Contract Report 645

An Analysis on Managed Flood Storage Options for Selected Levees along the Lower Illinois River for Enhancing Flood Protection

Report No. 4: Flood Storage Reservoirs and Flooding on the Lower Illinois River

by

Abiola A. Akanbi, Yanqing Lian, and Ta Wei Soong

Prepared for the
Office of Water Resources
Illinois Department of Natural Resources

June 1999

Illinois State Water Survey
Watershed Science Section
Champaign, Illinois

A Division of the Illinois Department of Natural Resources

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INTRODUCTION

The lower section of the Illinois River has experienced increased flooding and frequent levee overtopping since the early part of this century. This increase in flooding is partially due to the construction of 36 levee and drainage districts (LDD) on the floodplains, which resulted in the loss of about 180,000 acres or approximately 57 percent of the floodplain for flood conveyance; the acreages are estimated on the basis of the areas inundated in the 1844 flood (Alvord and Burdick, 1919). The effects of LDD storage on flood peaks were observed during the 1993 Midwestern flood on the Upper Mississippi River. During the 1993 Flood, flood stages at Quincy, Illinois and Hannibal, Missouri on the Upper Mississippi River showed clear drops after levee breaches upstream breached (p70, Bhowmik et al. 1995). Such drops meant significant flood protection for towns, cities, and LDDs downstream. However for that 1993 Flood event, due to its immense magnitude, the river stages came back after the LDDs were filled.

The primary goal of the Managed Flood Storage Option Project was to evaluate the benefits that could be gained by reducing flood peaks through converting a few selected LDDs in the Alton and La Grange Pools in the lower section of the Illinois River (Figure 1). Practically all LDDs along the Illinois River are in the lower reach from river mile (RM) 157.7 at Peoria to the confluence with the Mississippi River at Grafton. Figure 1 shows the locations of the LDDs and Appendix I presents the extent and configuration of each LDD along the Illinois River (Illinois Department of Business and Economic Development, 1971).

Existing levees in the Alton and La Grange Pools can protect most of the present LDDs against a 50-year flood. However, only a few levees in the La Grange Pool can provide protection against a 100-year flood. The conventional approach to increase the level of protection of the LDDs in the Alton Pool and the La Grange Pool is to raise the levees. But the drawback to this approach is the prohibitive cost; a 1987 study by the U.S. Army Corps of Engineers (USACOE) showed that raising levees to provide needed protection yields benefit to cost ratios of 0.08 to 0.18; a benefit to cost ratio of 1 is generally considered to be acceptable. Even if a levee were raised, the additional cost of pumping to keep the water table low in the LDD and the consequent reduction in crop yields would make farming in the area behind the levee marginally profitable (Ramamurthy et al., 1989).

An alternative approach, proposed in this study, is to reduce the flood elevations by admitting water into a few selected LDDs during flooding events. By opening a limited section on the selected levee, these selected LDDs can store part of the flood volumes

Figure 1. Levee and drainage districts in the La Grange and Alton Pools
of the Lower Illinois River

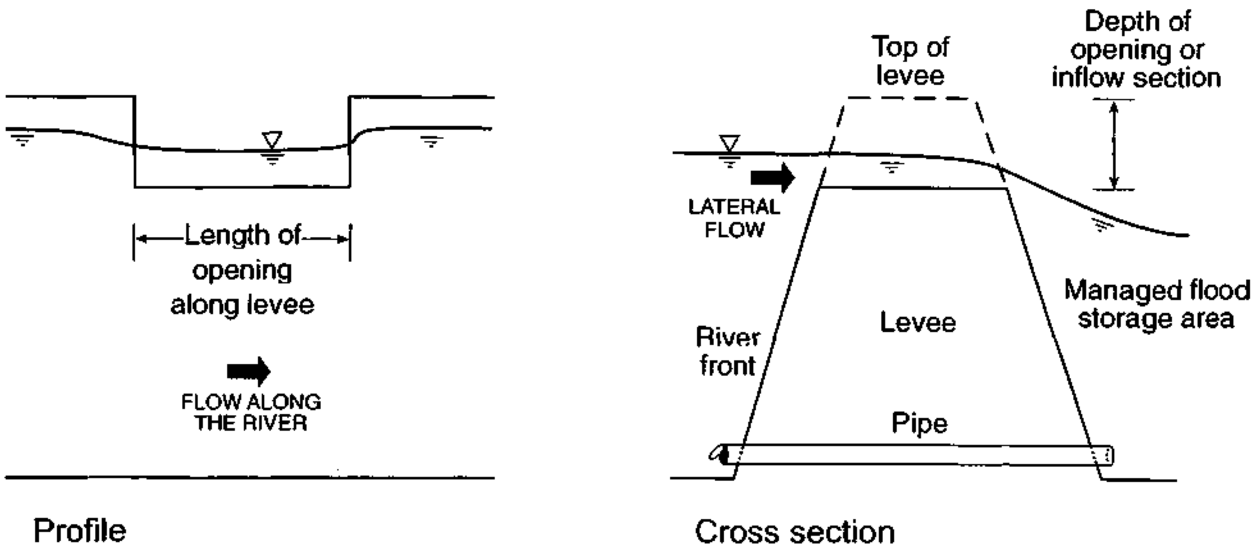

Figure 2. Managed flood storage levee with lateral inflow section

when river stages exceed the elevation of the openings (Figure 2) and contribute to a reduction in flood stages in the lower reach of the river. The effects from managing flood storage depend on the location and acreage of selected individual and combination of LDDs as well as the opening section on each selected levee. When the flood stages in the Illinois River drop below the bottom of the opening, the floodwaters in the storage area can return gradually to the Illinois River (Figure 2). During nonflooding seasons, these converted LDDs also can serve wetland functions and therefore provide added values to the management practices. The goal for evaluating different combinations of managed storage areas is to provide the maximum protection against design floods for other LDDs and at the same time, keep the number of converted LDDs to a minimum. This report describes the work done in the third and fourth phase of this project.

Objectives

The objectives for Phases III and IV are:

- Determine peak flood profiles for the 25-, 50-, and 100-year return periods.
- Examine the variability of flow in the Lower Illinois River and the timing and variability of flood peaks from tributaries of the Illinois River.
- Continue the UNET model simulations to determine changes in flooding elevations at selected sites and downstream through the La Grange and Alton Pools for various simulated floods and for various overflow section widths and elevations.
- Evaluate of reduction in peak flood stages due to conversion of selected LDDs in the Alton and La Grange Pools to managed flood storage areas.
- Outline the economic benefits, including costs, of conversion of the areas behind selected levees for managed flood storage.
- Perform UNET model simulations to estimate the change in stages along the Big Swan levee due to sediment accumulation on the floodplains.

Methodology

Due to the transient nature of flood waves, an unsteady flow model is necessary for this project. Unlike the steady-state calculations, the peak stages computed with the unsteady flow model do not produce an instantaneous profile of the flooding condition on the whole river. Therefore, the UNET model selected is described briefly in the following section. Evaluating optimal openings on the levees and determining effects on flood peak reductions from storage provided by individual LDDs and a combination of LDDs require modifying the unsteady flow model developed previously to describe the openings on the levees and perform numerical simulations using "design floods". The design floods are derived on the basis of previous work on the analyses of historical floods, which determined flood magnitudes and frequencies at stations along the Lower Illinois River.

Previous Work

Flood Frequency Analysis

Singh (1996) and Akanbi and Singh (1997) presented frequency analysis of peak flood discharge and stage. The flood frequency analysis involves the development of discharge-frequency relationships, using both log-Pearson Type III and mixed distributions (Singh, 1996) for gaging stations at Marseilles, Kingston Mines, and Meredosia on the Illinois River, and for gaging stations on the five major tributaries to the Illinois River including Mackinaw, Spoon, Sangamon, La Moine, and Macoupin. Table 1 shows the peak flood discharges and associated recurrence interval for these tributary stations and five other stations (Big Bureau, Big, Hadley, Bay Creeks, and Spring Lake) representing ungaged tributaries. Stage-frequency relations also were developed for Illinois River gages at Peoria Lock and Dam (L&D), Kingston Mines, Havana, Beardstown, La Grange L&D, Meredosia, Valley City, Florence, Pearl, Hardin, and Grafton. Singh (1996) and Akanbi and Singh (1997) reported the peak stages for different recurrence intervals (Table 2). The results in these tables were used to develop the 2-, 10-, 25-, 50-, and 100-year discharge and stage hydrographs required for the boundary conditions in the unsteady flow simulations.

Unsteady Flow Modeling on the Lower Illinois River

An unsteady flow model was selected for simulating historical floods along the La Grange (RM 157.7 to 80.1) and Alton (RM 80.1 to 0.0) Pools of the Lower Illinois River. The unsteady flow model, the UNET (HEC, 1993), solves the Saint Venant equations so it is appropriate for evaluating flood wave propagation with backwaters to tributaries or due to L and D structures. The Lower Illinois River UNET model also included tributaries and lateral inflows networks, levees, storage, and many other local features. This Lower Illinois River UNET model was derived from another Illinois River UNET model that was obtained from the USACOE Rock Island District (personal communication).

The Lower Illinois River UNET model consisted of a total of 412 cross sections. A section of the Sangamon River, the largest tributary in the lower reach of the Illinois River, from the confluence to Oakford was included. Akanbi and Singh (1997) calibrated parameters in the Lower Illinois River UNET model and validated it using the May 1979 and March 1985 floods. These floods are ranked, respectively, fourth and second at Meredosia, and sixth and third at Kingston Mines. Computed water surface elevations (WSEs) were compared with the recorded events at eight gaging stations between Peoria L&D and Grafton. Results showed that computed WSE profiles for these flood events fit the observed data generally within 0.5 foot for the stations in the La Grange Pool and generally within 1 foot at the stations in the Alton Pool using a Peoria L&D-Grafton one-reach model. The larger discrepancies in the Alton Pool were probably caused by the specification at the La Grange L&D, which was treated as a cross section. To improve the fit in the Alton Pool, a two-reach model also was devised from Peoria L&D to La Grange

Table 1. Peak Flood Discharges (cfs) at Given Recurrence Intervals for Selected Tributary Stations

Tributary	*2-year*	*10-year*	*25-year*	*50-year*	*100-year*	*500-year*
Mackinaw River	10,300	27,600	39,600	50,000	62,000	97,800
Spoon River	14,000	26,100	32,300	37,800	41,000	51,800
Sangamon River	24,500	46,300	59,000	72,000	87,500	138,000
LaMoine River	12,000	23,000	26,800	31,000	35,000	45,000
Macoupin Creek	10,800	22,800	29,300	34,600	39,600	59,400
Big Bureau Creek	4,210	8,100	10,000	11,500	12,800	15,500
Big Creek	773	1,157	1,336	1,465	1,592	1,882
Spring Creek	1,840	5,750	8,300	10,500	13,200	20,000
Hadley Creek	7,339	12,650	15,240	17,337	19,811	25,162
Bay Creek	5,134	10,541	13,210	15,291	17,491	23,499

Source: Akanbi and Singh, 1997, Table 9.

Table 2. Peak Stages (feet, NGVD 1929) for Gaging Stations in the Lower Illinois River

Gaging station	*2-year*	*10-year*	*25-year*	*50-year*	*100-year*	*500-year*
Peoria L&D TW	447.02	452.30	454.43	455.92	457.28	460.21
Kingston Mines	445.68	450.82	453.02	454.55	456.00	459.19
Havana	442.49	447.79	450.11	451.68	453.14	456.26
Beardstown	439.08	445.51	447.91	449.52	451.02	454.21
La Grange L&D TW	437.62	443.73	446.20	447.90	449.48	452.92
Meredosia	436.72	442.92	445.38	447.08	448.59	451.89
Valley City	434.85	441.22	443.84	445.61	447.29	450.88
Florence	433.73	440.19	442.85	444.66	446.35	449.97
Pearl	431.18	437.44	440.15	442.07	443.91	448.01
Hardin	427.58	434.55	437.49	439.53	441.47	445.91
Grafton	425.23	432.52	435.81	438.09	440.24	444.86

Source: Akanbi and Singh, 1997, Table 11.

L&D and from La Grange L&D to Grafton. The computed WSE for the 1979 flood using the two-reach model fit the observed WSE more closely than the one-reach model. However this two-reach model required specific information (i.e., the stage-discharge rating information at the La Grange L&D), hence it is not used in later simulations. The one-reach model was further validated by simulating the December 1982, June 1974, April 1973, and July 1993 flood events (Akanbi and Singh, 1997). However, the flood of May 1943, the highest flood at Meredosia and the second highest at Kingston Mines, was not simulated because of missing data in the records of some of the tributaries (Akanbi and Singh, 1997).

It is necessary to clarify that the present model reflects only the updated levee information. The levee crown elevations were obtained from the USACOE, dated May 1981. Floods had damaged a few of the levees, and these LDDs were modified for other uses or left behind unrepaired. For example, the Chautauqua LDD (3,320 acres) was overtopped in 1926 and was subsequently converted to a conservation area. The Big Prairie LDD (1,800 acres) was damaged in 1936 and was left to deterioration by natural processes. Moreover, Rocky Ford LDD (1,616 acres) was converted to a reservoir to provide cooling water for hydroelectric power generation. Comparisons with published levee elevations (Illinois Department of Public Works, 1952) indicated that there had been levee raises at South Beardstown and Kelly Lake. Also that flood heights presented in this report may be different from those observed in the 1930s, 1940s, or even 1950s because of different levee heights.

Acknowledgments

This study was conducted by Abi Akanbi of the Illinois State Water Survey and Yanqing Lian of the Hydrosystems Laboratory at the University of Illinois at Urbana-Champaign. Partial funding for the project was provided by the Office of Water Resources, Illinois Department of Natural Resources. Gary Clark, Office of Water Resources, served in a liaison capacity during the entire course of the study. Principal Scientist Emeritus, Krishan P. Singh, who initiated the study in 1994, provided valuable advice on various aspects of the project. He, TaWei David Soong (Illinois State Water Survey), and Bob Dalton (Office of Water Resources) provided technical review of the report. Following the reviewers' comments, TaWei David Soong, with assistance from Nani G. Bhowmik, Head of the Watershed Science Section, Illinois State Water Survey, revised the report after Akanbi left the Water Survey. Linda Hascall assisted with the graphics, and Eva Kingston and Agnes Dillon edited the report. The views expressed in this report are those of the authors and do not necessarily reflect the views of the sponsor or the Illinois State Water Survey.

DESIGN FLOODS FOR MODEL SIMULATIONS

In addition to calibration and verification of the Lower Illinois River UNET model, "design floods" that reflect realistic hydrologic conditions of the Lower Dlinois River are needed. Using the design floods with the model, one can define the "existing condition" for peak WSE profiles, which then served as the basis for evaluation and selection of LDDs for managed storage and for defining benefits. The design floods relate to the initial and boundary conditions specified in the model.

In this dendritic model, there are many ways to combine boundary conditions and initial conditions to obtain a WSE of a specific recurrence period. The boundary conditions involve either stage or inflow hydrographs at upstream (Peoria L&D) and downstream (Grafton) stations and tributary inputs. The timing of the tributary flows will affect the magnitude of the flood on the main river. For instance, if the peak flow of 123,000 cubic feet per second (cfs) that occurred at Oakford on the Sangamon River on May 20, 1943, had arrived at the mouth of the Illinois River and coincided with the peak flow of 83,100 cfs that occurred at Kingston Mines on May 23, 1943, the resulting peak flow at Meredosia on May 26, 1943, would have greatly exceeded the 123,000 cfs that was recorded that day.

Analysis of the annual peak flows at Kingston Mines and Meredosia also indicated that the 1993 floods at these stations had a recurrence interval of three to five years. The 1993 high flood stages observed in the Illinois River were not due to a major flood in the river itself. Instead, the high stages were due to the prolonged and unprecedented flooding on the Upper Mississippi River that caused backwater effects upstream on the Illinois River. The upstream extent of this backwater effect was estimated by ranking the annual peak WSE data (1941-1993) for the stations between Meredosia and Kingston Mines. Ranks for the 1993 flood stages at these stations indicated that the backwater effects extended up to Havana (RM 119.6) but not as far as Kingston Mines (RM 145.6).

These two instances exemplified the complexity in combining the incoming stage or discharge hydrographs and downstream boundary conditions.

Tributary and Lateral Inflow Hydrographs

Tributary flows and other lateral inflows are usually single-value estimates in the conventional steady-state analysis for flood routing in streams. However, flow hydrographs are specified at the tributary junctions and at lateral inflow sections in the UNET unsteady flow modeling. For the simulations in this study, the flow hydrograph for a tributary or lateral inflow section for a selected recurrence interval was obtained by considering that a representative or typical hydrograph exists for each station. Derivation of inflow hydrographs for gaged and ungaged tributaries are explained as follows.

Using records from the five major tributary stations (Mackinaw, Spoon, Sangamon, La Moine, and Macoupin), six top floods at each station were selected. Duration of each flood was selected as 20 days, including 10 days before and 10 days after the occurrence of the peak discharge. By matching the day of peak flows, the six hydrographs plotted with ordinates normalized by the corresponding peak discharge. A representative normalized hydrograph for each station then was obtained from the six hydrographs (Figures 24-33 in Akanbi and Singh, 1997). As much as possible, the representative hydrographs were drawn as closely as possible to the first three top-flood hydrographs. At ungaged streams (Big Bureau, Big, Hadley, Bay Creeks, and Spring Lake), synthetic hydrographs were derived by multiplying the area ratio with the nearest gaging station. For each design flood with a specified recurrence period, the inflow hydrograph is then obtained by multiplying the ordinates of the representative normalized hydrograph with the corresponding peak flow in Table 1. The normalized inflow hydrographs at each gaged and ungaged stream are shown in Figures 3 and 4, respectively.

