IV. HYDRAULICS

Clifford D. Muir and Jerome A. Westphal

While this section relates directly to contract items specified under hydraulics, the directly related subject of 1) river confinement by levees, 2) comparison of dike construction history with associated changes in top bank width, and 3) comparison of dike construction history with changes in stage-discharge relationship, are included in discussions within this section of the report.

The limited number of overbank velocity measurements obtainable within the time frame of the project proved to be a major constraint on efforts to develop velocity relationships on overbank areas. Most of the available data were flow rates in the overbank area. However, complete flood-stage measurements of velocities and associated depths for the St. Louis and Chester gage locations were obtained for years after 1935. Also, velocities in the overbank area were obtained for the Vicksburg area for the years 1929, 1933, 1935, and 1973.

Estimated velocity distributions at the St. Louis gage are shown on Figures 4.1a and 4.1b for July 1, 1942 and May 1, 1973, respectively. Isovels were contoured on the basis of point velocities taken for discharge measurements. Because of the flood wall at St. Louis, there is relatively little overbank area. There were apparently no striking changes in cross-section shape or velocity distributions at the section between 1942 and 1973.

Estimated velocity distributions at the Chester gage were made in the same manner as at the St. Louis gage and are shown on Figures 4.2a and 4.2b for July 17, 1951 and May 14, 1973, respectively. Although

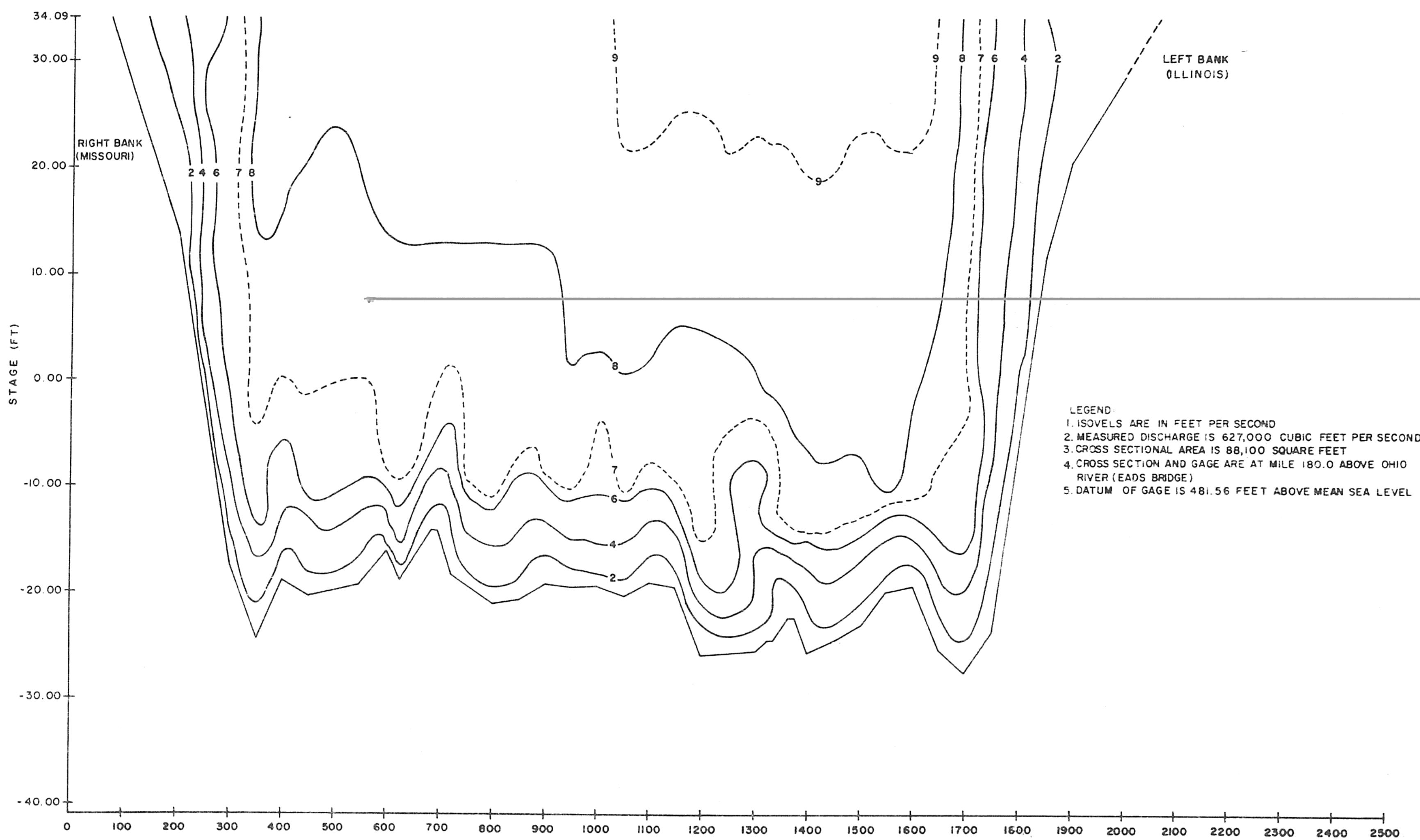

Figure 4.1a Estimated Velocity Distributions in the Mississippi River for July 1, 1942 near St. Louis, Missouri

