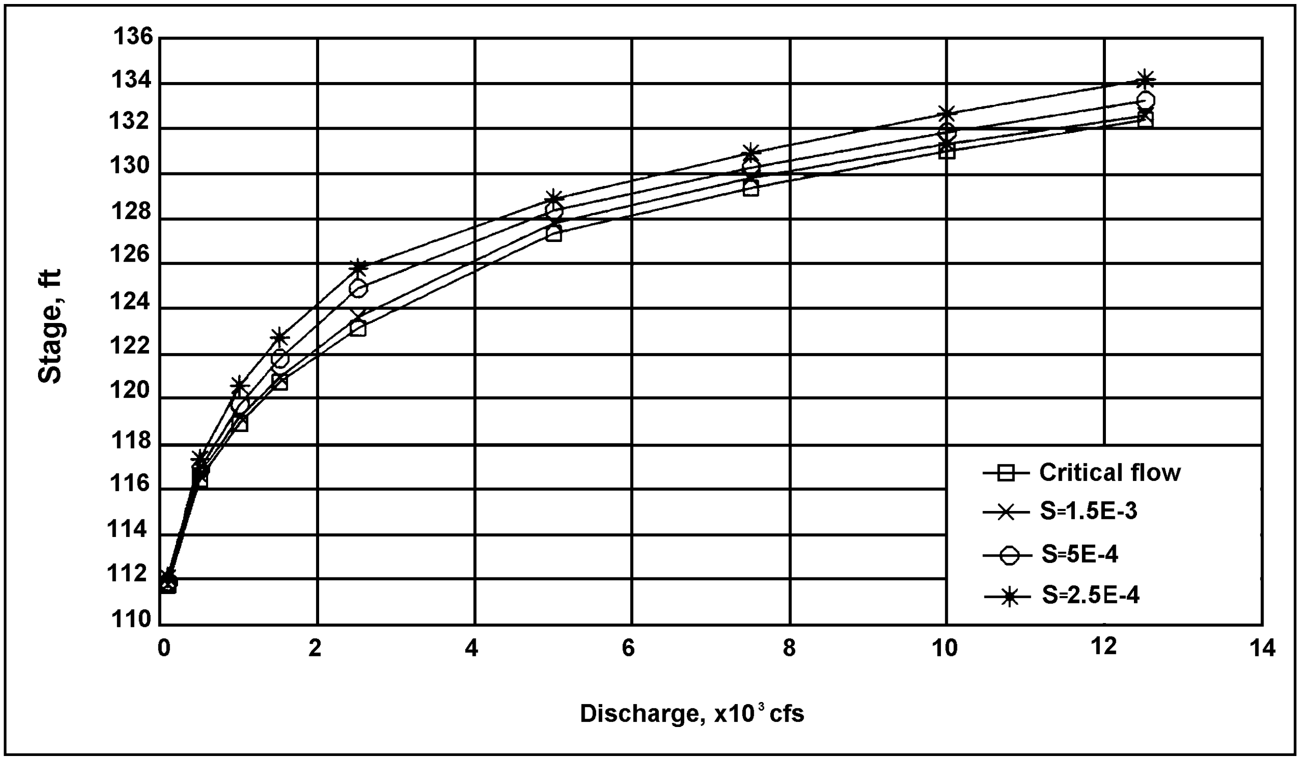

Figure 6. Stage-discharge relationship at Hall and Berry Brakes (with I-49 Scenario).

2. <u>Without I-49 Scenario</u>. For simulation, the left boundary for water surface profile computations was assumed to be located some 6000 ft to the left of Tapalcat Bayou. This distance is approximately the same as the axis of the slightly elevated terrain located on the east side of I-49. It must be taken into consideration, however, that under flooding conditions, and because the landscape is so flat, it might be possible that flow occurs from Pierre Bayou to Cow Bayou or vice versa, rendering any water divide definition useless. Since the actual water elevations on both creeks corresponding to the events of May 17-18, 1989 were not known, it was decided to assume that no diversion from one basin to the other occurred, and that the left boundary was fixed for all water levels. As before, the differences in water surface elevations near Berry Brake were the lowest, and a stage-discharge relationship for the reservoir was defined using the results from the most upstream cross section on Tapalcat Bayou. Such relationship is shown in Figure 7. Since the flow would occupy much more space at higher discharges, the variation in water levels would be much lower than when I-49 is included. For this reason, only two cases were considered.