Upstream and Downstream Boundary Conditions

The upstream and downstream boundaries of the one-reach model are at Peoria L&D and Grafton, respectively. Hydraulic flood routing usually specifies the boundary condition for a reach as a stage hydrograph at one end and a discharge hydrograph or rating relation at the other end. A stage hydrograph was specified at Peoria. The normalized stage hydrographs for this station were determined with a procedure similar to the one outlined in the previous section and then were multiplied by the peak WSE from Table 2. Figure 5 shows the normalized stage hydrograph for Peoria, La Grange, and Grafton.

At Grafton, the downstream boundary condition should be either a discharge hydrograph or a stage-discharge rating relation in the usual modeling approach. However, it was not possible to develop a clearly defined stage-discharge relation for Grafton similar to the one for La Grange. Grafton is a gaging station of the Upper Mississippi River; its flow records are the combination of those from the Upper Mississippi River and the Illinois River. The stage at Grafton also is controlled by the Melvin Price L&D (L&D26) at 15.3 miles downstream from Grafton. Many methods have been explored in" an attempt to derive an acceptable stage-discharge rating curve at Grafton for the Illinois River. However, the scatteredness of data led to a different set of rating curves; subsequent tests with the UNET model indicated any set of these rating curves cannot be used in all design floods for this project. Figure 6 shows the scatteredness of data. Clearly one can observe the data become more scattered as the discharge increases. The discharge data in the plot were the discharges at Meredosia. Dyhouse (1984) also has illustrated a scatter diagram of peak discharge against water-surface elevation at Grafton.

The UNET model allows and can handle stage-stage boundary conditions. Akanbi and - Singh (1997) have shown that the accuracy of the simulation results using the stage-stage conditions is sometimes superior to the stage-discharge boundary specification. Because the flow in the Upper Mississippi River is much larger than that in the Illinois River, the stages at Grafton would be governed by the flow in the Mississippi River. It should, therefore, be adequate to use a stage hydrograph at this boundary. The normalized stage hydrograph for Grafton (Figure 5) was developed using the approach described above.

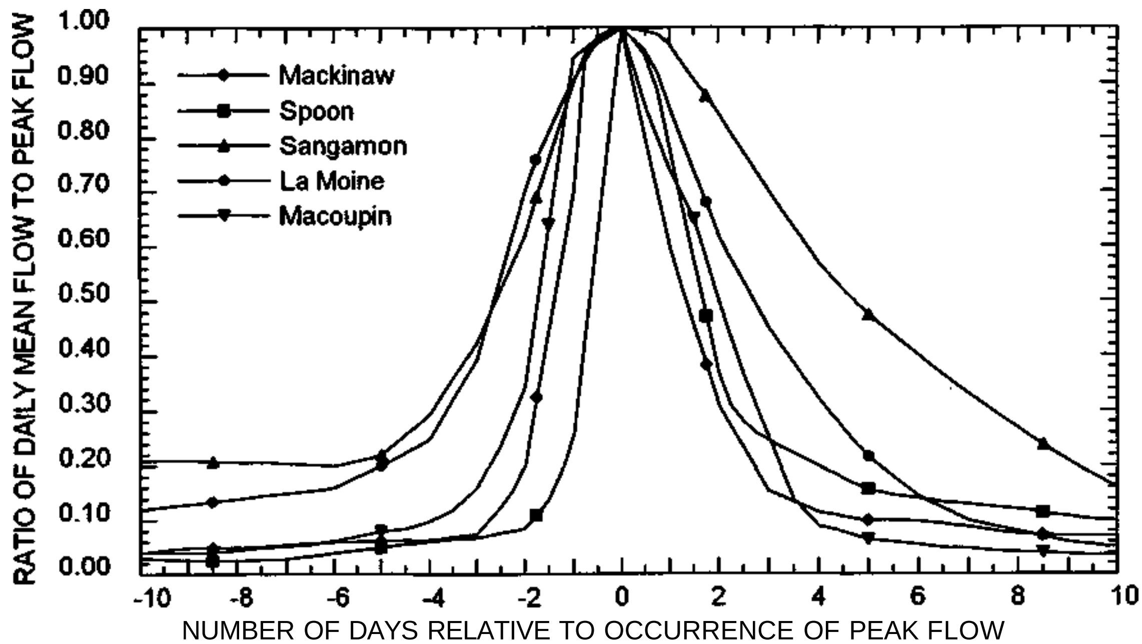

Figure 3. Normalized flow hydrographs for the five major tributaries
on the Lower Illinois River

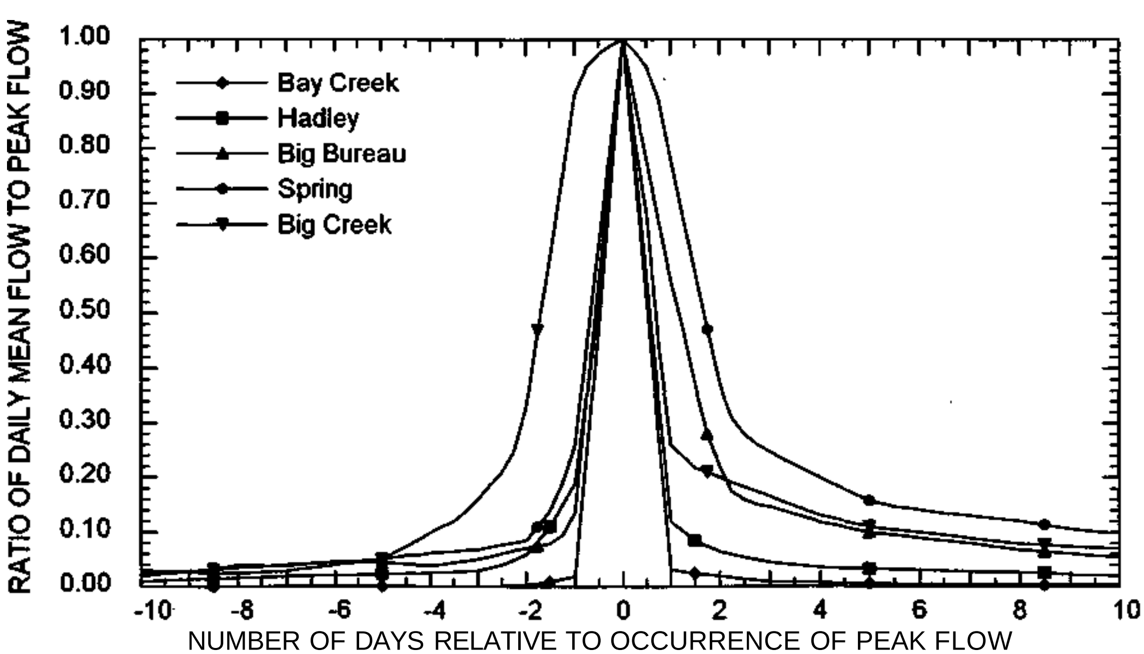

Figure 4. Normalized flow hydrographs for the streams representing
ungaged tributaries on the Illinois River

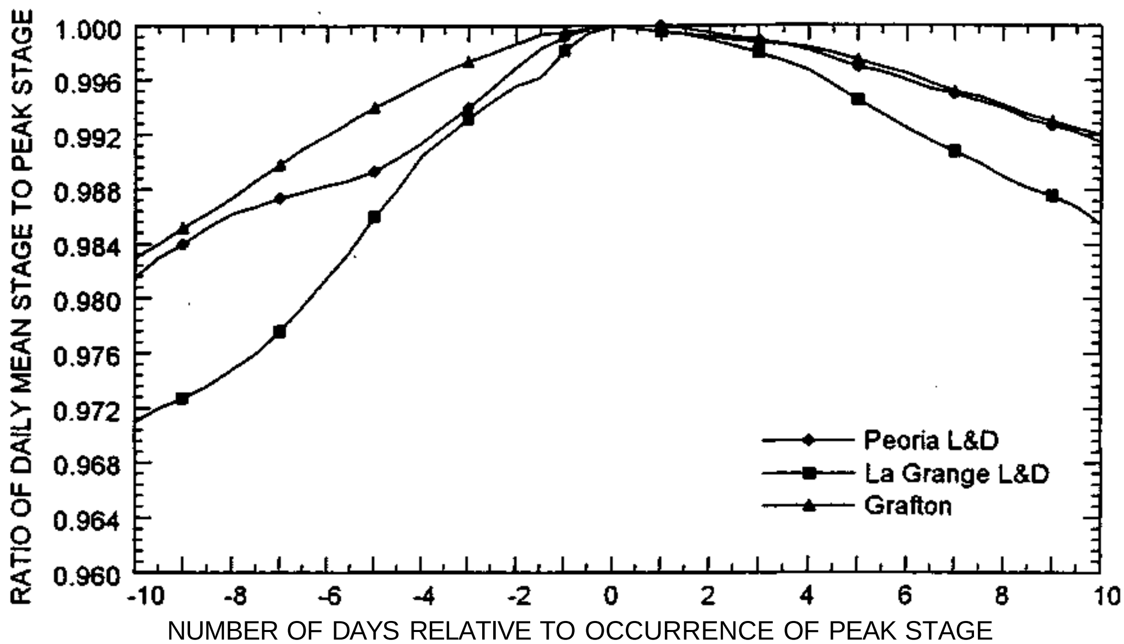

Figure 5. Normalized stage hydrographs for Peoria Lock and Dam (L & D), La Grange L&D, and Grafton

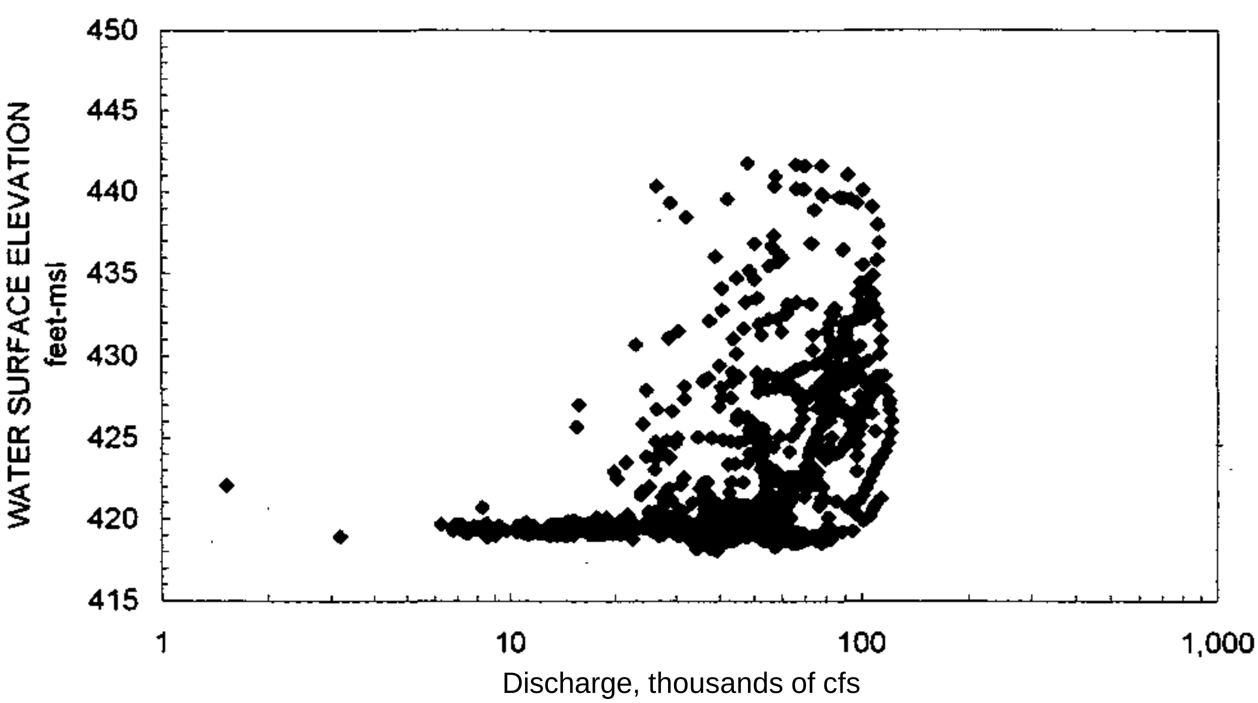

Figure 6. Stage-discharge relations at Grafton for the 1973, 1974, 1979, 1982, 1985, and 1993 floods using computed discharges from unsteady flow simulations

Interaction of Flows in the Illinois River and Its Tributaries

The synthetic flow and stage hydrographs generated in the previous section have to be lagged appropriately to reflect the dynamics of the flows in the Illinois River and its tributaries. The time lags were estimated by examining the timing of the peak floods for the floods of 1943, 1974, 1979, 1982, and 1985 (Akanbi and Singh, 1997), the same floods that were used to validate the model. It takes six to seven days for tributary flood peaks to reach Meredosia and about five days from Kingston Mines to Meredosia.

The interaction of flows in the Illinois River and its tributaries was examined by studying the frequencies of historical floods on the Illinois River and the frequencies of floods on the tributaries. Because Meredosia and Kingston Mines are the only Illinois River stations in the study reach with discharge records, the frequencies of floods on the major tributaries and the gages representing ungaged streams were related to these stations. The annual maximum peak flow data from 1941 to 1993 were ranked at Meredosia and Kingston Mines and at the tributary stations. Table 3 shows the top ten floods at Meredosia and the corresponding rank of each of those floods at tributary stations in the La Grange and Alton Pools. This information on the ranking of the peak flow data at Meredosia and the tributaries has been used to develop relationships between the frequencies of Illinois River flow at Meredosia and flows in the tributaries as shown in Table 4. The information on the flow frequencies in the table was used in the simulation of design floods described in the next section.

Design Floods for Simulations

The "design floods" were the selected combinations of inflow for the Lower Illinois River UNET model that could simulate WSE closely to match the 25-, 50-, and 100-year peak stages. The peak stage information was obtained from the stage frequency analysis (Table 2). To simulate the WSE for a particular recurrence interval, the flow and stage frequencies in Table 4 were used to select appropriate stage hydrographs for boundary conditions at Peoria L&D and Grafton as well as discharge hydrographs for tributary flows. The combination of inflows and up- and downstream stage hydrographs in each column is called the "Design Floods" for that recurrence period. For instance, the 100-year flood profile along the Illinois River was produced with 50-year stage hydrographs at Peoria L&D and Grafton; 50-year flow hydrographs (Mackinaw, Sangamon, and Macoupin); 10-year flow hydrographs (Spoon and La Moine); and 2-year hydrographs at ungaged tributaries and lateral inflow sections. Figure 7 shows the simulated 25-, 50- and 100-year WSE profiles along the lower Illinois River and their comparisons with peak - stages.

The 25- and 50-year WSE profiles closely fit the stage frequency analysis values at the gages on the Illinois River. The 100-year profile also produced a good match along the river, with the exception of the reach between Beardstown and Meredosia. The apparent large deviation at Peoria and Havana was due to the 50-year stage hydrograph that was applied as a boundary condition at Peoria L&D based on the stage-frequency information in Table 3b. As mentioned earlier, there are many ways to combine inflows and derive

Table 3. Ranked Annual Maximum Daily Flows at Meredosia and the Tributary Stations in the La Grange and Alton Pools

Meredosia				*Flood ranks*									
Year	*Date*	*Flood flows (cfs)*	*Rank*	*Mackinaw*	*Spoon*	*Sangamon*	*La Moine*	*Macoupin*	*Big Bureau*	*Big*	*Spring Lake*	*Hadley*	*Bay*
1943	5/26	123,000	1		32	1	16	1	27			21	33
1985	3/10	120,000	2	7	2	17	1	9	10	9	29	13	32
1982	12/12	112,000	3	2	11	2	5	6	22	5	25	10	44
1974	6/29	110,000	4	6	1	6	27	20	1	2	21	20	19
1979	4/19	109,000	5	10	29	3	33	5	6	7	7	14	6
1944	4/29	101,000	6		18	5	8	2	18			1	24
1973	5/2	101,000	7	11	21	4	21	26	8	6	10	6	4
1983	4/19	94,600	8	26	7	27	12	13	20	3	2	2	31
1970	5/22	94,000	9	24	5	7	3	12	14		9	9	7
1962	3/29	90,500	10	18	33	21	31	15	16		22	22	17

Note: The period of record was 19 years for Big, 40 years for Spring Lake, 44 years for Hadley, 30 years for Bay, and 50 years for all other stations.