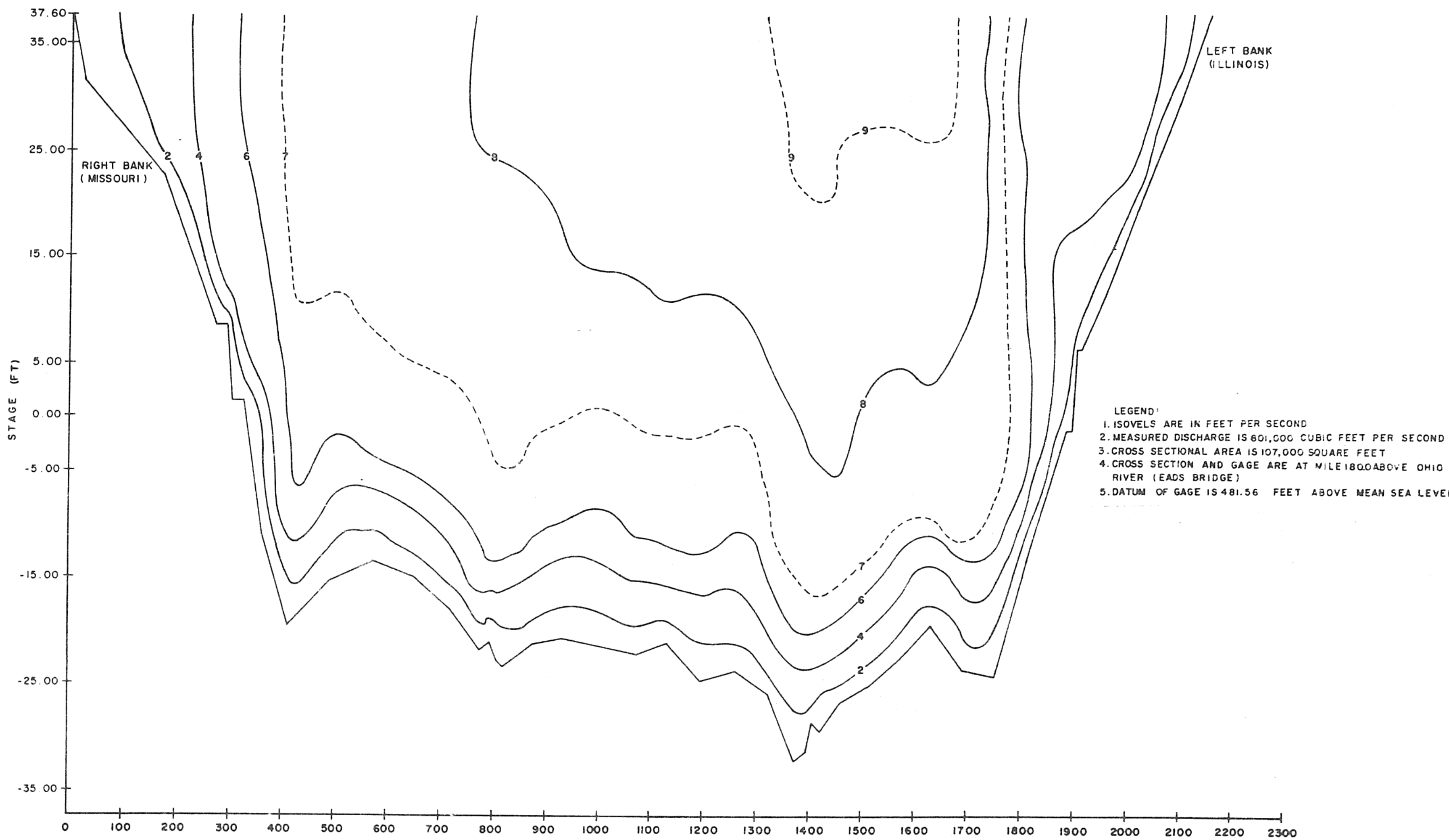

Figure 4.1b Estimated Velocity Distributions in the Mississippi River for May 1, 1973 near St. Louis, Missouri

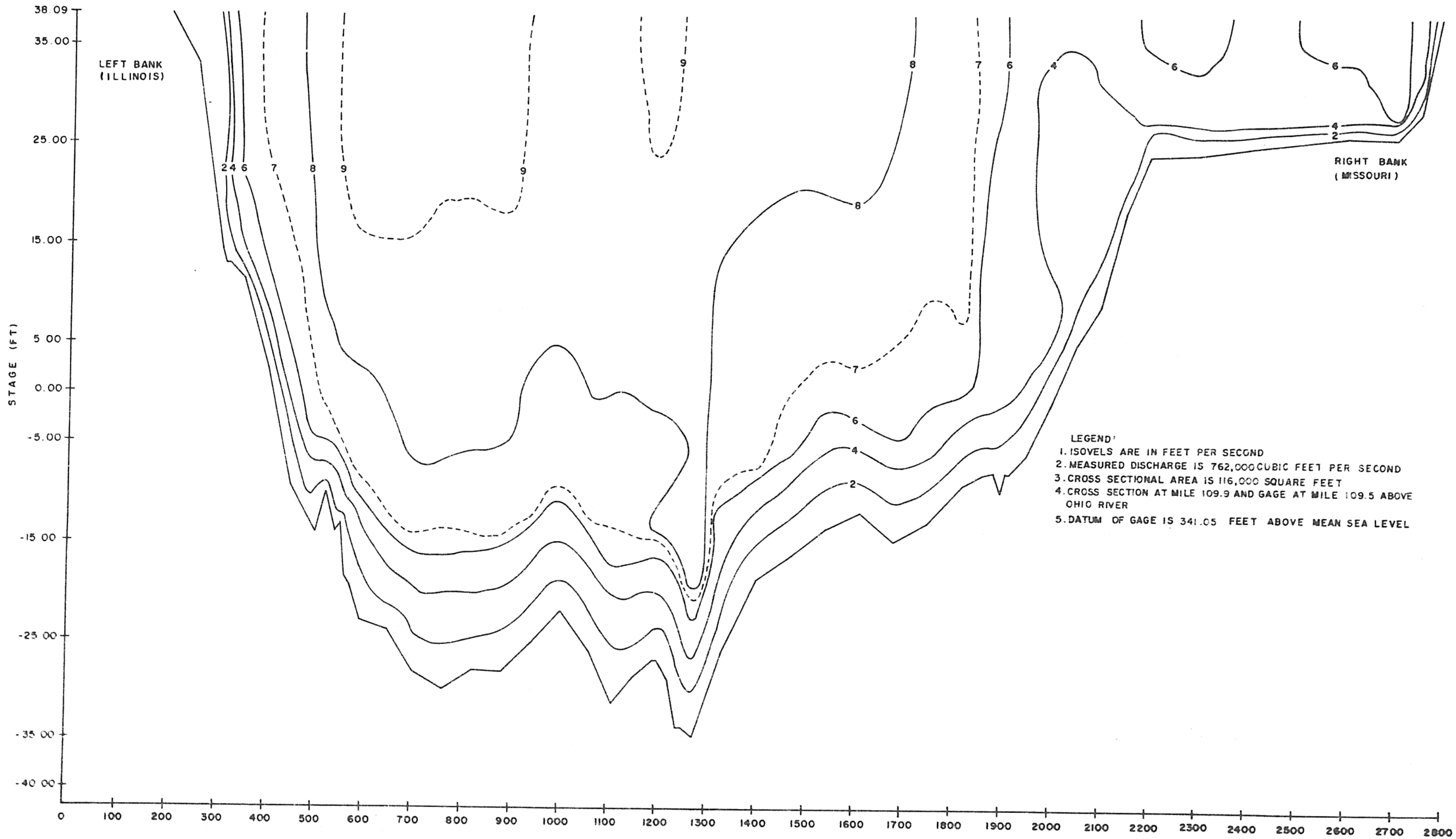

Figure 4.2 a Estimated Velocity Distributions in the Mississippi River for July 17, 1951 near Chester, Illinois