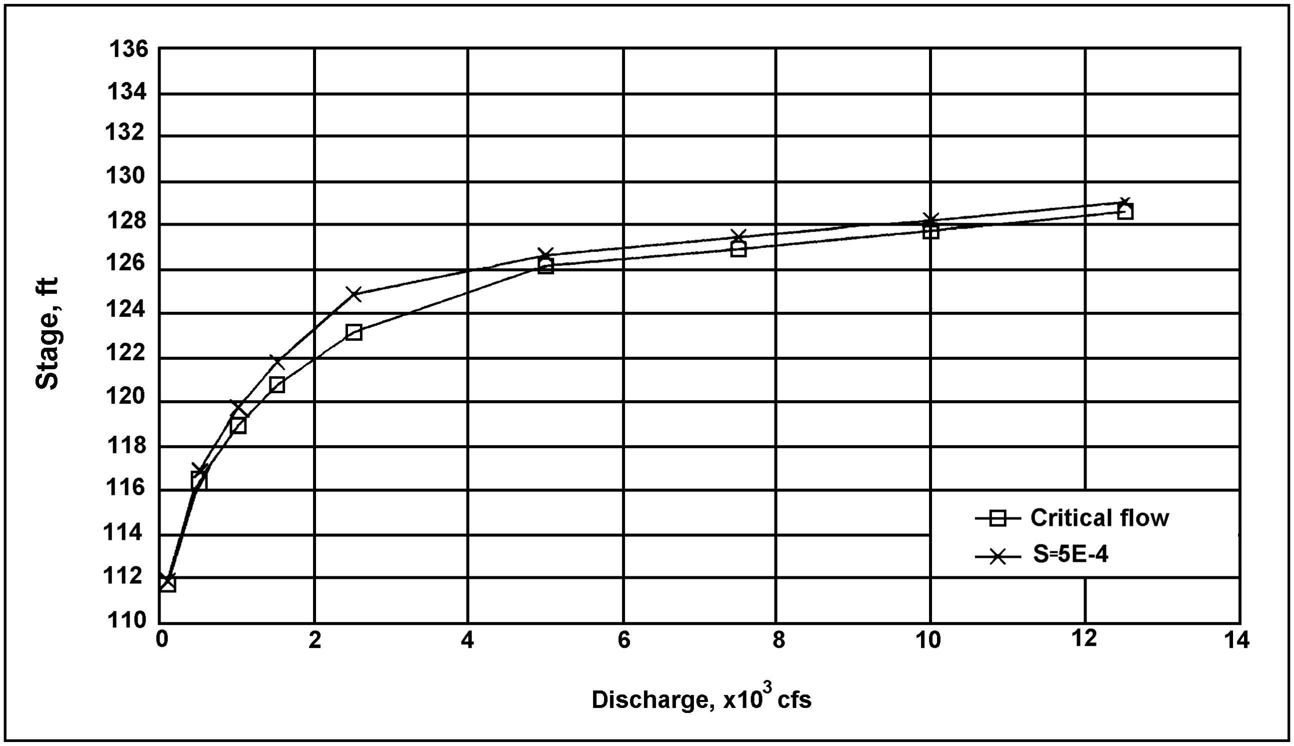

Figure 7. Stage-discharge relationship at Halls and Berry Brakes (without I-49 Scenario).

Flow Routing

The stage-storage data shown in Table 5, and the data corresponding to each of the stage-discharge curves shown in Figures 6 and 7, were used in the HEC-1 model to route the inflow hydrograph at Halls and Berry Brakes. Two sets of results were obtained: one assuming I-49 in place, and the other one assuming original landscape conditions.