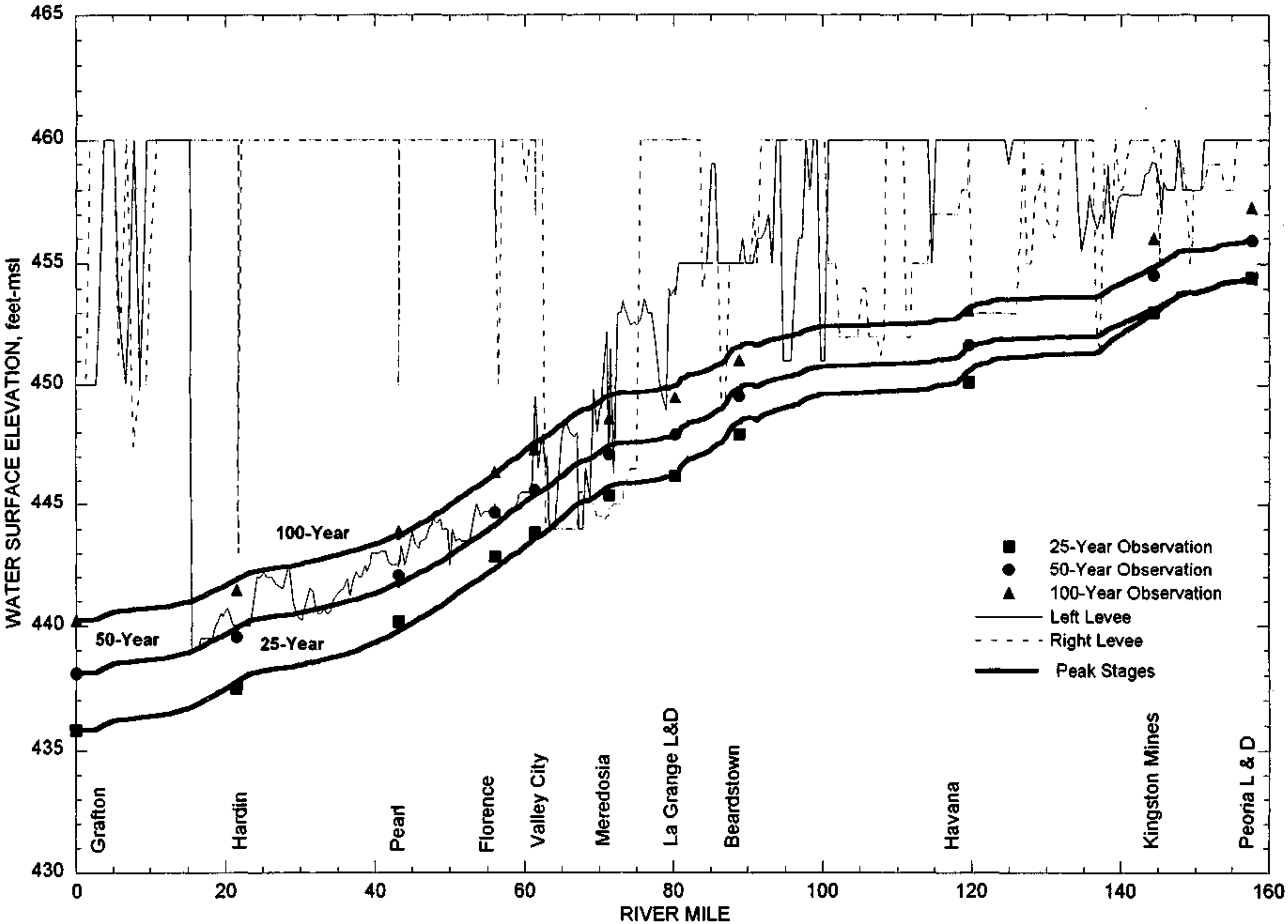

Figure 7. Peak stages for various return periods computed from UNET simulations
and stage frequency analysis according to the design floods

Table 4. Flow and Stage Frequency Relationships Between the Illinois River and Its Tributaries

	Return period for Illinois River stages		
Station	*25-Year*	*50-Year*	*100-Year*
	Selected Discharge Hydrograph from Tributaries		
Mackinaw	25	25	50
Spoon	2	10	10
Sangamon	25	50	50
La Moine	2	10	10
Macoupin	2	50	50
	Selected Stage Hydrograph at Illinois River Stations		
Peoria L&D	25	25	50
Grafton	25	50	100

Note: A 2-year recurrence interval was assumed for all ungaged streams and lateral inflows.

the desired recurrence period, and the investigators have exhausted possible combinations and evaluated their applicability. For instance, when the 100-year stage hydrograph was applied at Peoria L&D in one of the test runs with no lateral inflow and 2-year flow from the tributaries, the resulting WSE profiles were even closer to the analyzed peak stages (Figure 8.) However, this scenario represented a solitary flood wave from upstream that was routed through the channel without any significant input from tributaries. The probability for such a case was considered less possible than the presently selected combinations. After consulting with the previous principal investigator, Krishan P. Singh, the later combination was not used. Therefore, the analyses conducted in this report are applicable to the conditions suitable for the design floods. However, Figure 8 illustrates that the model was properly calibrated for the Lower Illinois River.

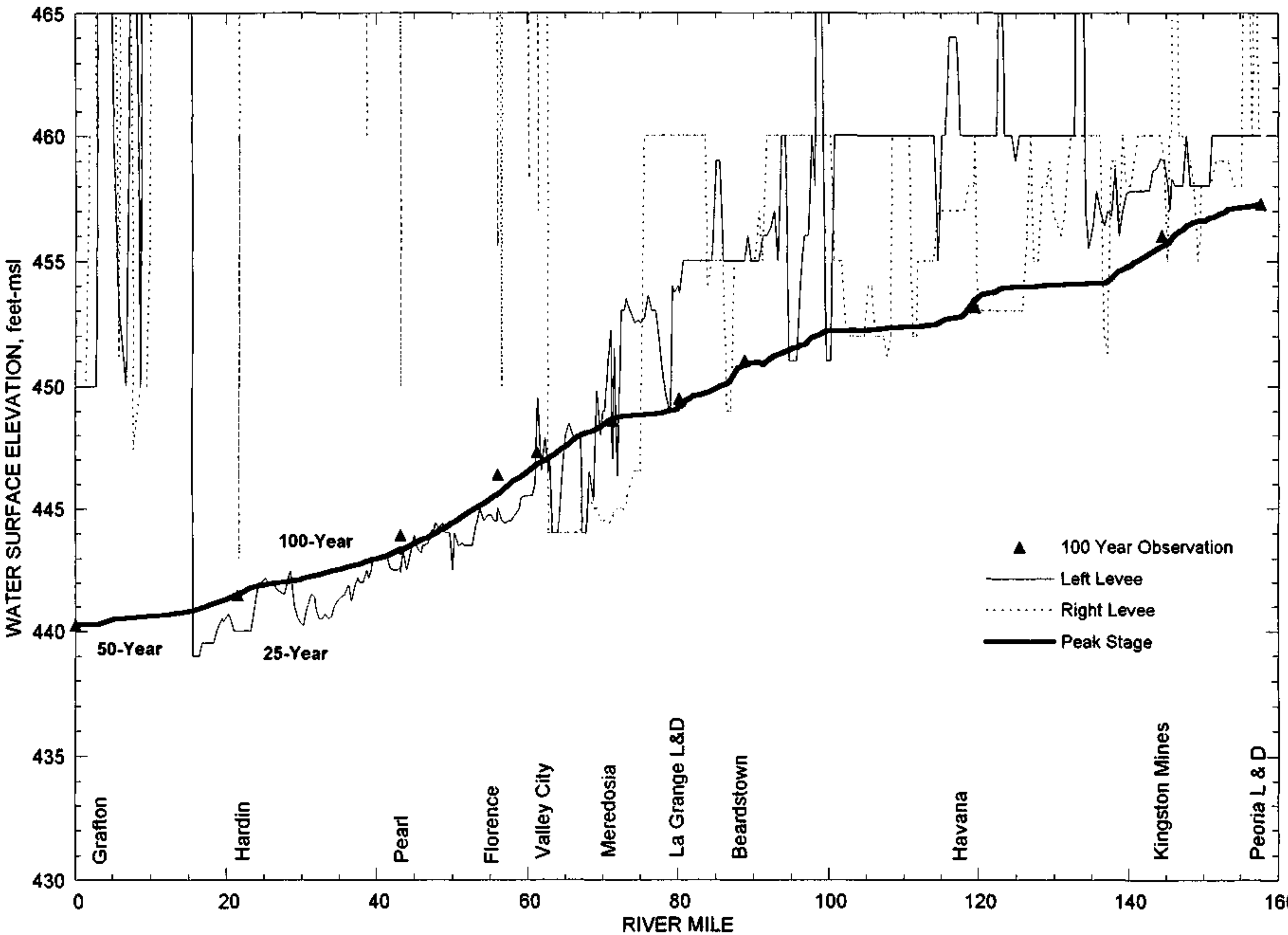

Figure 8. Peak Stages for 100-year return periods computed from UNET simulations and stage frequency analysis according to 100-year stage hydrograph at Peoria and 2 year inflow from tributaries and later inflow sections

EFFECT OF MANAGED FLOOD STORAGE ON FLOOD PEAKS

To evaluate the potential impact on WSE profiles due to the conversion of selected LDDs in the La Grange Pool and the Alton Pool for managed flood storage areas, two scenarios were tested with the Lower Illinois River UNET model. The model was first run with design floods to define the existing WSE profiles, the model was then modified to include lateral inflow sections on selected levees and run for the same design flood for evaluating changes. Because the levee heights in both pools were sufficient for protecting the 50-year flood, the following investigations were for a 100-year design flood.

Optimal Size for Lateral Inflow Sections

The rate of inflows to the LDDs, and the location and available volume of LDDs are all controlling factors for the dimension of inflow sections. For practical purposes, a managed storage area was represented by an opening ranging from 250 to 4000 feet along the levee and 2 to 6 feet below the top of the levee. Profiles then were computed for various dimensions of the inflow section for all the levees between Peoria and Pearl (RM 43.2). Levees below Hillview were not considered for the managed storage option because the WSEs are governed by the backwaters from the Mississippi River. Figure 9 is a plot showing the relationships between the width of the opening and the maximum volume stored (after the peak passed and there were no more inflows) at Lacey LDD, and the peak stage of the whole reach. Clearly an opening approximately 1000 feet reached the optimal condition. Figures 10 and 11, respectively, show the variations in the peak WSEs against depth and width of the inflow section, for Spring Lake, McGee Creek, and Lacey-Langellier-W. Matanzas-Kerton Valley (Lacey) LDDs (see Figure 1 for locations). These figures show that, in general, a 1000-foot length and 4- to 6-foot depth of lowered sections are the most promising in lowering the flood elevations.

Flood Stage Reduction for Individual Storage Reservoirs

Table 5 shows the maximum reduction in peak WSE for individual managed storage LDD due to a 1000 foot opening with either a 4 or 6 foot depth. The location of the maximum drop on the whole reach was identified on the column of river mile. For the 4-foot depth of opening, significant reductions in the peak WSE were found, in sequence from large to small but all greater than 0.45 foot, at McGee Creek, Scott County, Spring Lake, Thompson Lake, Lacey, and Crane Creek LDDs; for the 6-foot opening the sequence was: Lacey, Spring Lake, McGee Creek, Scott County. Appendix II shows the peak WSE profiles along the study reach for a 1000 foot by 6 foot opening on the levee of each LDD. Figure 12 depicts the reduction in peak stages at these six levees.

Flood Stage Reduction for Combined Storage Reservoirs

Based on the results for individual levees, some of the managed storage LDDs were combined to produce the greatest reduction in stages that are feasible in the La Grange and Alton Pools. Table 6 is a summary of simulations that show the reduction in peak stages for the combination of Spring Lake with each of the following LDDs: Lacey,

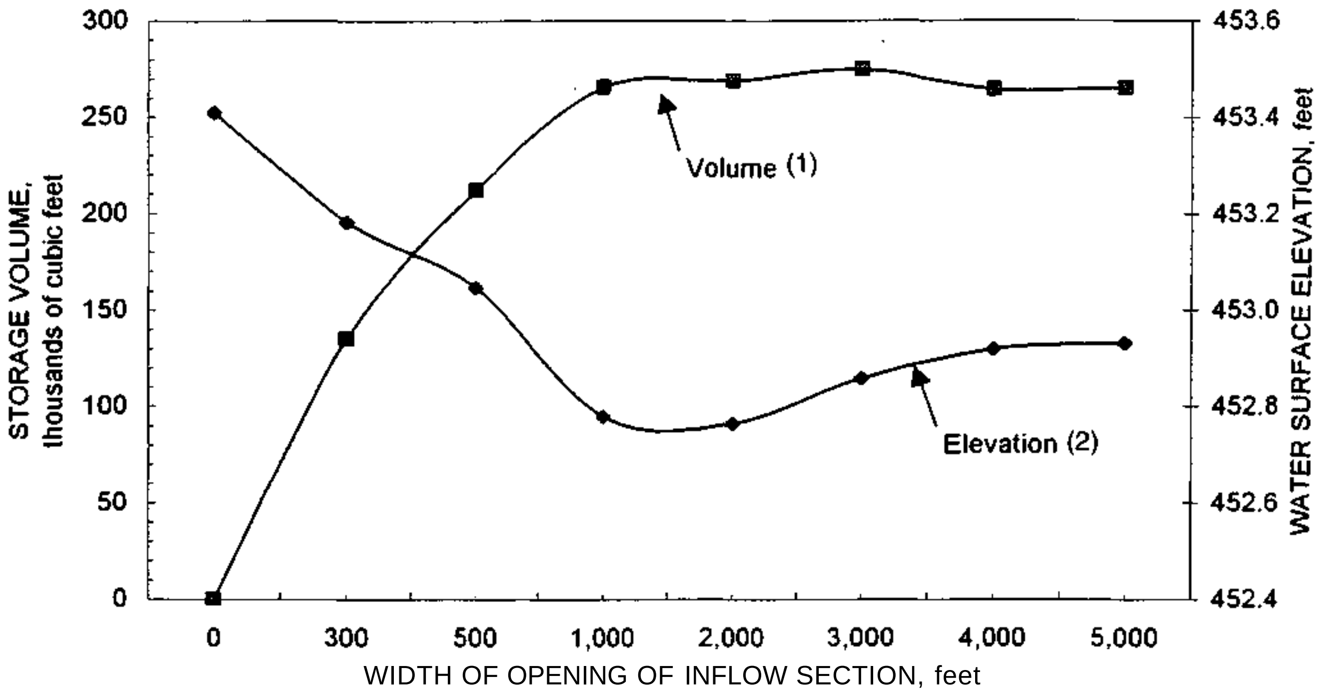

Figure 9. Relationships between the width of the opening and (1) the maximum volume stored (after the peak passed and no more inflows) at Lacey LDD, and (2) the peak stage of the whole reach

McGee Creek, Crane Creek, and Scott County LDDs; and Lacey with each of the following LDDs: McGee Creek, Crane Creek, and Scott County LDDs. The combined managed storage areas of Lacey-McGee Creek LDDs produced the largest reduction of 1.27 feet in peak stage at RM 70.8 followed by combinations of Spring Lake-McGee Creek, Spring Lake-Lacey, and Spring Lake-Scott County LDDs with peak stage reductions of 1.26, 1.13 and 1.04 feet, respectively. Figure 13 depicts the changes in the peak stages. Figures 14 and 15 depict the reduction in the 100-year peak WSE profiles resulting from the conversion of Lacey-McGee and Spring Lake-Scott County LDDs to managed storage areas. Figure 14 shows that practically all the levees are safe against the

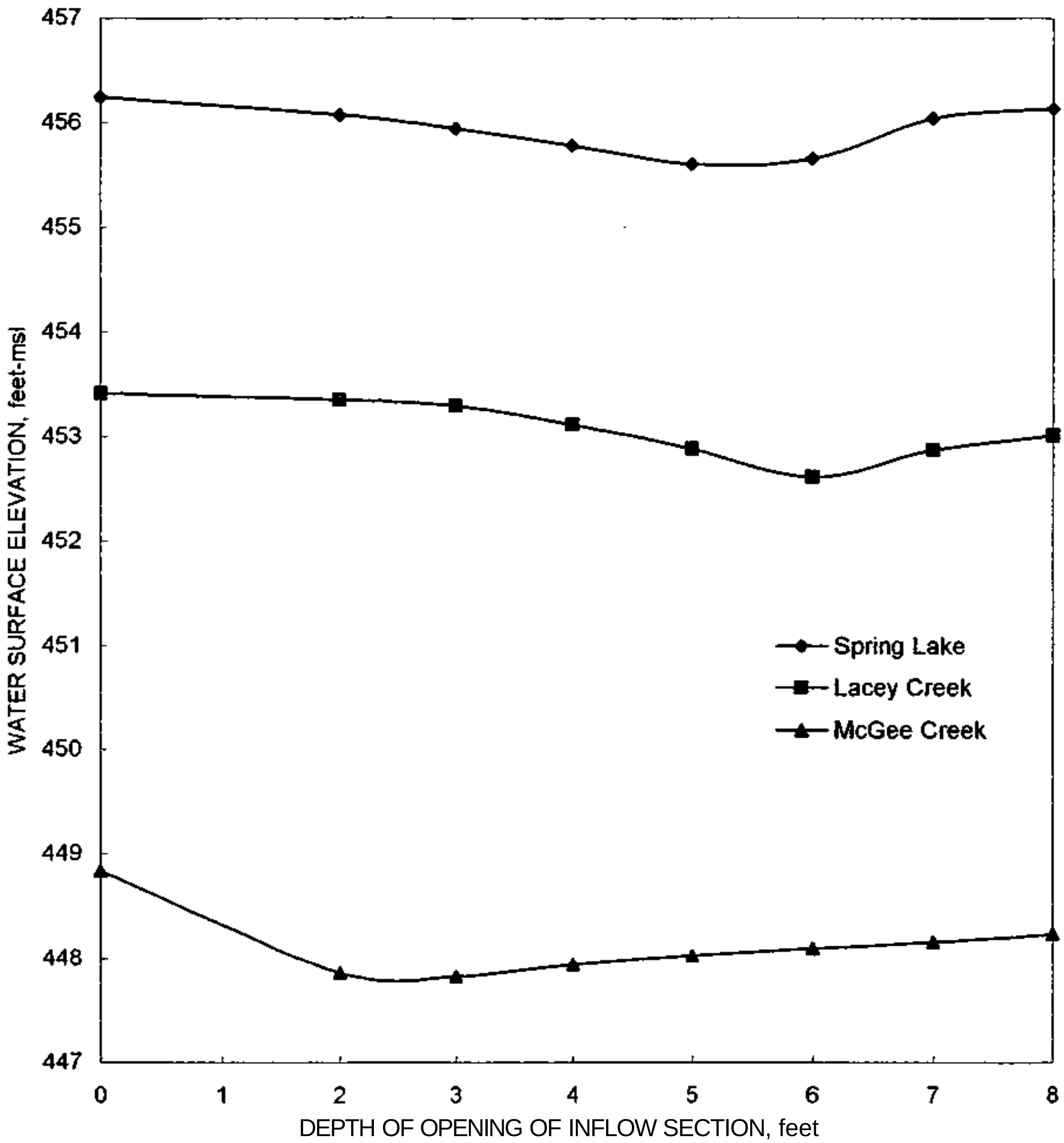

Figure 10. Changes in water surface elevation with varying depth of inflow section at Spring Lake, Lacey, and McGee Creek Levee and Drainage Districts