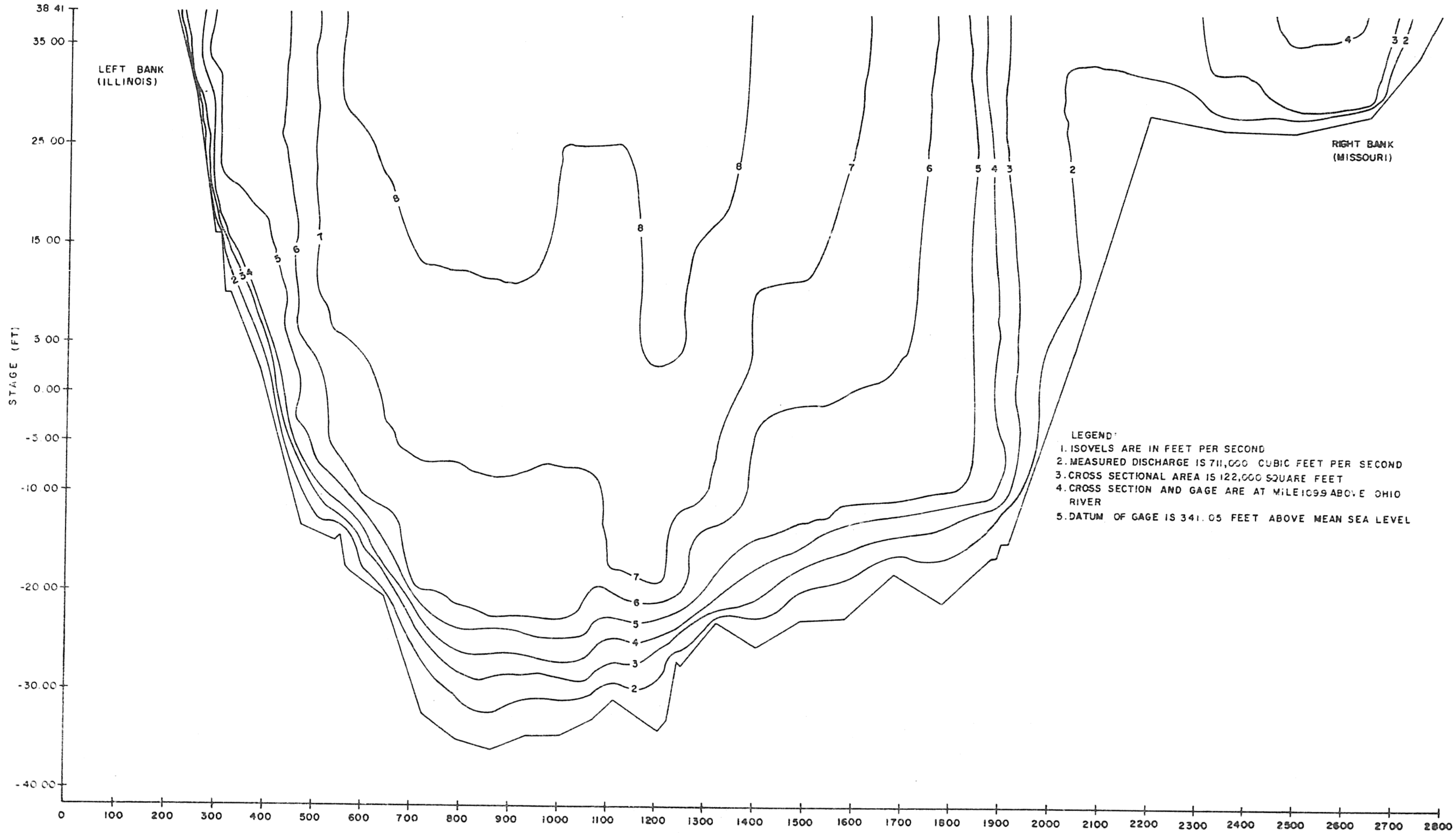

Figure 4.2b Estimated Velocity Distributions in the Mississippi River for May 14, 1973 near Chester, Illinois

the thalweg has apparently migrated toward the left bank (Illinois side) in the period between 1951 and 1973, cross-sectional area and shape are nearly the same for the two dates. The similarity of isovel patterns is probably a reflection of the similarity of cross-section geometry.

Typical overbank velocities for several locations at Vicksburg are included in Tables 4.1, 4.2, and 4.3. Attempts to correlate velocity on the overbank with depth of flow were unproductive. For the range of observations made thus far, there is no discernible functional relationship between depth of flow on the overbank and overbank velocities. Before difinitive statements can be made in this regard, detailed information regarding energy dissipaters are needed. For example, in the St. Louis District where brush had been cleared in front of the levees, a secondary channel developed during flood stages. To overcome overbank velocities and secondary channels, the St. Louis District has constructed spur levees and abatis dikes and has encouraged willow growth in old borrow pits. Therefore, overbank velocities at a given site probably reflect local conditions more than the generalized influence of levees or confinement.

Maximum velocities on the overbank are less than main channel maximum velocities. The 1933 overbank maximum velocity of 2.55 ft/sec at Vicksburg corresponded to a main channel maximum velocity of 7.98 ft/sec. The Chester overbank maximum for May 14, 1973, appeared to be about one-half the main-channel maximum.

A comparison of flow rates in the overbank area and river stage in the vicinity of Vicksburg is presented in Figure 4.3. The 1973 values were for a cross section 0.4 miles downstream from the Vicksburg Bridge. Zero gage at this section was 46.25 feet above mean sea level. The 1929

TABLE 4.1

OVERBANK FLOW

LOWER DELTA POINT, LA. JUNE 7, 1929

STAGE 55.1 FT.