1. <u>With 1-49 Scenario</u>. The inflow and outflow peak discharges, as well as the maximum water elevations, at Halls and Berry Brakes, are shown in Table 6. Figure 8 shows, as an example, the inflow

Table 6. Routing Results at Halls and Berry Brake

Alternative	Inflow Hydrograph		Outflow Hydrograph		Stage	
	t_p (h)	Q_p (cfs)	t_p (h)	Q_p (cfs)	t_p (h)	H_{max} (ft)
With I-49 Scenario						
Critical Flow	9	26177	18	4598	18	126.66
$S=1.5x10^{-3}$	9	26177	18	4383	18	126.76
$S=5.0x10^{-4}$	9	26177	18	3992	18	126.95
$S=2.5x10^{-4}$	9		19	3603	19	127.13
Without I-49 Scenario		26177				
Critical Flow	9	26177	16	4622	18	125.65
$S=5.0x10^{-4}$	9	26177	16	4006	18	125.90

Note: 1 cfs = $2.832x10^{-2}$ m^3/s; 1 ft = 0.3048 m

and outflow hydrographs, and the water level variation, for $S=1.5x10^{-3}$. Evident in the results is the high attenuating power of the reservoir at Halls and Berry Brakes. From more than 26,000 cfs, the peak discharge was reduced to about 4,000 cfs. Obviously, the actual peak discharge at the reservoir outlet may have varied somewhat depending on the conditions at the bridge on LA 485. The highest value obtained, 4,598 cfs, resulted from assuming critical flow conditions at the bridge. The lowest value obtained, 3,603 cfs, resulted from assuming an initial energy slope of $2.5x10^{4}$ (or 0.025%) at

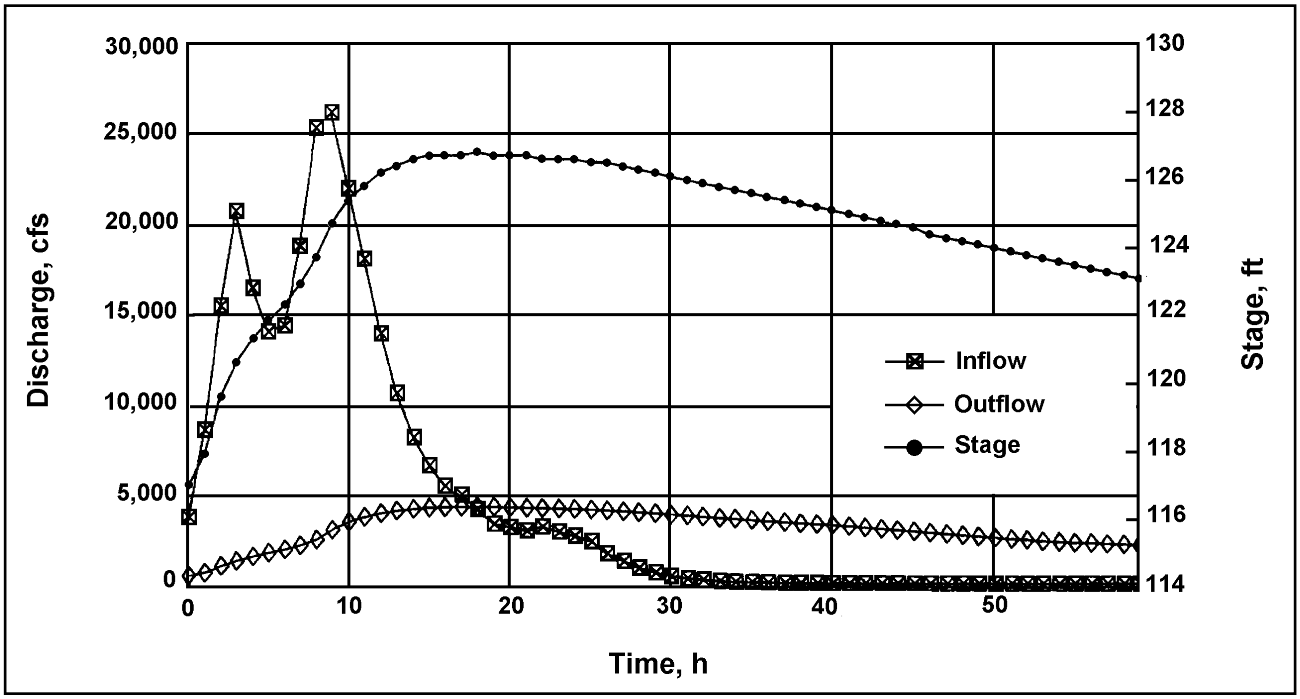

Figure 8. Routing results at Halls and Berry Brakes (with I-49 scenario).

the bridge. Because of the high attenuating power of the reservoir, maximum water stage values experienced low variation: from 126.66 ft for 4,598 cfs, to 127.13 ft for 3,603 cfs.