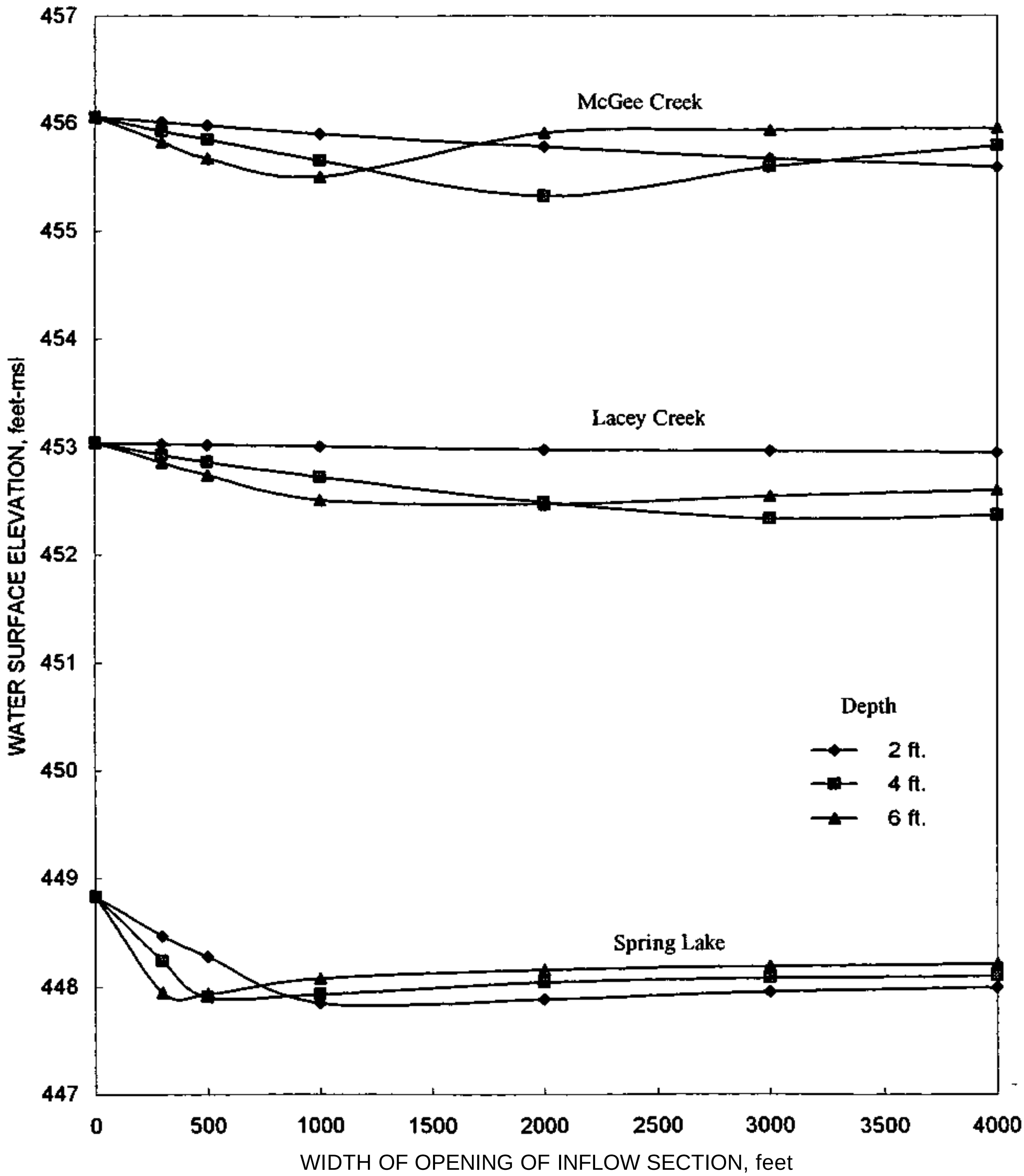

Figure 11. Reduction in water surface elevation with varying width of inflow section at Spring Lake, Lacey, and McGee Creek Levee and Drainage Districts

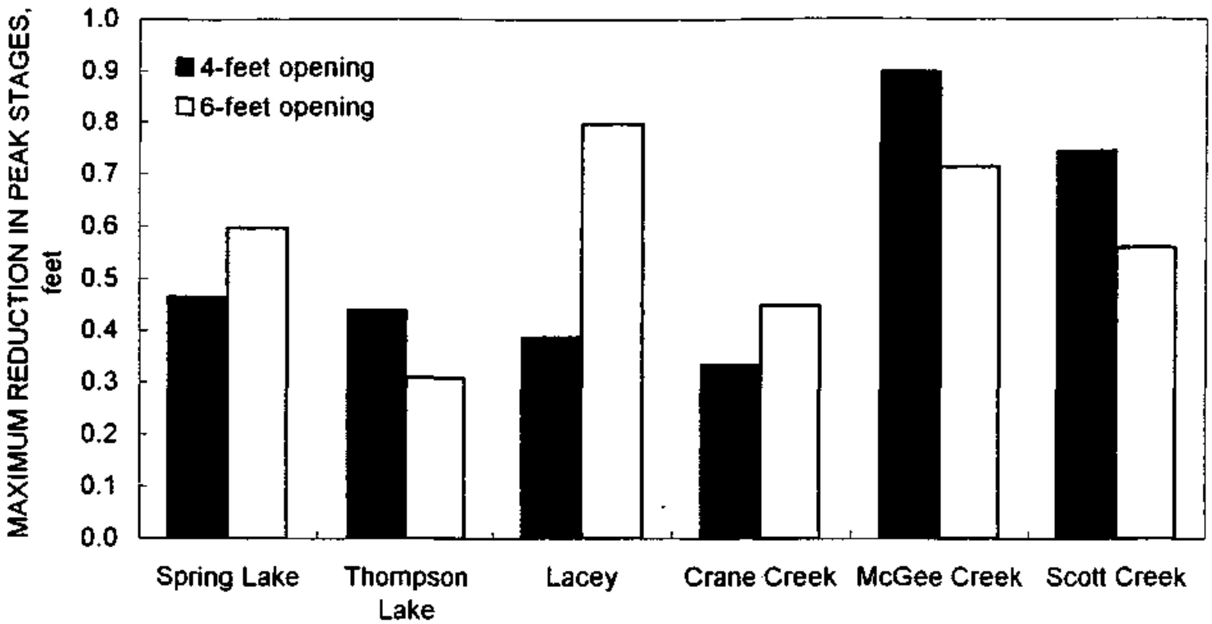

Figure 12. Reduction in stages due to combined managed storage areas for 4-foot and 6-foot depth of opening

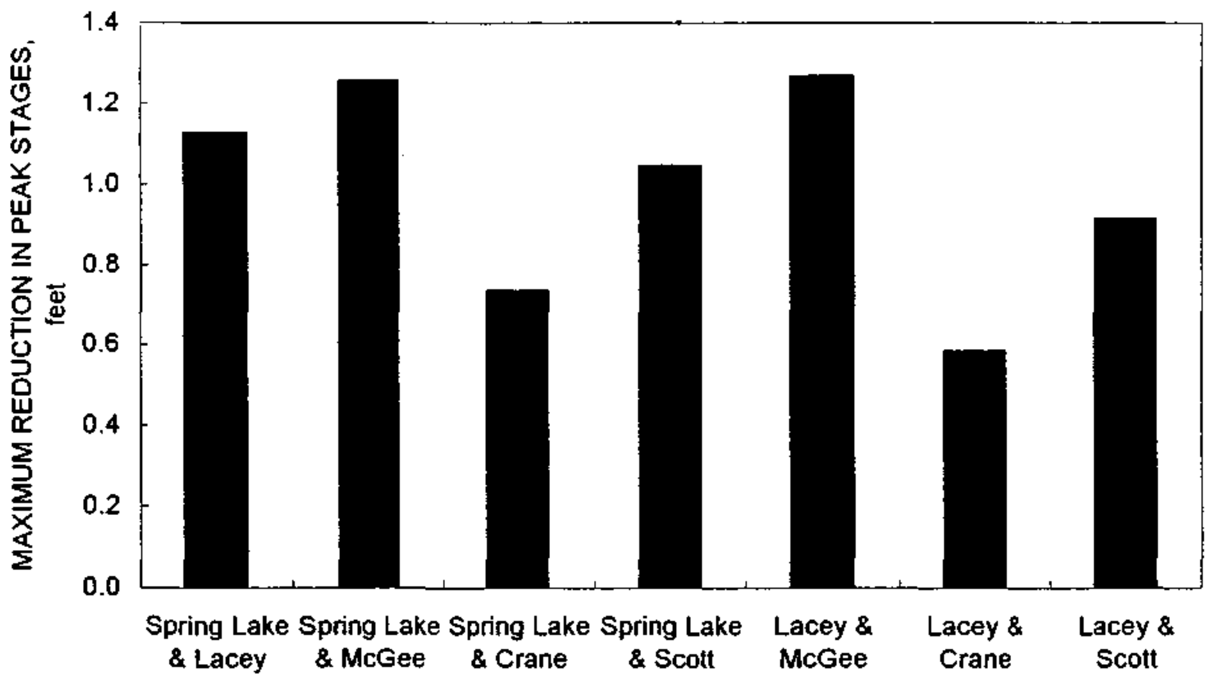

Figure 13. Changes in water surface elevations with and without managed storage option for selected combinations of individual levee and drainage districts

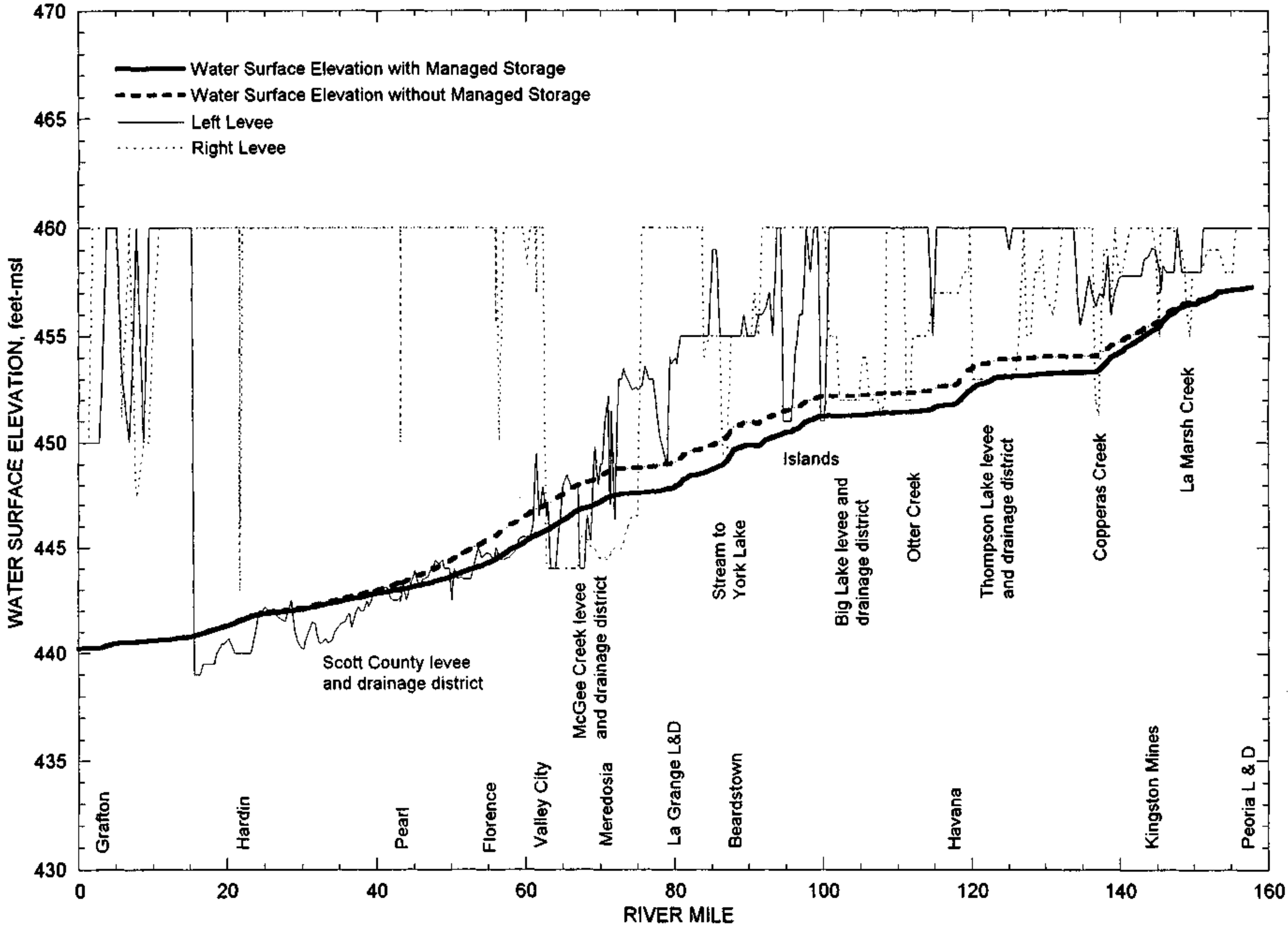

Figure 14. Water surface elevation profiles (100-year) with and without managed flood storage at Lacey (6-foot depth) and McGee (4-foot depth) Levee and Drainage Districts

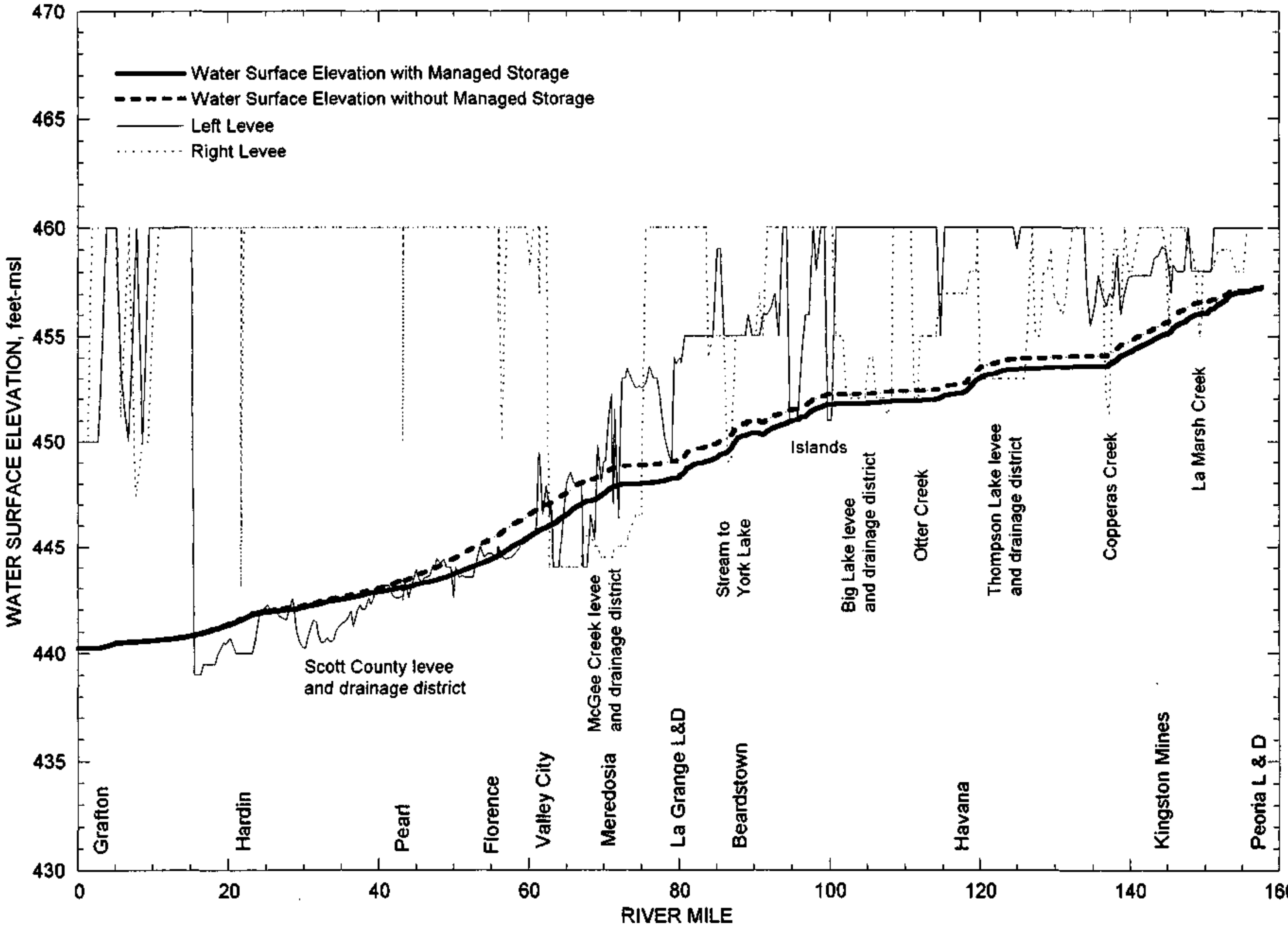

Figure 15. P6ak stage profiles (100-year) with and without managed flood storage at Spring Lake (6 foot depth) and Scott County (4-foot depth) Levee and Drainage Districts