Distance from River Toward Levee (ft)	Depth (ft)	Mean Velocity (fps)
0	6	0.60
500	8	0.37
1000	14	0.47
1500	15	0.43
2000	17	0.13
2500	18	0.13
3000	17	0.43
3500	18	0.47
4000	19	0.64
4500	20	0.80
4600	16	0.53
4700	12	1.01
4800	11	1.07
4900	11	1.21
5000	12	1.34
5100	11	1.53
5200	11	1.64
5300	9	1.81
5400	7	1.14
5500	5	1.58
5700	0	0.00

TABLE 4.2

OVERBANK FLOW
LOWER DELTA POINT, LA.

Date	Stage (ft)	Distance from River Toward Levee (ft)	Depth (ft)	Mean Velocity (fps)
June 5, 1929	55.10	0	4	0.68
		500	6	1.22
		1000	12	0.32
		1500	17	0.58
		2000	19	0.78
		2500	17	0.35
		3000	17	0.65
		3500	19	0.68
		4000	19	0.58
		4500	18	0.55
April 22, 1933	48.03	100	6.5	0.77
		160	5.5	1.50
		250	6.2	2.05
		275	4.7	2.35
		390	5.5	1.88
		450	5.0	2.22
		530	5.3	2.09
		595	5.5	2.40
		620	5.5	2.46
		660	5.5	2.24
		705	5.5	2.55
		750	0.0	0.00

TABLE 4.3

OVERBANK FLOW

VICKSBURG-HIGHWAY BRIDGE; MAY 14, 1973

Distance from River Toward Levee (ft)	Depth (ft)	Mean Velocity (fps)
0	6.0	.216
200	5.4	.320
400	5.6	.560
600	4.9	.720
800	6.2	.510
1000	11.6	.720
1200	12.8	.710
1400	13.2	.560
1600	11.4	.590
1800	12.1	.710
2000	12.2	.640
2200	13.4	.780
2400	15.2	.500
2600	15.5	.590
2800	16.8	.500
3000	16.7	.910
3200	16.0	.820
3400	16.0	1.100
3600	16.7	.930
3800	17.8	.558
4000	18.7	.940
4200	12.1	.880
4400	8.3	1.290
4600	10.1	1.26
4800	8.6	.864
4900	7.6	1.170
5000	4.1	1.188
5050	Close to 0	

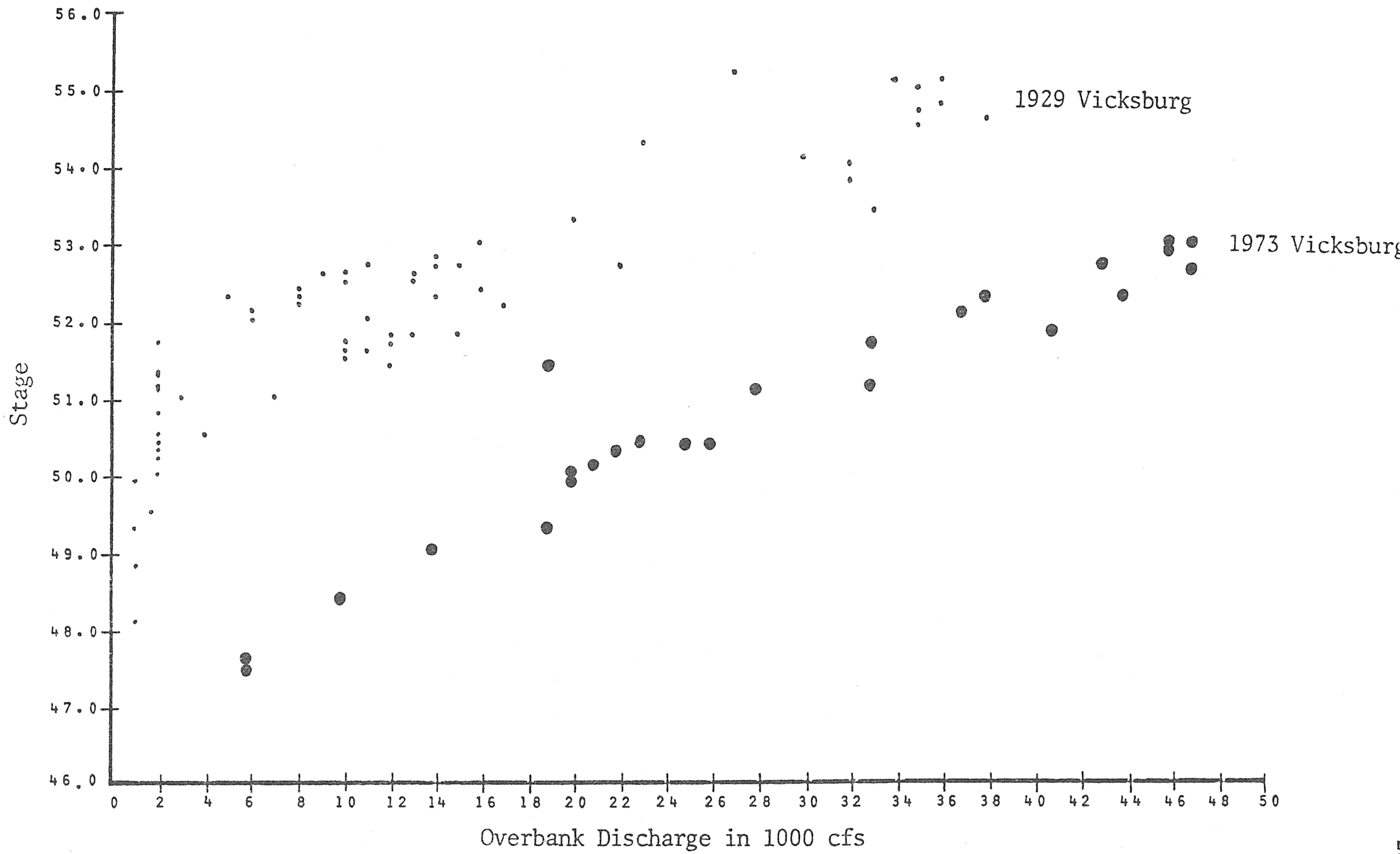

Figure 4.3. River Stage Versus Overbank Discharge Near Vicksburg for 1929 and 1973

values were taken at Lower Delta Point (precise location unknown). Zero gage at Lower Delta Point was 46.16 feet above mean Gulf level. The 1929 data were much more variable than the 1973 data, possibly because of variation of flow-measurement techniques in 1929. However, stage-discharge relationships are reasonably well defined for both periods. Therefore, it appears that on the overbank, localized influences such as vegetation and channelization are sufficient to mask depth-velocity relationships but do not affect the relationship between average flow rate and river stage.