Cox (1990) reported that reverse flow occurred in some of the culverts that crossed I-49. An analysis of the timings of the hydrographs on both sides of the highway confirms this observation. If the outlets of subareas XIII-a, XIII-b, and XIII-c had been free, these subareas would have been drained completely before 10 hours. However, the water levels on the southwest side of I-49 kept increasing well beyond 10 hours and, as a result, reverse flow through the structures occurred. Since the areas on the northeast side of I-49 normally drain in a south-southwest fashion, reverse flow through the structures necessarily meant flooding on the northeast side of I-49.

Estimating the magnitude of such flooding is not an easy task because flow through the structures was most likely dependent on tailwater elevations which, as mentioned earlier, may have been influenced by water levels on Pierre Bayou, and were unknown. However, a preliminary analysis indicates that for a head water elevation of 126.76 ft and a tail water elevation of 125.76 ft (differential heads of the order of 1 ft were reported (Cox, 1990)), around 2200 cfs may have crossed I-49 to the east. According to Figure 8, head water levels of the order of 126.7 ft could have lasted around 10 hours and, for this time period, approximately 1800 acre-ft of water could have crossed I-49. This volume of water is of the same order of magnitude as the storage on the northeast side of I-49 at an elevation of 125.76 ft (3000 acre-ft according to the data shown in Table 5). Clearly, flooding on the northeast side could have taken place even with I-49 in place.

2. Without I-49 Scenario. The inflow and outflow peak discharges, as well as the maximum water levels on the reservoir, are shown in Table 6. Figure 9 shows, as an example, the inflow and outflow hydrographs, and the water level variation for $S=5x10^{-4}$. As before, the high attenuating power of the reservoir was evident. In this case, however, because of the increase in storage associated with the east side of I-49, maximum water levels in the reservoir were found to be around one foot lower than when I-49 was assumed to be in place. Outflow peak discharges turned out to be slightly larger.