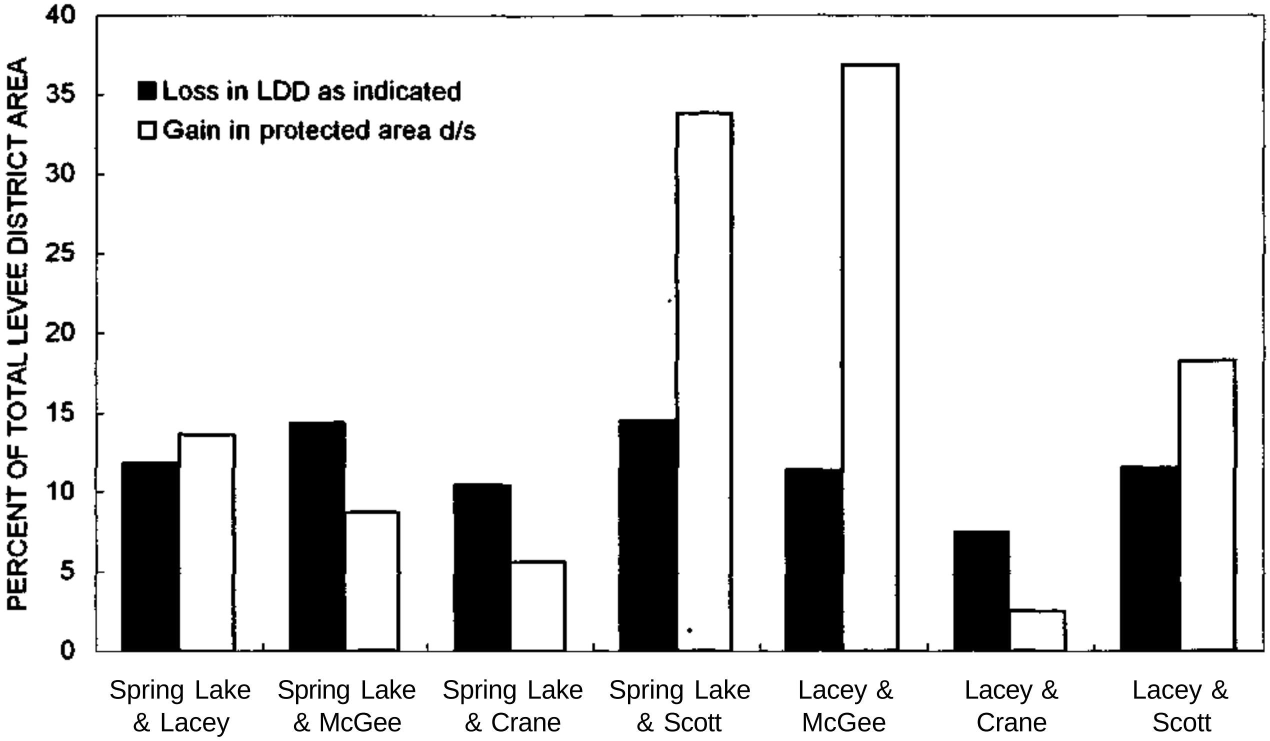

Figure 16. Percent area of managed storage levee and drainage district or LDD (LOSS) and additional LDD areas (GAIN) protected against a 100-year flood

Table 5. Maximum Reduction in Water Surface Elevations (feet) With Managed Storage for Individual Levee and Drainage Districts

	4-foot depth		*6-foot depth*	
Levee/Drainage District	*WSE reduction*	*River mile*	*WSE reduction*	*River mile*
Pekin & La Marsh			0.01	75.50
Spring Lake	0.46	147.20	0.60	147.20
Banner Special			0.06	87.50
East Liverpool	0.24	131.70	0.13	76.50
Liverpool	0.43	133.20	0.17	71.32
Thompson Lake	0.44	124.90	0.31	67.20
Lacey	0.39	119.40	0.80	119.40
Big Lake	0.21	76.50	0.18	70.00
Kelly Lake	0.08	102.20	0.08	120.75
Coal Creek	0.05	91.20	0.33	91.20
Crane Creek	0.33	85.00	0.45	71.32
Little Creek	0.14	66.00	0.12	65.50
McGee Creek	0.90	70.20	0.72	66.00
Mauvaise Terre	0.17	66.00	0.16	66.00
Valley City	0.19	66.00	0.17	66.61
Scott County	0.75	55.50	0.56	44.50
Big Swan	0.56	44.50	0.51	38.70
Hillview			0.72	41.80

Table 6. Maximum Reduction in Water Surface Elevations (feet) With Managed Storage for Combined Levee and Drainage Districts

LDD #1		*LDD #2*			
	Depth of opening (feet)		*Depth of opening (feet)*	*WSE reduction (feet)*	*River mile*
Spring Lake	*6*	Lacey	*6*	1.13	123.40
Spring Lake	*6*	McGee	4	1.26	71.44
Spring Lake	*6*	Crane Creek	6	0.74	70.80
Spring Lake	*6*	Scott County	4	1.04	61.30
Lacey	*6*	McGee	4	1.27	70.80
Lacey	6	Crane Creek	6	0.58	70.80
Lacey	6	Scott County	4	0.91	63.30

100-year flood up to RM 30 when the Lacey and McGee Creek LDDs are converted to flood storage areas. The reach further downstream of RM 30 is greatly affected by the Mississippi backwaters, and the Keach, Eldred-Spanky, and Nutwood levees in this reach all will have to be raised to safeguard them against the 100-year event. Figure 13 shows that the Spring Lake-Scott County LDDs will not provide sufficient protection for some of the levees in the La Grange Pool, but the protection in the Alton Pool will be comparable to that provided in the Lacey-McGee Creek storage areas.

Selection of Candidate Levees

Table 7 and Figure 16 show the effectiveness and benefits of some of the combined flood storage LDDs. Between Thompson Lake and Hartwell, the conversion of the Lacey-McGee Creek LDDs will provide 100-year flood protection for an additional 36.9 percent of downstream LDD areas. The Spring Lake-Scott County storage areas will also provide comparable protection for an additional 33.8 percent of downstream LDDs. However, the area lost to managed storage is 14.4 percent for the Spring Lake-Scott County combination , and 11.4 percent is lost for the Lacey-McGee Creek combination. These results indicate that the Lacey-McGee Creek combination will be the most suitable for managed flood storage conversion on the Lower Illinois River to provide additional protection against a 100-year flood for downstream LDDs. However, with the current level of protection for the proposed managed storage LDDs, the Lacey-McGee Creek storage areas currently have a higher level of protection against a 100-year flood than the Spring Lake-Scott County combination. These observations indicate that there is a trade-off between the physical conditions of the candidate levees for the managed storage option and the benefits of providing protection against a 100-year flood for additional downstream LDDs.

Table 7. Protected Levee and Drainage Districts (LDDs) with Combined Managed Storage and Corresponding Acres of LDDs Receiving Additional Protection for the 100-year Flood

		LDDs initially unprotected against 100-year flood											
Combined managed storage	*Overtopping return period LLD#1-LLD#2 (year)*	*Thompson Lake*	*Big Lake*	*Kelly Lake*	*Crane Creek*	*Scott County*	*Big Swan*	*Hillview*	*Hartwell*	*Combined storage areas (acres)*	*Storage area as % of Peoria-Hartwell area*	*Additional area of protected LDDs (acres)*	*Additional protected area as % of Peoria-Hartwell area*
LL-Mcg	500-25	P	P	P	P	P	P	P	P	20,200	13.74	65,262	44.39
LL-Crn	500-500		P	P						13,217	8.99	4,446	3.02
LL-Sct	500-50		P	P			P	P		20,500	13.94	32,346	22.00
SL-Crn	250-500	P	P	P						18,517	12.59	9,944	6.76
SL-Sct	250-50	P	P	P		P	P	P	P	25,800	17.55	59,845	40.70
SL-Mcg	250-25	P	P	P	P					25,500	17.34	15,361	10.45
SL-LL	250-500	P	P	P			P			20,900	14.22	24,144	16.42

Notes:

Existing protected LDD areas for the 100-year flood = 57,292 acres
Total LDD area between Peoria L&D and Hartwell = 147,024 acres
Total LDD area below Hartwell to Grafton = 30,100 acres
Combined LDD area below Peoria L&D to Grafton = 177,124 acres
P = Protected LDD
Crn = Crane Creek LDD
LL = Lacey, Langellier, W. Matanzas & Kerton Valley LDDs
Mcg = McGee Creek LDD
Sct = Scott County LDD
SL = Spring Lake LDD

POTENTIAL APPLICATIONS AND RECOMMENDATIONS FOR FUTURE STUDY

With the developed unsteady flow model for the Lower Illinois River, one can investigate various scenarios based on management requirements. Although the current model cannot handle sediment transport, the consequence of sedimentation on floodplain on flood peaks can be determined. One series of simulation runs examined the impact on flood stages due to sediment accumulation in the Big Swan LDD.

Effect of Floodplain Sedimentation along the Big Swan Levee

The effect of floodplain sedimentation on flood stages along the Big Swan levee waterfront was evaluated by raising the floodplain elevations in the cross sections 1 to 2 feet and running the unsteady flow model to determine changes in the flood elevations. No changes were observed in the WSE along the Big Swan Levee waterfront. Changes in the WSE profiles for increases of 1 to 2 feet in floodplain elevations, shown in Table 8, were less than 0.01 foot.

Recommendations for Future Study

Current conclusions are applicable to the 100-year flood scenario with downstream boundary specified as the 100-year stage hydrograph. To assess a range of impacts and benefits for the management purposes, other useful combinations can be tested. There are also the overall issues about the reliability of computer simulations and the risks involved in the flood protections. The following list of selected topics could be investigated by taking advantage of the developed model.

Conduct Sensitivity Analysis on Up- and Downstream Boundary Conditions

Further analyses are necessary using different combinations of up- and downstream boundary conditions for concerns such as developing guidelines for flood fighting, or evaluating impacts due to Illinois River floods only. These objective-oriented investigations can be conducted with the current model.

Examine Hypothetical Scenarios on No-Overtopping-of-Levees or Prelevee-Construction Conditions

Although hypothetical, the model can be used for such purposes. Often questions have been raised about raising the levee heights or wanting to know WSE profiles and channel conveyance without the levee conditions. The corresponding WSE profiles and conveyance can be evaluated by modifying the corresponding geometry in cross sections that describe the levees. The results should serve as science-based information for management practices.

Expand Modeling Efforts To Include the Sangaman River from the Junction to Oakford, Approximately 25.7 Mile Upstream or Beyond from the Junction

The Sangamon River is the largest tributary to the Lower Illinois River. It contributes a significant amount of flow (and probably sediment) to the Illinois River. Current modeling efforts did not evaluate impacts from the modifications of the Sangamon River LDDs or backwater effects on the Sangamon River LDDs.

Perform Reliability Analysis on the Lower Illinois River UNET Model

All the analyses suggested involve uncertainties, thus the reliability of the model results needs to be assessed because the parameters used in the model and inputs contain probability distributions. The importance of reliability analysis has been gradually realized by the federal agencies and has been a required component in large-scaled projects. Although procedures have not been clearly defined for modeling work, it is recommended that researchers take on this direction and lay out foundations for future analysis.

Perform Risk Analysis on the Flood Protection in the Lower Illinois River

Risk analysis of the existing configuration and crown height of levees in the Lower Illinois River should be undertaken for floods with high recurrence intervals. With a complete understanding of the risk analysis results, management decisions on optimizing goals such as public safety or construction or repair costs can be made according to specific risk levels.

Table 8. Change in Water Surface Elevation (feet) Due to Sediment Deposition on Big Swan Overbank Areas

Time	*Downstream (RM 50.05)*					*Upstream (RM 56.0)*				
(hour)	*Initial*	*1-ft*	*Diff.*	*2-ft*	*Diff.*	*Initial*	*1-ft*	*Diff.*	*2-ft*	*Dijf.*
0	431.713	431.711	0.002	431.710	0.003	432.619	432.621	-0.002	432.623	-0.004
6	431.716	431.714	0.002	431.712	0.004	432.616	432.618	-0.002	432.620	-0.004
12	431.716	431.715	0.001	431.713	0.003	432.611	432.613	-0.002	432.615	-0.004
18	431.717	431.715	0.002	431.713	0.004	432.606	432.608	-0.002	432.609	-0.003
24	431.719	431.717	0.002	431.716	0.003	432.602	432.604	-0.002	432.605	-0.003
30	431.723	431.722	0.001	431.720	0.003	432.599	432.601	-0.002	432.603	-0.004
36	431.732	431.73	0.002	431.728	0.004	432.6	432.602	-0.002	432.603	-0.003
42	431.748	431.746	0.002	431.745	0.003	432.608	432.609	-0.001	432.611	-0.003
48	431.774	431.773	0.001	431.771	0.003	432.625	432.626	-0.001	432.628	-0.003
54	431.814	431.812	0.002	431.811	0.004	432.653	432.655	-0.002	432.657	-0.004
60	431.869	431.867	0.002	431.866	0.003	432.696	432.697	-0.001	432.699	-0.003
66	431.942	431.94	0.002	431.939	0.003	432.753	432.754	-0.001	432.756	-0.003
72	432.034	432.032	0.002	432.031	0.003	432.825	432.827	-0.002	432.829	-0.004
78	432.152	432.151	0.001	432.150	0.002	432.918	432.919	-0.001	432.921	-0.003
84	432.303	432.302	0.001	432.301	0.002	433.033	433.035	-0.002	433.036	-0.003
90	432.494	432.493	0.001	432.492	0.002	433.176	433.177	-0.001	433.179	-0.003
96	432.738	432.737	0.001	432.736	0.002	433.356	433.357	-0.001	433.359	-0.003
102	433.034	433.033	0.001	433.033	0.001	433.581	433.583	-0.002	433.584	-0.003
108	433.368	433.367	0.001	433.367	0.001	433.851	433.852	-0.001	433.854	-0.002
114	433.738	433.737	0.001	433.737	0.001	434.158	434.159	-0.001	434.160	-0.002
120	434.167	434.167	0	434.167	0.000	434.515	434.516	-0.001	434.517	-0.002
126	434.63	434.63	0	434.630	0.000	434.914	434.915	-0.001	434.916	-0.002
132	435.066	435.066	0	435.066	0.000	435.316	435.317	-0.001	435.318	-0.002
138	435.504	435.504	0	435.504	0.000	435.72	435.721	-0.001	435.721	-0.001
144	436	436	0	436.000	0.000	436.16	436.16	0	436.161	-0.001
150	436.454	436.454	0	436.454	0.000	436.596	436.597	-0.001	436.597	-0.001

Notes: Diff. - difference
RM - river mile

SUMMARY

In the first part of this century, about 36 levees were constructed along the Lower Illinois River. The levees removed about 180,000 acres of land from the floodplain, leading to increased flood elevations, more concentrated flows, and habitat impacts for many aquatic species. Because of the gradual increase in flood stages with continued levee construction, the flood protection for already completed levees decreased. The common response to protect an at-risk levee (overtopped during a 100-year or lesser flood) is to raise the levee, but the benefit-to-cost ratio, about 0.1, is far less than the generally acceptable standard at 1.0. Even if the levee is raised, the additional cost of pumping to keep the water table low in the levee and drainage district and the consequent reduction in crop yield make farming in the area behind the levee less profitable. The proposal put forward in this study is to convert a few selected levees with marginally profitable farmlands to managed flood storage areas so they can provide both flood storage and wetland or conservation functions, while also providing greater protection against flooding to agricultural lands served by other levees.

Most of the LDDs along the Illinois River are in the Peoria-Grafton reach. Extreme flood stages occurring in this reach of the river can be lowered by converting the areas behind a few levees to managed flood storage areas. A suitable, limited section of a selected levee would be lowered to a predetermined elevation so that floodwaters can flow into the area behind the levee for temporary storage.

Historical floods were analyzed to determine flood stages and frequencies at which overtopping of levees in the Peoria-Grafton section of the Illinois River occurs. An unsteady flow model was applied to simulate WSE profiles for existing conditions and for individual and various combinations of pairs of levee districts converted to managed storage areas. Various dimensions of the lateral inflow section along the top of the levees were simulated to determine the opening size that will provide maximum reduction in peak stages and thus provide maximum protection against design floods for other levees. A width of 1000 feet (along the levee) and a depth of 4 or 6 feet were the optimum size for the inflow section. The simulation results indicate significant reductions in peak stages when six of the levee districts are converted to managed flood storage areas. With combinations of selected levee districts to managed storage areas, the area of levee districts that will have 100-year flood protection increased by as much as 65,262 acres for levees upstream of RM 43.2. Levees downstream of this section will have to be raised by 1 - 3 feet to protect them against a 100-year flood because the reach below RM 43.2 is usually affected by the Upper Mississippi River backwaters during major flood events. With Lacey-McGee Creek LDD combined managed storage, the additional area of protected LDDs is 65,262 acres. Added to LDDs already safe for a 100-year flood (57,292 acres), the total LDD area protected between Peoria L&D and Hartwell amounts to 122,554 acres, with a combined managed storage area of 20,200 acres. The total area in this reach is 147,024 acres. The managed flood storage area also will serve wetland and conservation functions and create sizable new wetland areas.