River confinement by levees is both detrimental and beneficial. It prevents the enrichment of agricultural lands which accompanies the deposition of waterborne fine material on protected areas. However, the rather large benefits derived from flood protection cannot be disregarded. According to personal communication from Mr. Michael Dace, III, of the St. Louis District, some farmers whose lands are unprotected by federal levees would accept spur levees over the more expensive closed levees. The purpose would be to allow controlled flooding with the accompanying benefit of deposition of fine materials.

Deposition of coarse materials on agricultural land is detrimental. In the absence of data necessary to evaluate the main channel as a source of coarse sediments which are deposited on overbank areas during floods, a theoretical approach must be used. The problems are 1) to determine the amount of coarse sediments in suspension above banks and 2) to determine if these sediments would be deposited on overbank areas.

An equation commonly used to calculate particle concentrations is:

$$\frac{C}{C_a} = \left\{\frac{(d-y)}{y} \frac{a}{(d-a)}\right\}^z \qquad (4.1)$$

where C = the concentration of particles of a given size at a distance y above the bed,

d = mean depth of flow,

y = height of point above river bed,

a = some reference point above the river bed,

C_a = the concentration of particles of a given size at reference point a, and

w = settling velocity of the particles.

The exponent $z = w/ku_*$ (4.2)

where u_* is the shear velocity expressed as $u_* = \sqrt{gRS}$, and (4.3)

S = slope of the hydraulic grade line,

R = hydraulic radius (R = d for rivers),

g = acceleration due to gravity, and

k = von Karman's universal constant.

Toffaleti (1963) developed the following simplified relationships for the lower Mississippi River:

$$C = b(d/y)^z \qquad (4.4)$$

where b is a constant equal to the particle concentration when d/y equals unity, and

$$z = \frac{\overline{U}w}{252Sd} . \qquad (4.5)$$

Toffaleti (1968) states that the distribution of particles less than 0.062 millimeters was fairly uniform at Simmesport on the Atchafalaya River. He listed typical sediment size ranges and classifications as:

Description	Size Range (mm)	Geometric Mean Diameter (mm)	Geometric Mean Diameter (ft)
Silts and Clays (S&C)	0.062		
Very Fine Sand (VFS)	0.062-0.125	0.0880	0.00029
Fine Sand (FS)	0.125-0.250	0.177	0.00058
Medium Sand (MS)	0.250-0.500	0.354	0.00116
Coarse Sand (CS)	0.500-1.000	0.707	0.00232

Using a typical section at Talbert's Landing on the Mississippi River it was assumed that bed-load gradation would be similar to that at Simmesport shown above. The following table shows the fraction of bed material and settling velocities for particle sizes in the assumed gradation:

Geometric Mean Diameter (ft)	Fraction of Bed Mtl.	Settling Velocity (ft/sec) 70°F	Settling Velocity (ft/sec) 80°F
0.00029	0.071	0.023	0.025
0.00058	0.283	0.069	0.075
0.00116	0.564	0.171	0.183
0.00232	0.078	0.356	0.373

Other assumptions made for calculations were as follows:

	Case I	Case II
Q (main channel)	1,395,800 cfs	1,008,200 cfs
River width	3,830 ft	3,730 ft
Temperature	70°F	80°F
Slope	0.0000382	0.0000382
Area of flow	188,400 ft^2	173,900 ft^2
Ht. of water surface above elevation at which overbank flow begins	10 ft	3.5 ft

The assumed flows and widths correspond to data given for May 5, 1973 and June 12, 1973, respectively. The coefficient b from equation 4.4 was assumed to be the same as that determined for the commensurate particle size by Toffaleti (1968).

The above assumptions were used in equations 4.4 and 4.5 to make determinations of sediment transport. The resulting determinations are given in tabular form. Quantities are given in both tons per day per square foot ($t/d/ft^2$) and parts per million (ppm).

Sediment Transported

Height (ft) Above River Bed	Case I: Geometric Mean Diameter (ft)				Case II: Geometric Mean Diameter (ft)			
	0.00029		0.00058		0.00029		0.00058	
	$t/d/ft^2$	ppm	$t/d/ft^2$	ppm	$t/d/ft^2$	ppm	$t/d/ft^2$	ppm
1	7.79	389	7.29	364	4.24	721	4.34	227
10	4.95	274	0.87	43	2.84	181	0.62	30.0
30	3.40	170	0.19	9.5	2.00	128	0.15	9.6
39.2	3.00	150	0.12	6.0				
43.1					1.65	105	0.09	5.8
46.2					1.62	103	0.08	5.10
49.20	2.69	134	0.08	3.99				

To examine the possibility of material being lifted or maintained in suspension, Toffaleti's (1968) version of Einstein's weight-to-lift ratio was used. This equation is

$$\frac{W}{L} = \frac{T}{U^2} 10^4 D \tag{4.6}$$

where W = weight of sediment particle underwater,

L = lift force on bed particle,

T = temperature dependent variable (0.063 for temperature = 70°F), and

D = particle size diameter.

Assuming a maximum weight-to-lift ratio (W/L) of unity and T equal to 0.063, the minimum velocities required to move sediments on the overbank at a water temperature equal to 70°F may be calculated from equation 4.6. For geometric mean diameters of 0.00029 ft. and 0.00058 ft. the minimum velocities are 0.43 ft/sec and 0.60 ft/sec, respectively.

Overbank velocities given for the Vicksburg area in tables 4.1, 4.2, and 4.3, indicate average velocities which are greater than those needed to transport fine sand. Such overbank velocities will not only transport the suspended sediment reaching the overbank but will cause local scour. Deposition will occur only where velocities are less than about 0.43 ft/sec and will be a combination of material supplied by mainstream flow and scour from the overbank. Therefore, deposition of sediment on the overbank will generally be determined by the local configuration (either manmade or natural) of a particular area.