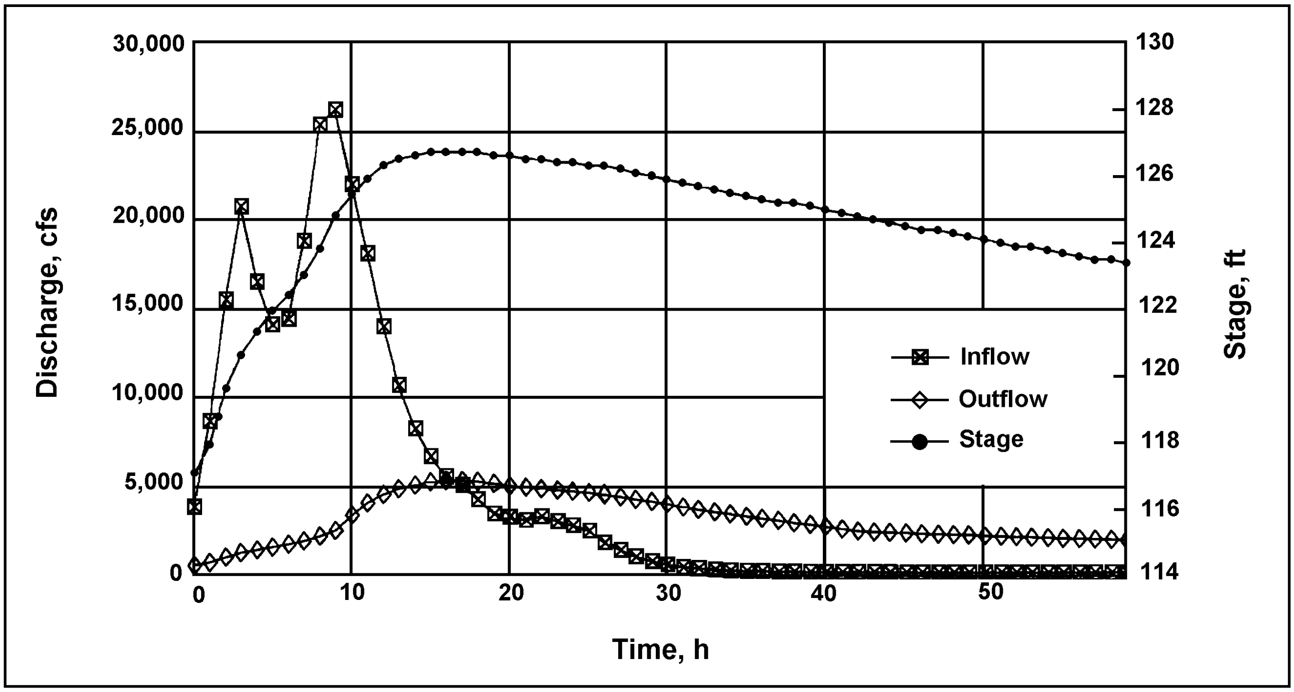

Figure 9. Routing results at Halls and Berry Brakes (without I-49 scenario).

DISCUSSION OF RESULTS

An analysis of the results obtained in the previous two sections indicates the following:

(a) The reservoir has a high attenuating power, as evidenced by the differences between the inflow and outflow hydrographs at Halls and Berry Brakes. This was true regardless of the existence of I-49 and regardless of the conditions assumed downstream on Tapalcat Bayou. The time to the peak was also significantly offset.

(b) The presence of I-49 did play a role in the extent and distribution of flooding, but the role turned out to be marginal. With I-49 in place, maximum water levels in the reservoir could have been close to 127 ft above sea level. According to Table 5, the area flooded on the southwest part of I-49 would have been around 2,750 acres. Most of the flood was contained in the southwest side of the highway because of the embankment. However, the existence of culverts allowed water to cross the highway, helping to extend the flood to the northeast side.

If I-49 had not been built, the maximum water levels in the reservoir could have been around 126 ft above sea level, i.e., one foot lower than with I-49 in place. According to Table 5, the area flooded could have been around 3,450 acres, 2,580 acres of which could have been located southwest of I-49. Clearly, then, I-49 could have been responsible for the flooding of 150 acres located on the southwest side of the embankment. Except for a small portion close to the highway, which was devoted to agricultural activities, most of this area was covered with woods, which means that the actual impact of the additional flooding was much less severe.

On the east side of I-49, the opposite probably occurred. Without I-49, no artificial embankment would have been present to contain the water, but a natural barrier given by the slightly elevated terrain located to the left of Tapalcat Bayou would have replaced the effect. Around 870 acres could have been flooded there. With I-49 in place, more water would have been stored southwest of the embankment but, at the same time, less water would have crossed the highway, which means that less than 870 acres of the flooded area northeast of I-49 could have been attributed to the construction of the highway. It must be added here that most of the area on the northeast side of the highway is agricultural, which means that any reduction of flooded area attributed to the highway plays an important role in measuring the impact of the flood.

An interesting observation at this point is that the accuracy of the HEC models to predict water levels is within 1 foot, and that this accuracy is normally considered sufficient for practical purposes. Being standard modeling tools as they are, required in many legal instances in the United States, the obvious conclusion from a practical point of view is that the one foot difference between the two scenarios considered is barely sufficient to make I-49 responsible for the flooding levels on the southwest side of the highway.

SENSITIVITY ANALYSIS

An analysis of the sensitivity of the results, particularly water levels, to the parameters used in the computations was performed. Two aspects were considered: (1) the effect of the Curve Number CN; and (2) the effect of selecting an initial water level on the reservoir. This last aspect was deemed important because significant rain had fallen on the area on the days previous to the major storm, and there was no indication as to what the water level was at the beginning of the main event.

Effect of CN

The CN selected for computing the ER with the SCS-CN method was 86. Two additional runs of the HEC-1 model assuming CN=70 and CN=95 were made. Only after I-49 conditions were considered. CN=70 would represent the extreme condition of assuming that the soil was not wet, and that the soil type was not C but possibly B. CN=95 would represent the equally extreme condition of assuming that the entire area was open, with very low vegetation.