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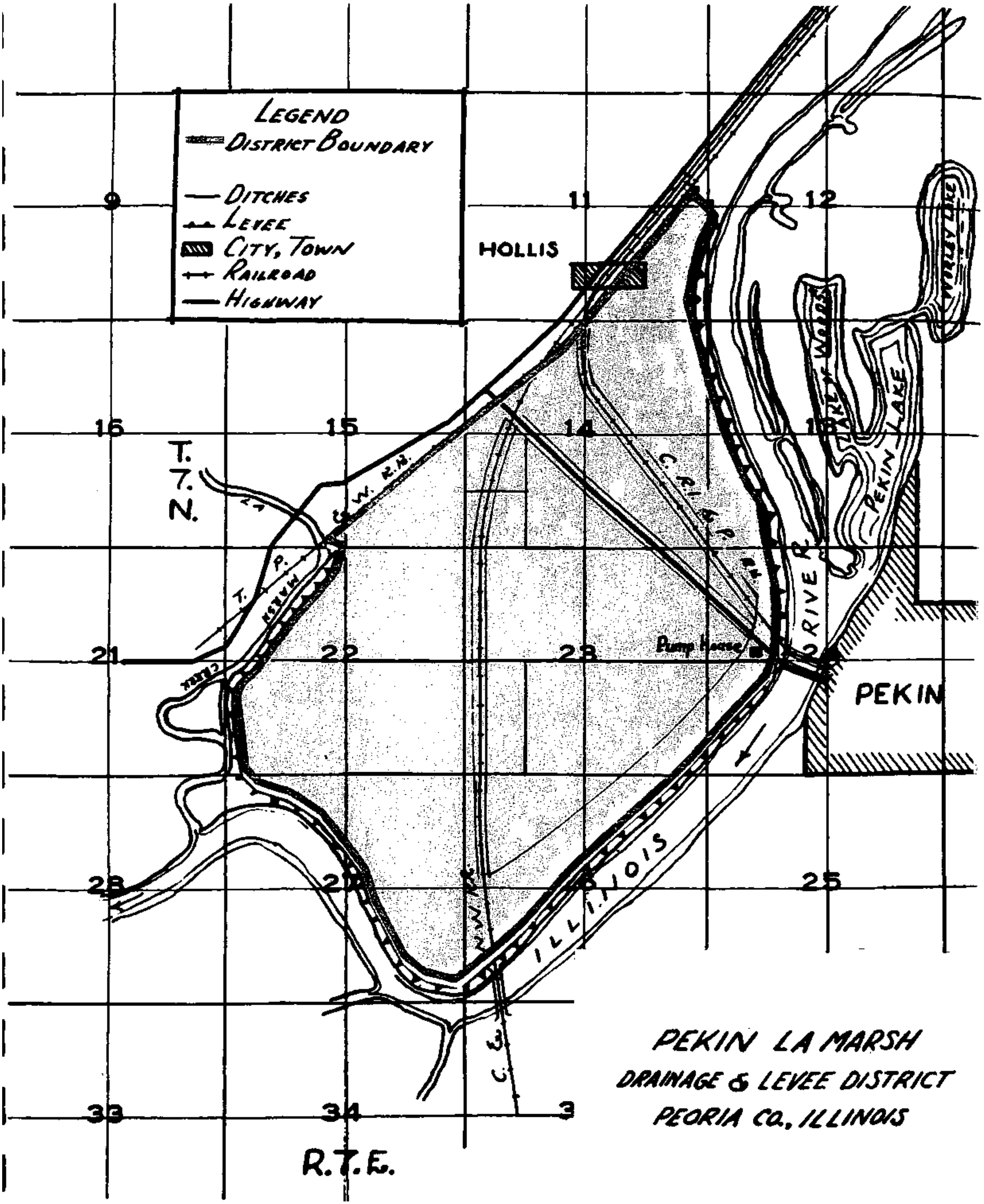
APPENDIX I.

LOCATIONS AND PLAN VIEWS OF THE LEVEE AND DRAINAGE DISTRICTS ALONG THE ILLINOIS RIVER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DITCHES
LEVEE
CITY, TOWN
RAILROAD
HIGHWAY
HOLLIS
PEKIN
T.
7.
N.
R.7.E.
9
11
12
16
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13
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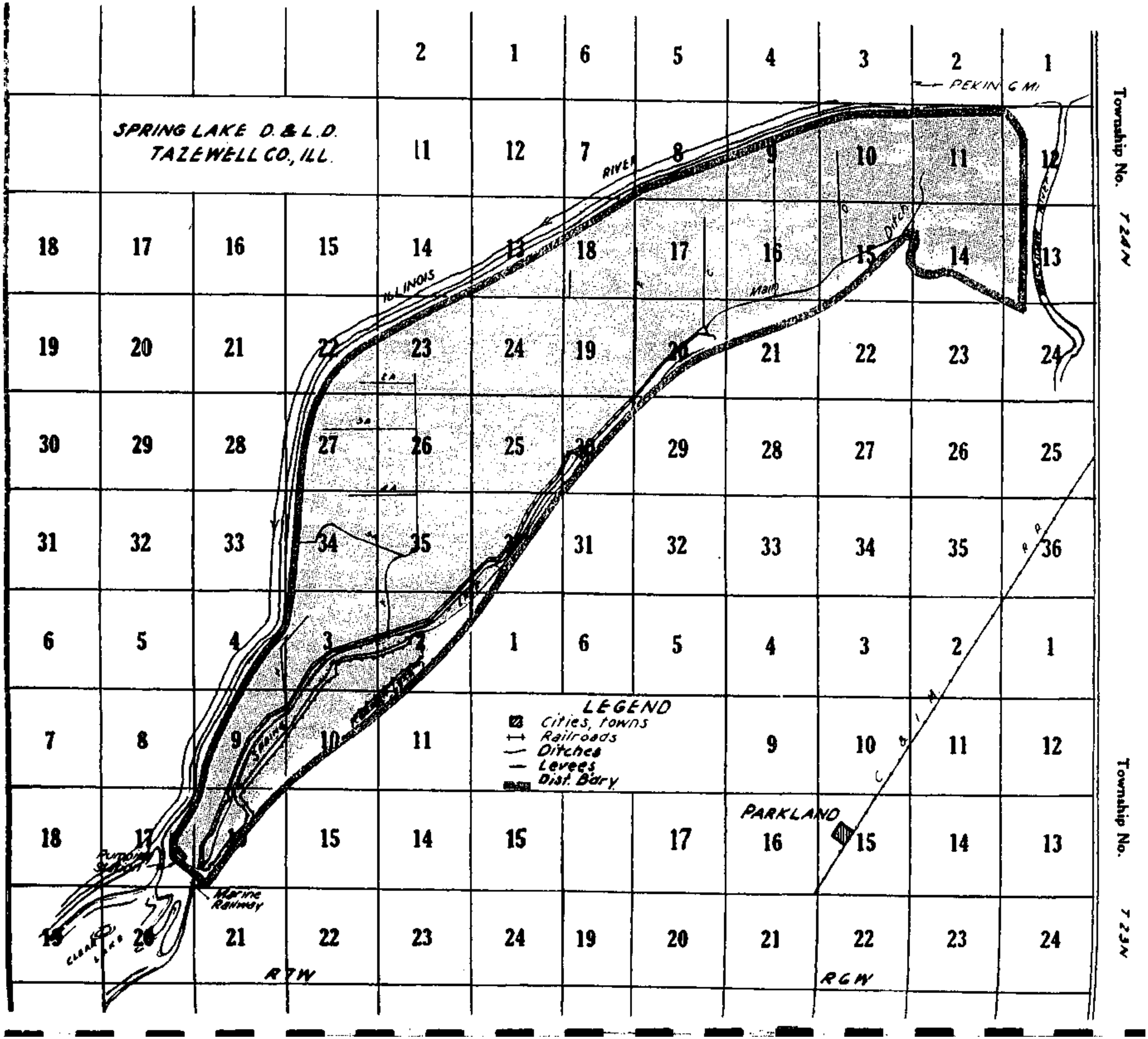
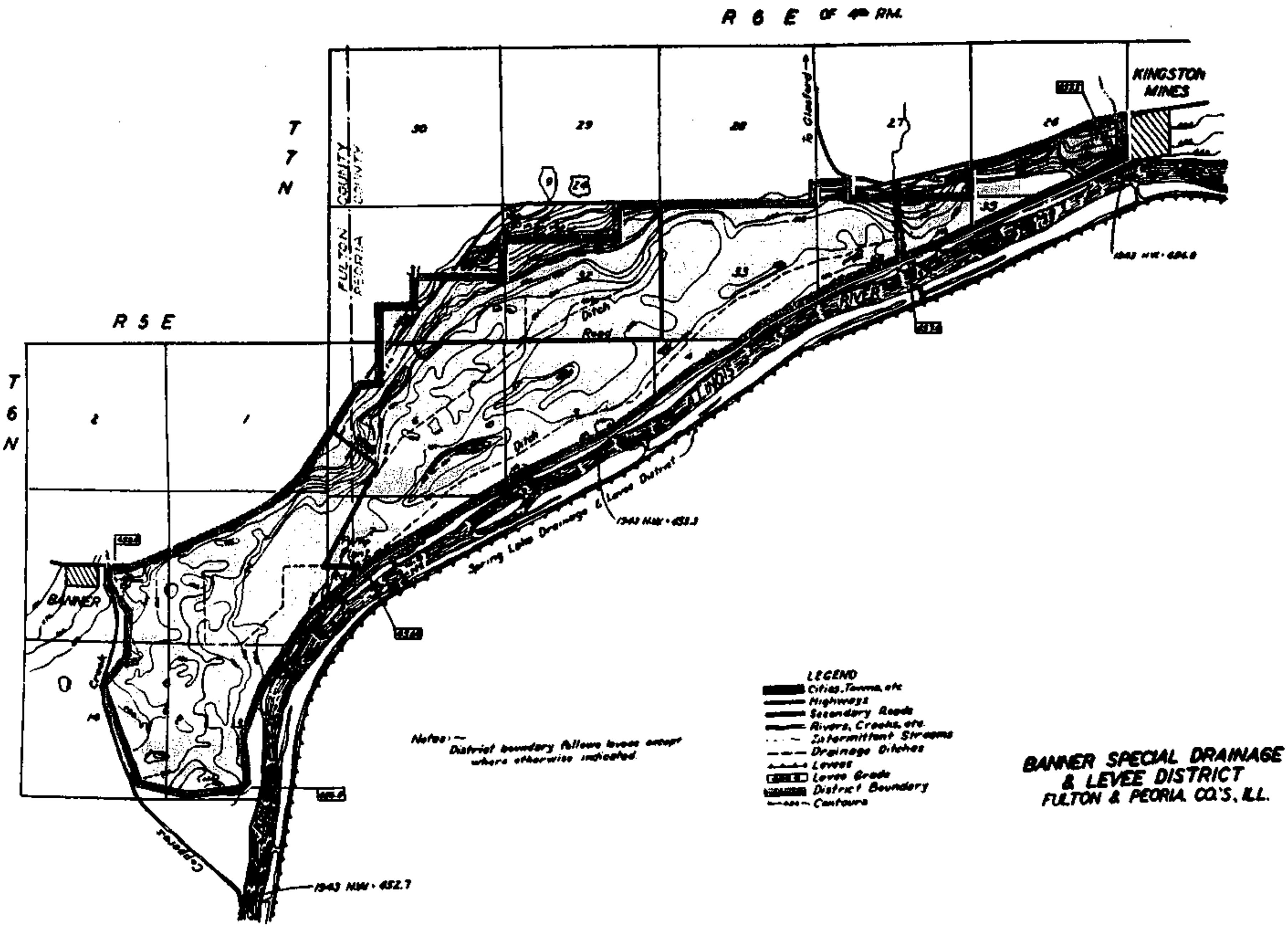
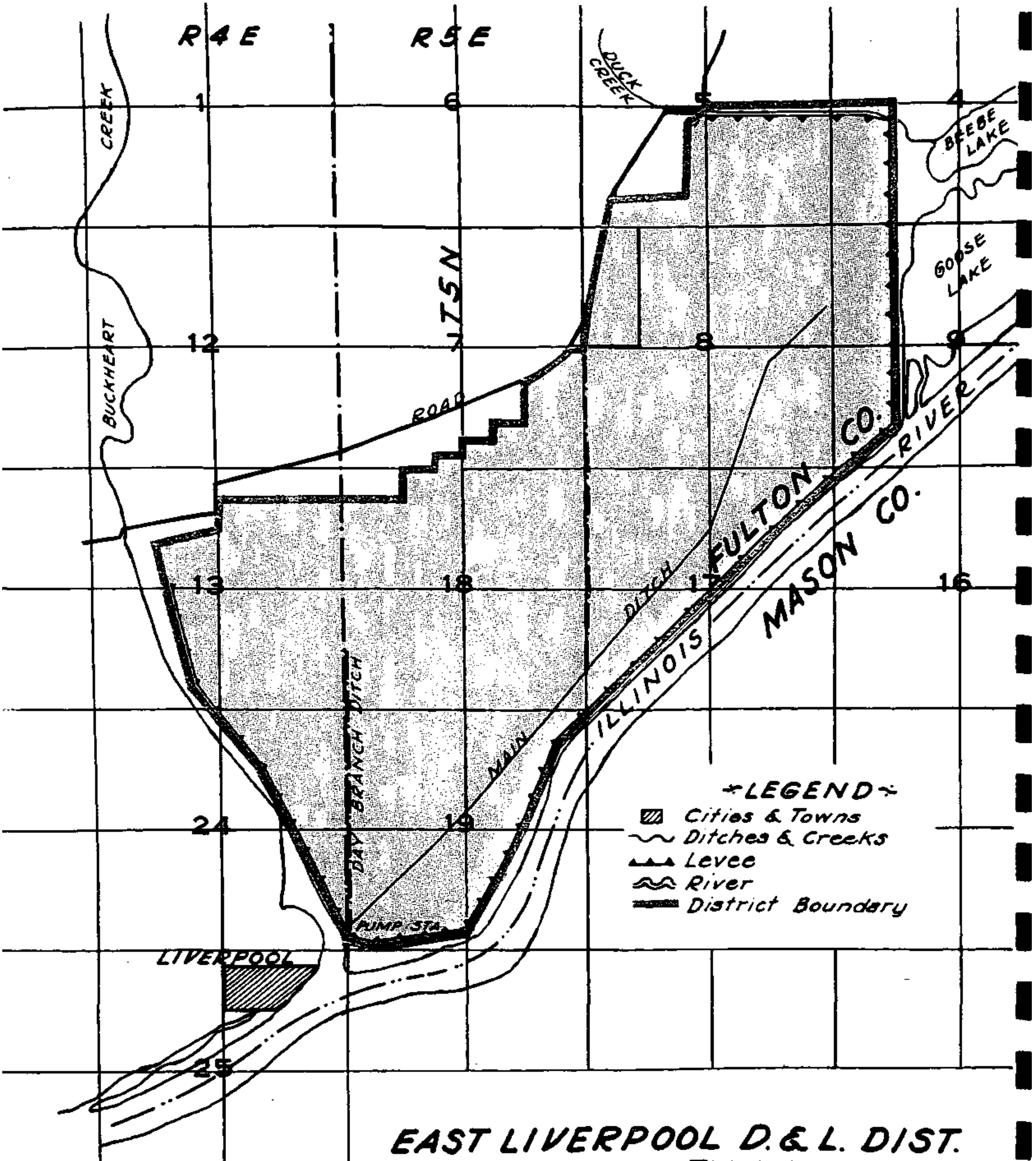
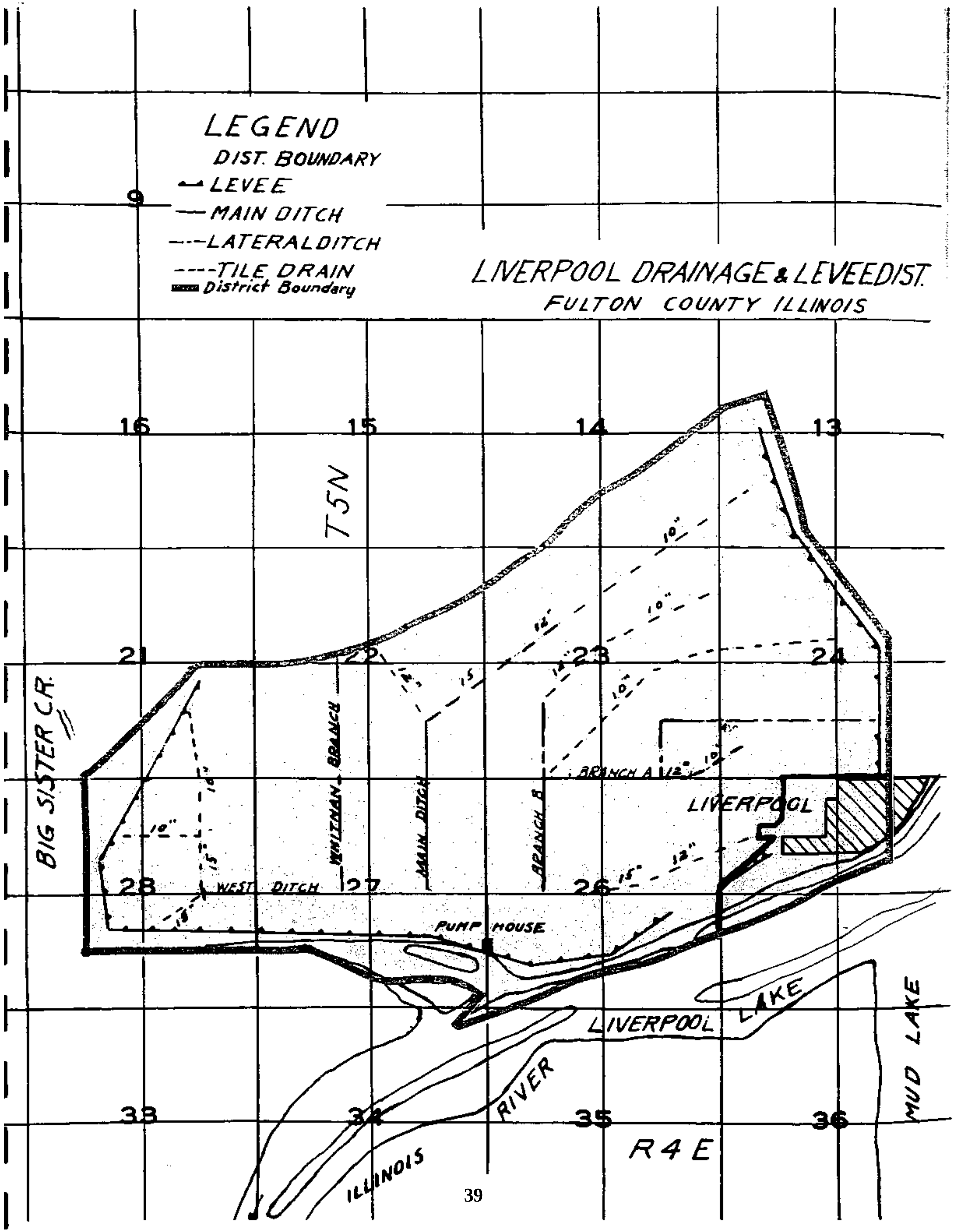
25
28
27
33
34
3
Pump House
C. R. I. & P. RR.
RIVER
ILLINOIS
PEKIN LAKE
PEKIN LA MARSH
DRAINAGE & LEVEE DISTRICT
PEORIA CO., ILLINOIS