The preceding determinations are very sensitive to assumptions and tend to overestimate the amount of sediment in transport. They indicate the presence of fine sand above bank in the main channel. However, the concentration of fine sand and larger materials at or above the river bank elevation is relatively small. Because it appears from theoretical considerations that concentrations of fine sands are very small near the surface in the main channel, and because velocities on the overbank appear to be greater than the minimum required to keep fine sands in suspension, it is unlikely that the main channel is a significant source of sands deposited on the overbank during floods.

To evaluate effects of confinement and nonconfinement on sediment deposition in overbank areas, 13 cross sections were selected for comparisons of 1974 conditions with those existing during the decade 1879 to 1889. Cross sections were taken at intervals of approximately 10 miles beginning at mile 44.2 and ending at mile 169.8 above Cairo. At most of the cross sections, surveys made during the 1800's did not extend the full width of the flood plain.

At any given cross section, deposition and scour depend on relatively localized factors such as alignment of the river, channel stabilization measures instituted, and development associated with the various forms of land use on the flood plain.

The majority of channel configuration changes are attributable to channel stabilization structures, except at Kaskaskia Island where the Mississippi River captured a part of the Kaskaskia River and formed a cutoff which shortened the Mississippi River by about 11 miles. The old channel subsequently filled with sediment. The majority of changes on the flood plain are attributable to activities of riparian owners and installation of drainage facilities which were accomplished under authority of the various drainage districts.

The specific effect of levees is indistinguishable from effects of other factors which influence scour and deposition on the overbank. Although the topographic data were insufficient for detailed analysis of change, it appears that in spite of considerable flood-plain development, there has been relatively little change in the valley cross sections during the last 100 years.

Water-surface profiles for annual high and low stages in the Middle Mississippi River for the period, 1967 through 1974, are shown in Figures 4.4 through 4.11. The method used for Figures 4.4 through 4.7 was to select annual maximum and minimum stages at St. Louis gage, then plot corresponding stages at all other stations for that date. For Figures 4.8 through 4.11, annual maximum and minimum stages at each station were plotted irrespective of date of occurrence. Average slope of the annual water-surface profiles shown in Figures 4.4 through 4.7 and the average slope of these for the entire period from 1967 through 1974 is presented in Table 4.4. Also shown in Table 4.4 are slopes of annual maximum and minimum water-surface profiles and the average slope for the period, 1967 through 1974, for selected reaches of the Upper Mississippi, Missouri, and Illinois Rivers.

Although there is considerable variability in slope of the water-surface profile from year to year in each reach, there is no discernible time trend. However, as shown in Figure 4.4, average slope of the high-water profile is steeper than the low-water profile for the Middle

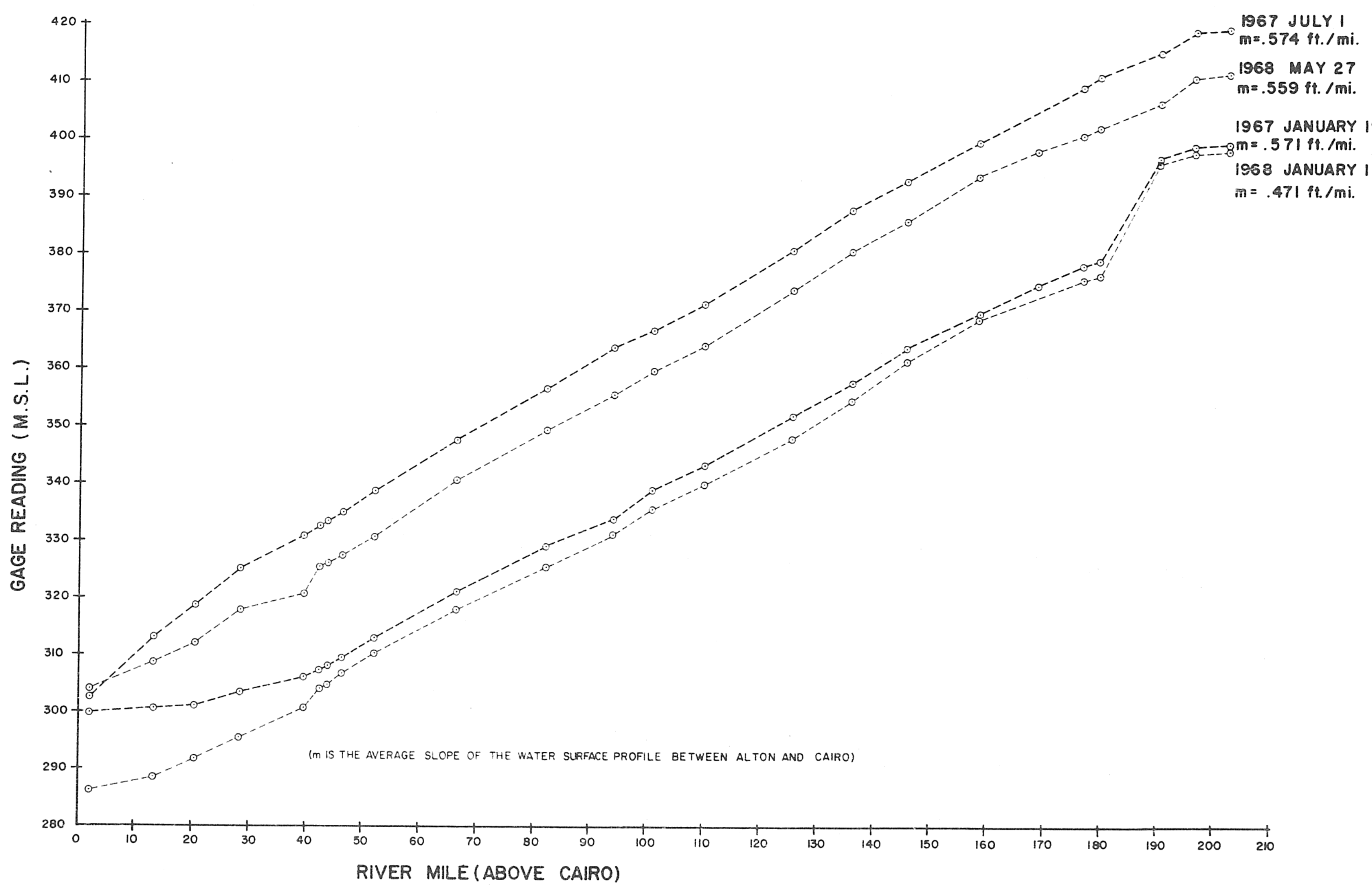

Figure 4.4 Surface Profiles of the Mississippi River Between Alton and Cairo for Maximum and Minimum Stages at St. Louis for 1967 and 1968