Table 7 shows the corresponding peak inflow and outflow discharges, and the maximum water levels on the reservoir. When CN decreased to 70 (a variation of 18.6%), the inflow peak discharge decreased to 21,533 cfs (or 17.7 %), the outflow peak discharge decreased to 3,487 cfs (or 20.4 %), maximum stage decreased by 1.51 ft (or 15.5 %, with respect to the original assumed maximum water depth, i.e., 126.76 - 117 = 9.76 ft). Conversely, an increment of CN to 95 (or an increase of 10.5%) caused an increase to 27,896 cfs (or 6.6 %) and 4,871 cfs (or 11.1 %) for the peak discharges, and 127.57 ft (or 8.3 %) in the maximum stage.

Clearly, the results depend on the assumed value of CN. However, the two values used for the

Table 7. Effect of Changing CN on Peak Discharges and Stage

	Inflow Hydrograph		Outflow Hydrograph		Stage	
Alternative	t_p (h)	Q_p (cfs)	t_p (h)	Q_p (cfs)	t_p (h)	H_{max} (ft)
CN = 86	9	26177	18	4383	18	126.76
CN = 70	9	21533	18	3487	18	125.25
Variation:	0	-4644	0	-896	0	-1.51
CN = 95	9	27896	17	4871	17	127.57
Variation:	0	+1719	-1	+488	-1	+0.81

comparison, 70 and 95, are very extreme values. Most likely, the CN value could have been located between 80 and 90, and for this range, the expected variability of the results would not exceed ±6%. In particular, the estimate of the maximum water level is not expected to vary more than ±0.6 ft.

Effect of the Initial Water Level

All of the results obtained so far assume that the initial stage on the reservoir was 117 ft. This assumption was considered reasonable given the amount of rain that had fallen in the days previous to the major storm and given the geometry of Tapalcat Bayou. To test this assumption, two additional runs of the HEC-1 model were made: one with an initial stage of 115 ft, and the other with 120 ft. The stage of 115 ft would mean that the reservoir had essentially drained before the storm occurred. The stage of 120 ft would mean that the system had not drained completely and that flow on Tapalcat Bayou was on the verge of flooding. While this last hypothesis might be considered extreme, it was decided to simulate it anyway.

Table 8 shows the corresponding peak discharges and stage. As noted, the effect of changing the initial water level in the reservoir is practically negligible. In particular, the maximum water level is not expected to vary by more than 0.4 ft (or 4.1 %).

Table 8. Effect of the Initial Stage on Peak Discharge and Stage

Initial Stage (ft)	Inflow Hydrograph		Outflow Hydrograph		Stage	
	t_p (h)	Q_p (cfs)	t_p (h)	Q_p (cfs)	t_p (h)	H_{max} (ft)
117.0	9	26177	18	4383	18	126.76
115.0	9	26177	18	4385	18	126.76
Variation:	0	0	0	+2	0	0
120.0	9	26177	18	4626	18	127.16
Variation:	0	0	0	+243	0	+0.40

CONCLUSIONS

The following conclusions are drawn from this study:

(1) Between May 17 and 18, 1989, severe rainfall occurred in northwest Louisiana, causing flooding in several places. In the vicinity of Allen, nearly 10 inches of rain fell. For a total duration of 24 h, the corresponding return period of the storm was estimated to be between 50 and 100 years. The analysis showed that 9 inches may have fallen in the first 12 hours, and for this condition, the associated return period was found to be 100 years. Clearly, the event was a very extreme one.

(2) Two methodologies, based on the versions of the Snyder unit hydrograph, were used to estimate the hydrographs associated with all the subareas. Results showed similar watershed response in both cases, and, as a result, only one of the methodologies (the one based on Snyder's unit hydrograph version implemented in the HEC-l model) was used for the routing process afterwards.

(3) Results from the routing at Halls and Berry Brakes show that the maximum water level on the southwest side of I-49 could have been close to 127 ft, causing considerable flooding on that area. This high stage could have also caused flooding on the northeast side of the highway as reverse flow

was generated through the culverts that cross I-49. Maximum water levels on the northeast side of the highway could have been one foot lower than those on the southwest side.

(4) Results also show, however, that flooding could have occurred if the highway embankment had not been built. The maximum water level at Halls and Berry Brakes would have been a little lower, of the order of one foot lower than with I-49 in place, but high enough to cause flooding. However, since no artificial embankment would have been present to contain the water, the extent of the flooding would have been larger.

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