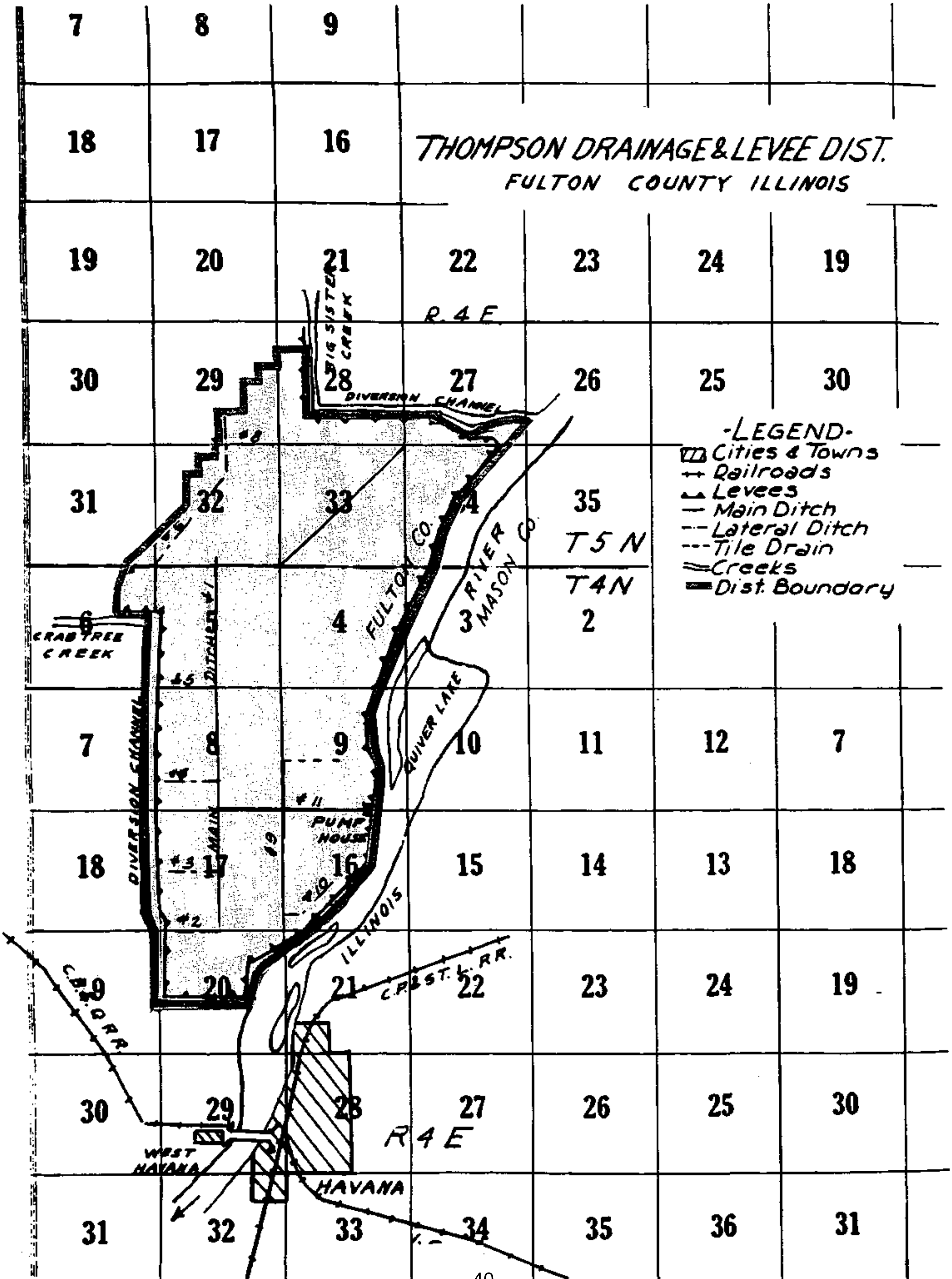

SPRING LAKE D. & L. D.
TAZEWELL CO., ILL.
PEKIN 6 MI
ILLINOIS
RIVER
Main
Ditch
LEGEND
Cities, towns
Railroads
Ditches
Levees
Dist. Bdry.
PARKLAND
Pumping Station
Marine Railway
CLEAR LAKE
R7W
R6W
Township No. T24N
Township No. T23N

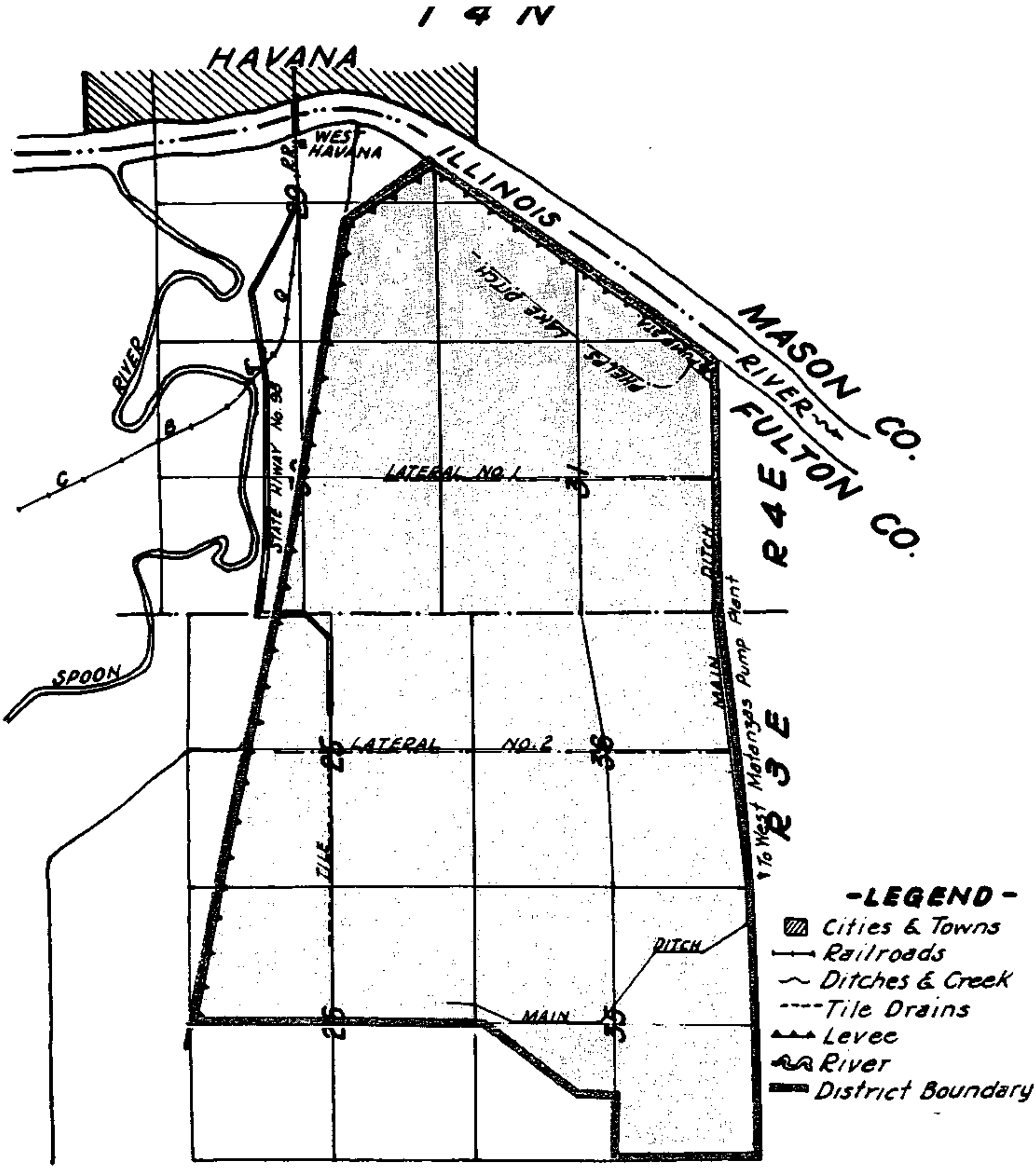

R 6 E OF 4th P.M.
R 5 E
T 7 N
T 6 N
FULTON COUNTY
PEORIA COUNTY
To Glasford
KINGSTON MINES
BANNER
30
29
28
27
26
32
33
35
2
1
Ditch
Road
ILLINOIS RIVER
Spring Lake Drainage & Levee District
1943 H.W. 452.7
Creek
Coppers
Notes: —
District boundary follows levees except where otherwise indicated.
LEGEND
Cities, Towns, etc.
Highways
Secondary Roads
Rivers, Creeks, etc.
Intermittent Streams
Drainage Ditches
Levees
Levee Grade
District Boundary
Contours
BANNER SPECIAL DRAINAGE & LEVEE DISTRICT
FULTON & PEORIA CO'S, ILL.



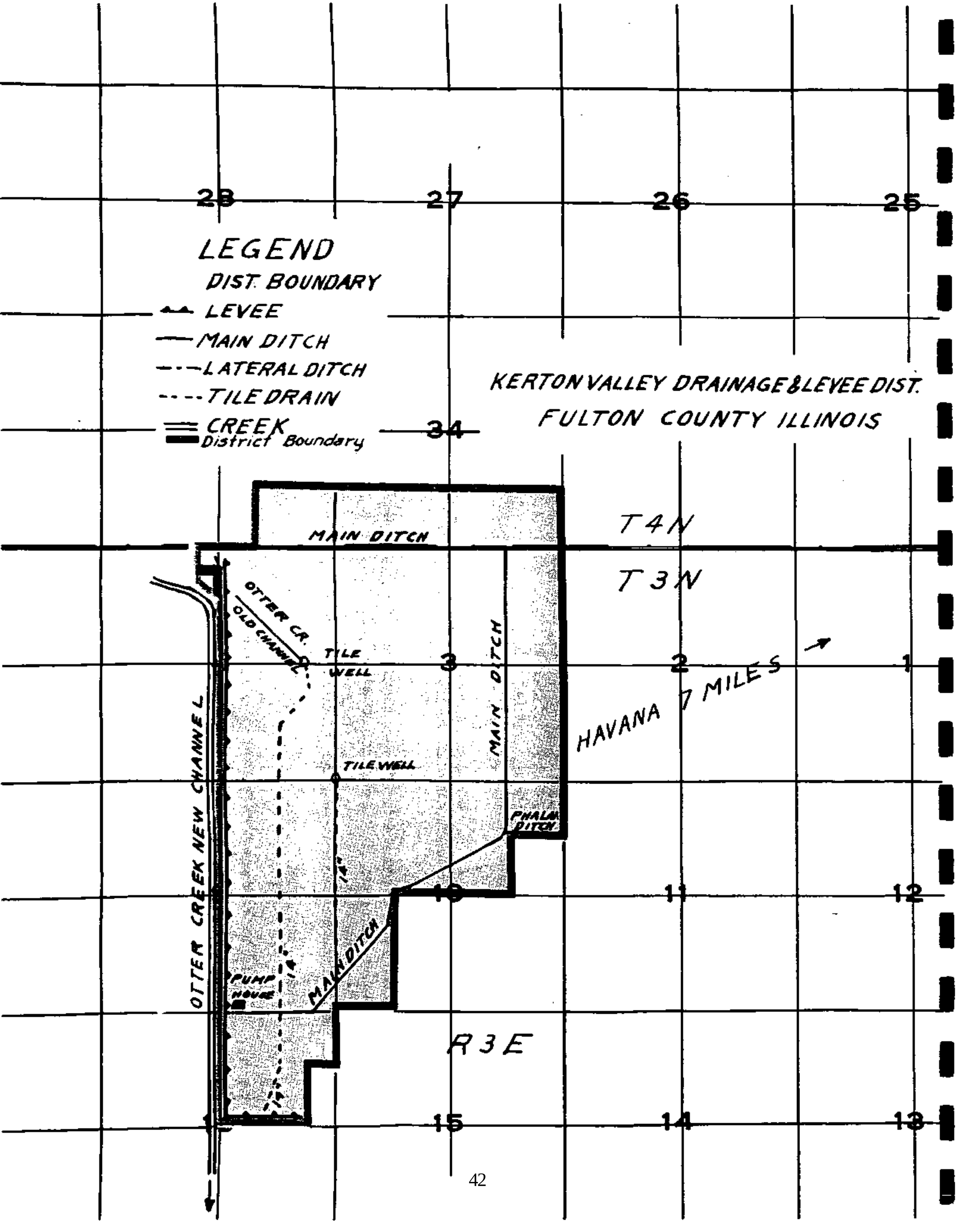
EAST LIVERPOOL D.& L. DIST.
FULTON COUNTY ILLINOIS

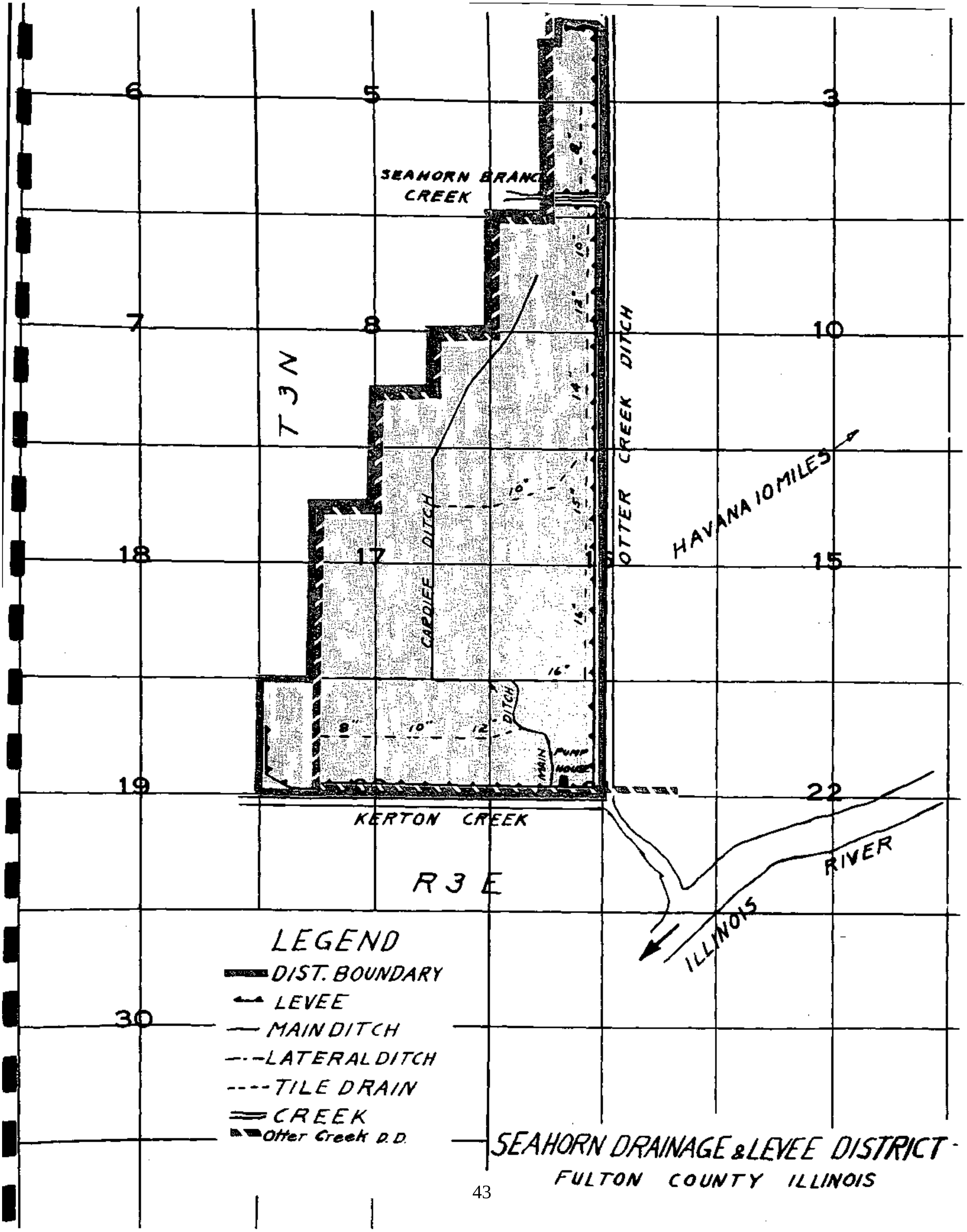

LEGEND
DIST. BOUNDARY
LEVEE
MAIN DITCH
LATERAL DITCH
TILE DRAIN
District Boundary
LIVERPOOL DRAINAGE & LEVEE DIST.
FULTON COUNTY ILLINOIS
T5N
R4E
BIG SISTER CR.
WHITMAN BRANCH
MAIN DITCH
BRANCH B
BRANCH A
WEST DITCH
PUMP HOUSE
LIVERPOOL
LIVERPOOL LAKE
MUD LAKE
ILLINOIS RIVER