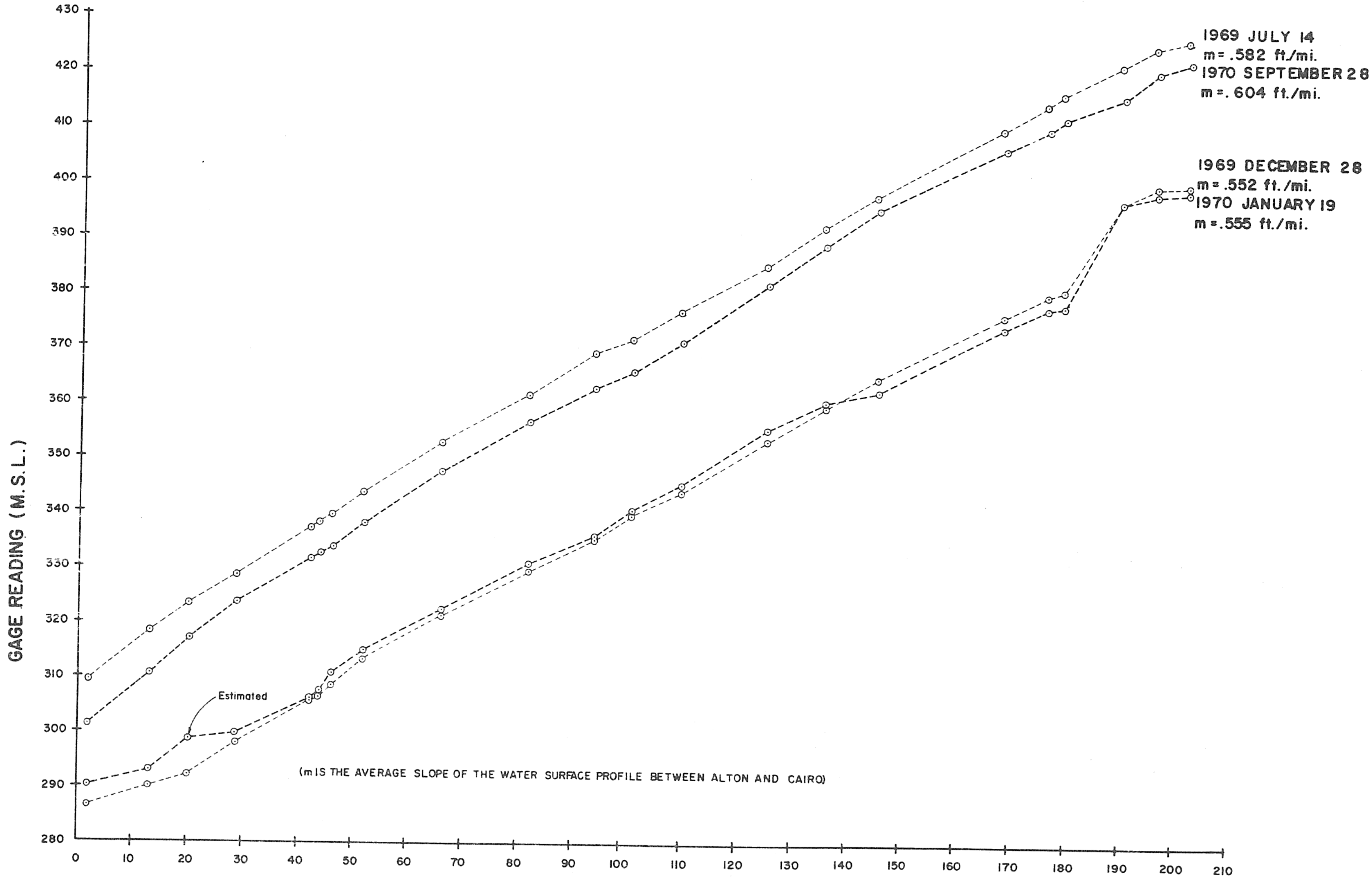

Figure 4.5 Surface Profiles of the Mississippi River Between Alton and Cairo for Maximum and Minimum Stages at St. Louis for 1969 and 1970

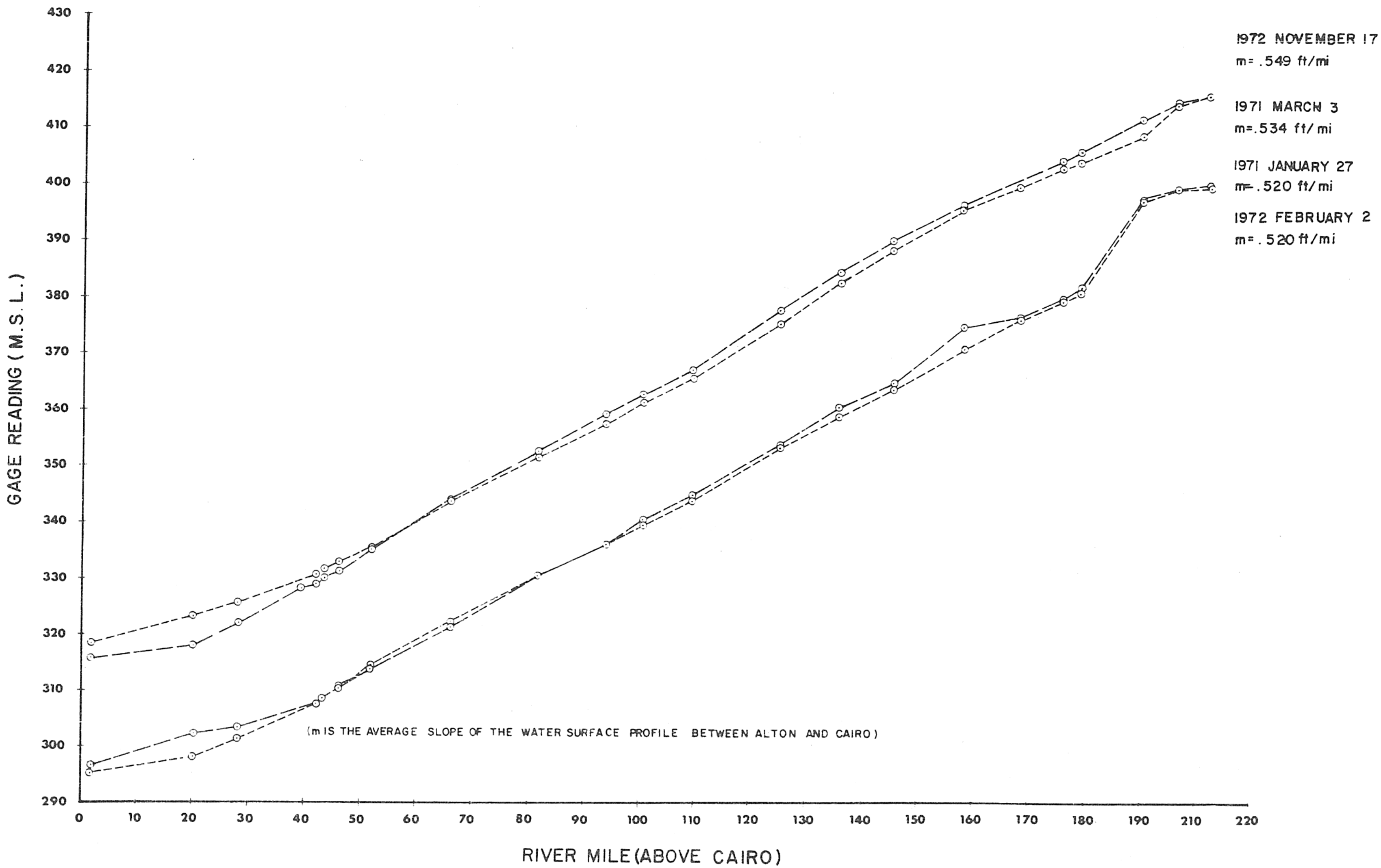

Figure 4.6 Surface Profiles of the Mississippi River Between Alton and Cairo for Maximum and Minimum Stages at St. Louis for 1971 and 1972

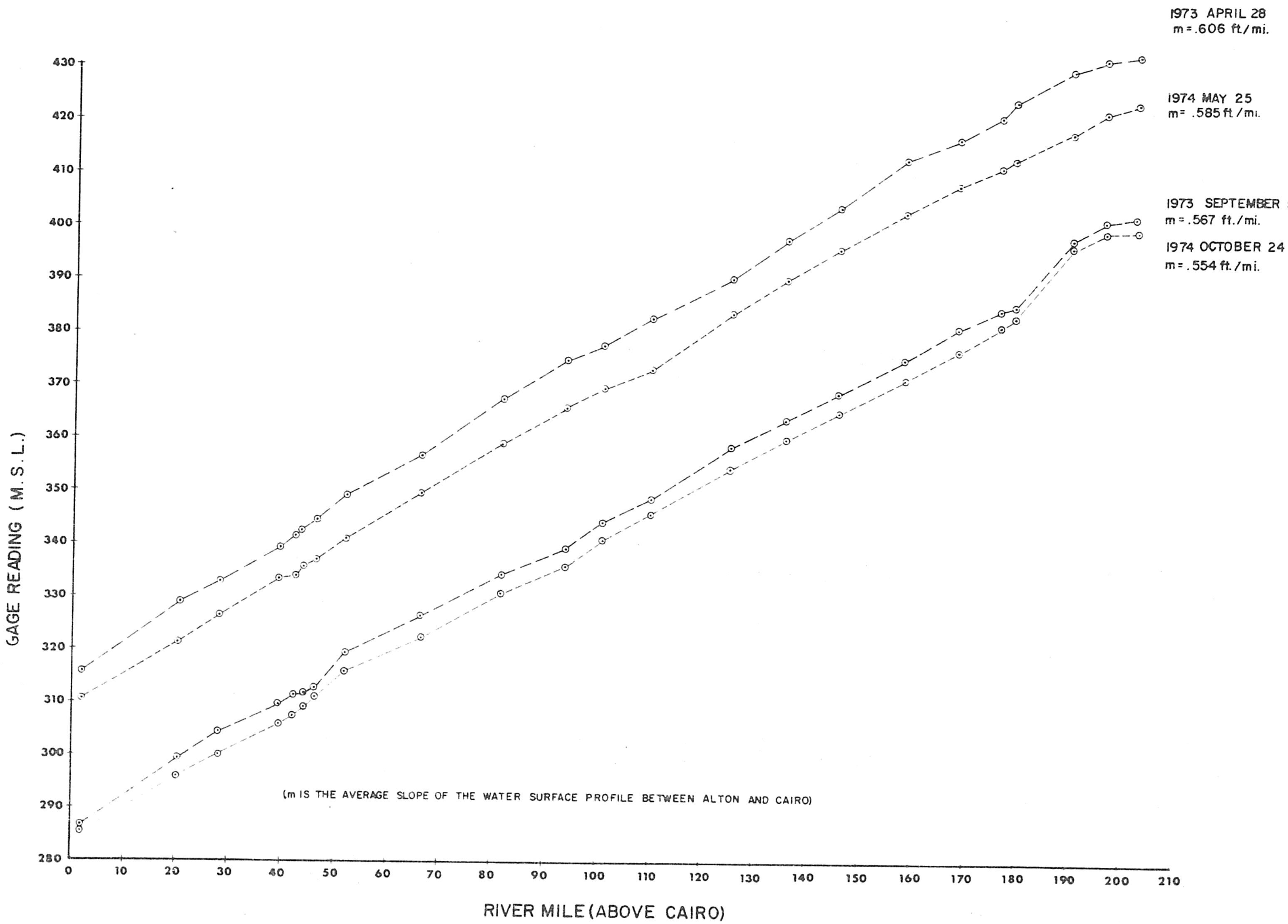

Figure 4.7 Surface Profiles of the Mississippi River Between Alton and Cairo for Maximum and Minimum Stages at St. Louis for 1973 and 1974