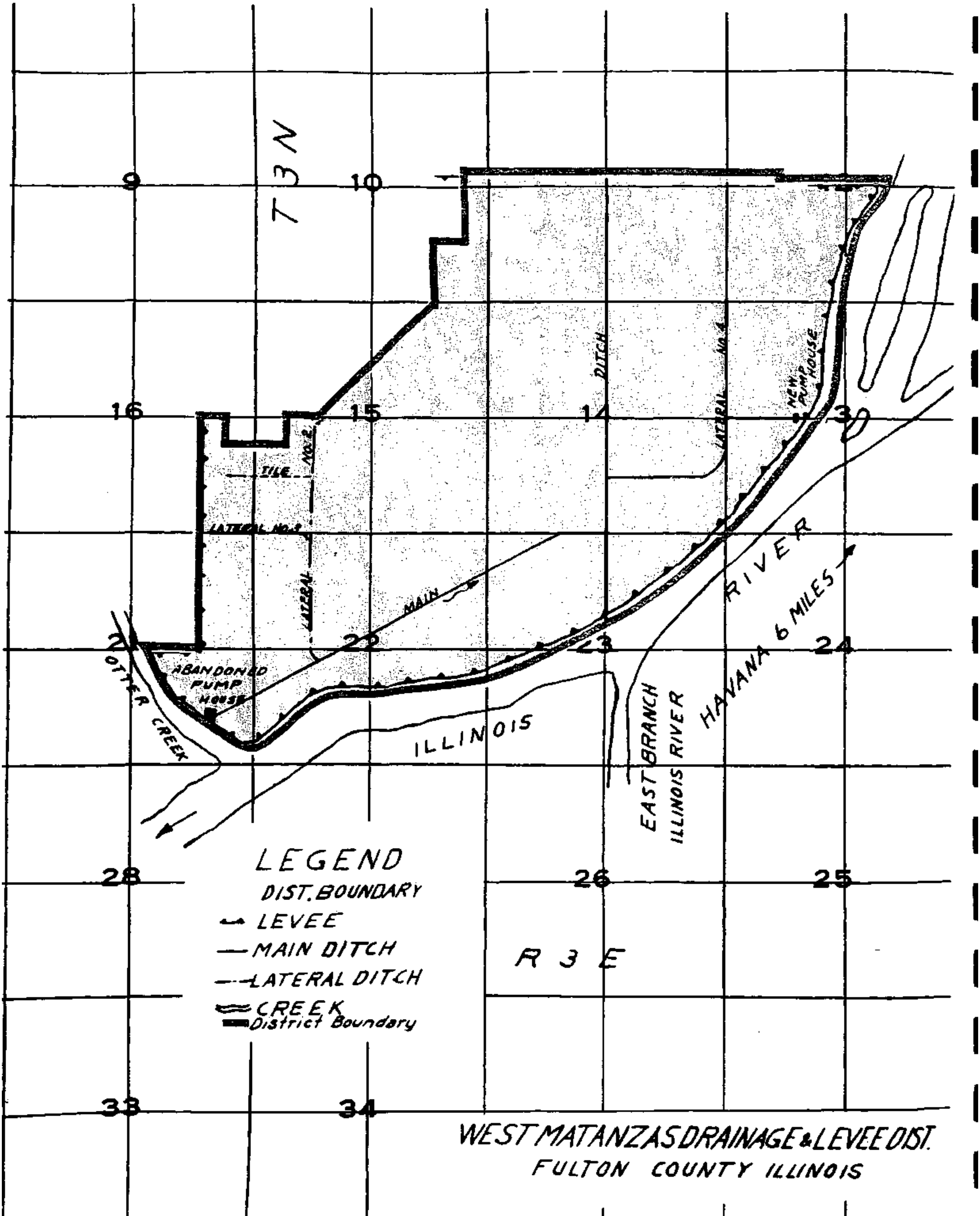

THOMPSON DRAINAGE & LEVEE DIST.
FULTON COUNTY ILLINOIS
-LEGEND-
Cities & Towns
Railroads
Levees
Main Ditch
Lateral Ditch
Tile Drain
Creeks
Dist. Boundary
R. 4 E
T 5 N
T 4 N
R 4 E
BIG SISTER CREEK
DIVERSION CHANNEL
CRAB TREE CREEK
DIVERSION CHANNEL
MAIN DITCHES #1
PUMP HOUSE
FULTON CO.
MASON CO.
RIVER
QUIVER LAKE
ILLINOIS
C.B.&Q.R.R.
C.P.&ST.L.R.R.
WEST HAVANA
HAVANA

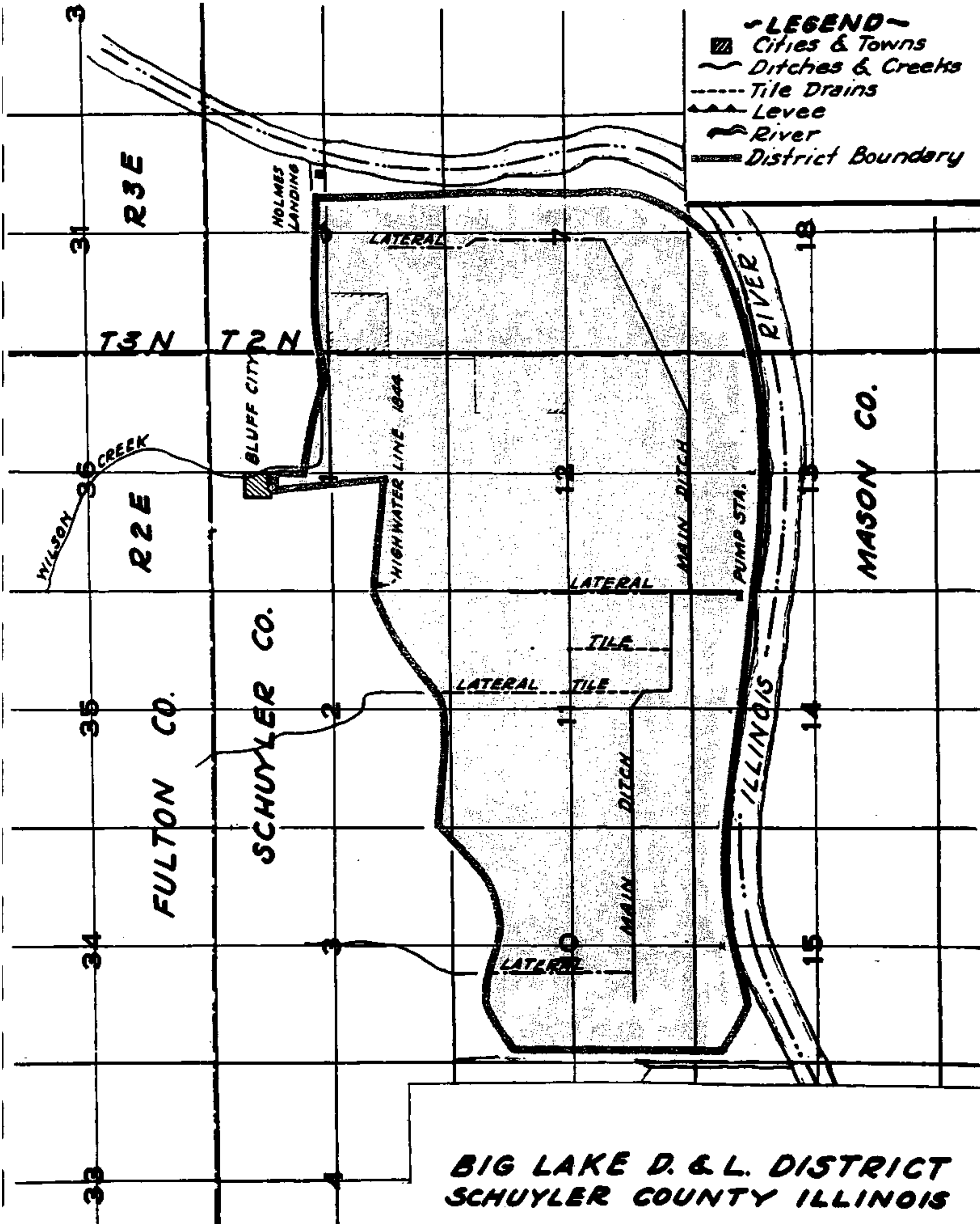


LACEY DRAINAGE & LEVEE DIST.
FULTON COUNTY ILLINOIS

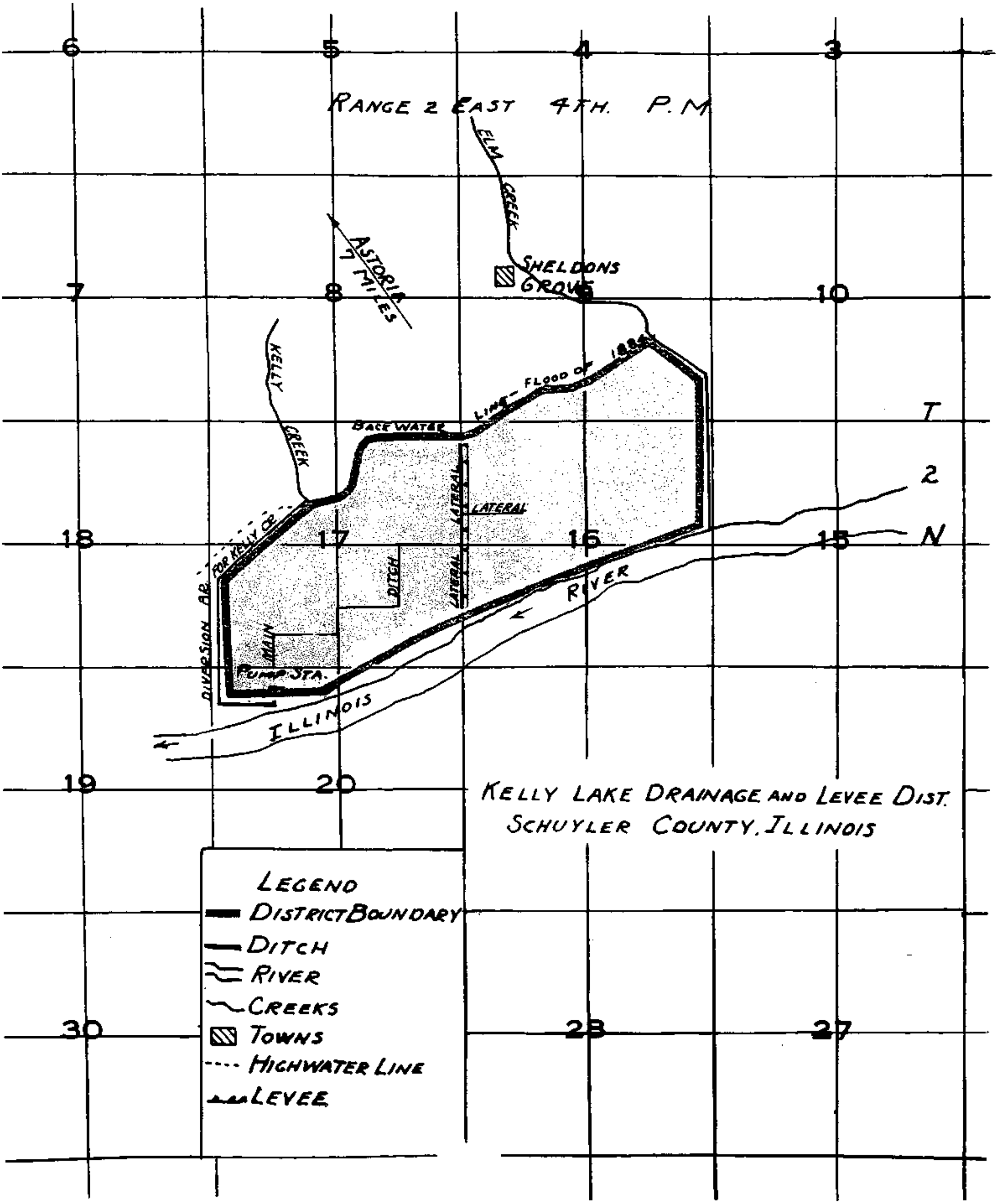

KERTON VALLEY DRAINAGE & LEVEE DIST.
FULTON COUNTY ILLINOIS
LEGEND
DIST. BOUNDARY
LEVEE
MAIN DITCH
LATERAL DITCH
TILE DRAIN
CREEK
District Boundary
28
27
26
25
34
3
2
1
10
11
12
15
14
13
T4N
T3N
R3E
MAIN DITCH
MAIN DITCH
MAIN DITCH
OTTER CR.
OLD CHANNEL
TILE WELL
TILE WELL
PHALAN DITCH
PUMP HOUSE
OTTER CREEK NEW CHANNEL
HAVANA 7 MILES
14"
14"


6
5
3
SEAHORN BRANCH CREEK
7
8
10
T 3 N
OTTER CREEK DITCH
HAVANA 10 MILES
18
17
16
15
CARDIEE DITCH
MAIN DITCH
PUMP HOUSE
19
22
KERTON CREEK
R 3 E
ILLINOIS RIVER
LEGEND
DIST. BOUNDARY
LEVEE
MAIN DITCH
LATERAL DITCH
TILE DRAIN
CREEK
Otter Creek D.D.
30
SEAHORN DRAINAGE & LEVEE DISTRICT
FULTON COUNTY ILLINOIS

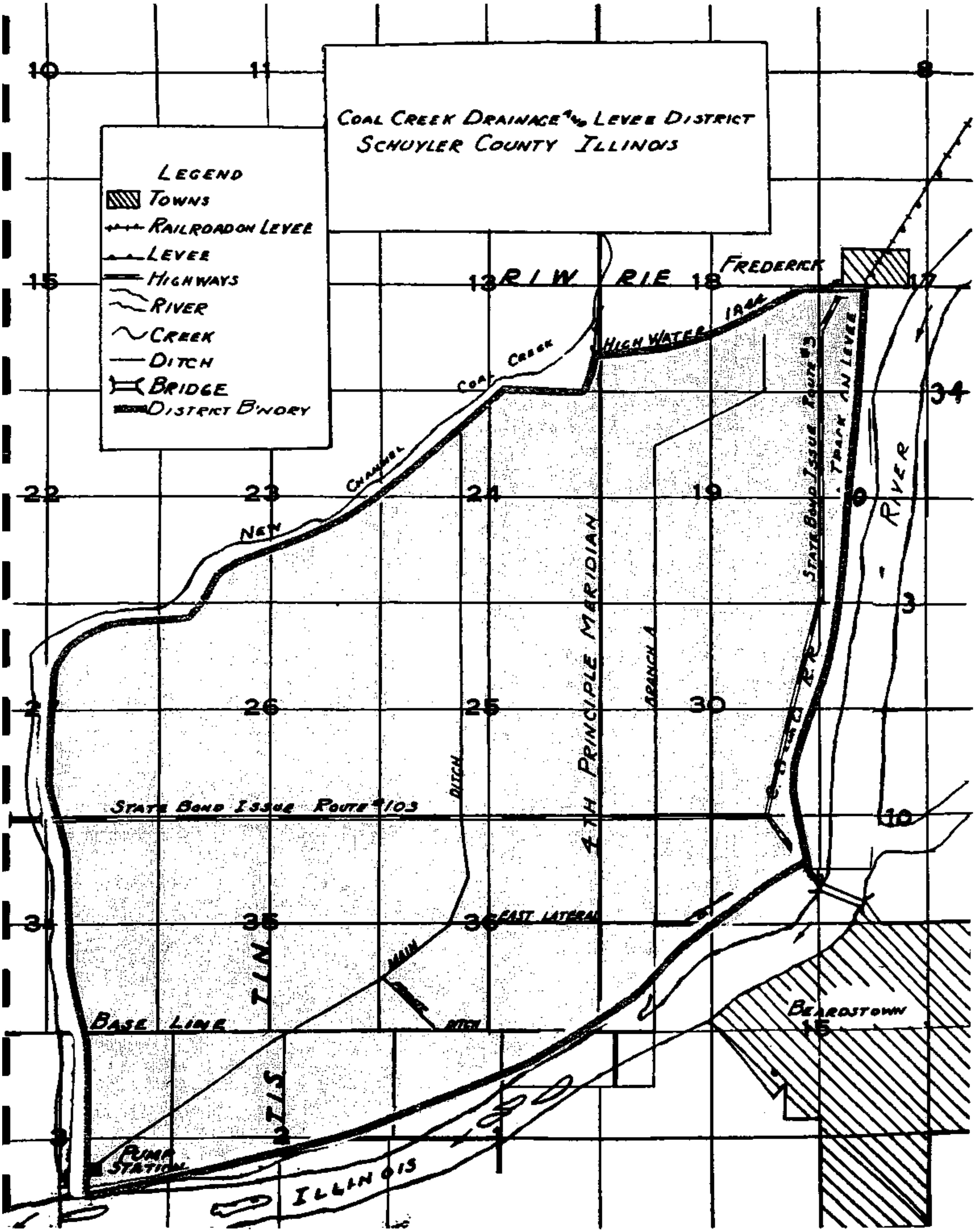

T 3 N
9
10
16
15
14
30
21
22
23
24
28
26
25
33
34
DITCH
LATERAL No.4
NEW PUMP HOUSE
TILE
LATERAL No.2
LATERAL No.1
LATERAL
MAIN
ABANDONED PUMP HOUSE
OTTER CREEK
ILLINOIS
RIVER
HAVANA 6 MILES
EAST BRANCH
ILLINOIS RIVER
LEGEND
DIST. BOUNDARY
LEVEE
MAIN DITCH
LATERAL DITCH
CREEK
District Boundary
R 3 E
WEST MATANZAS DRAINAGE & LEVEE DIST.
FULTON COUNTY ILLINOIS

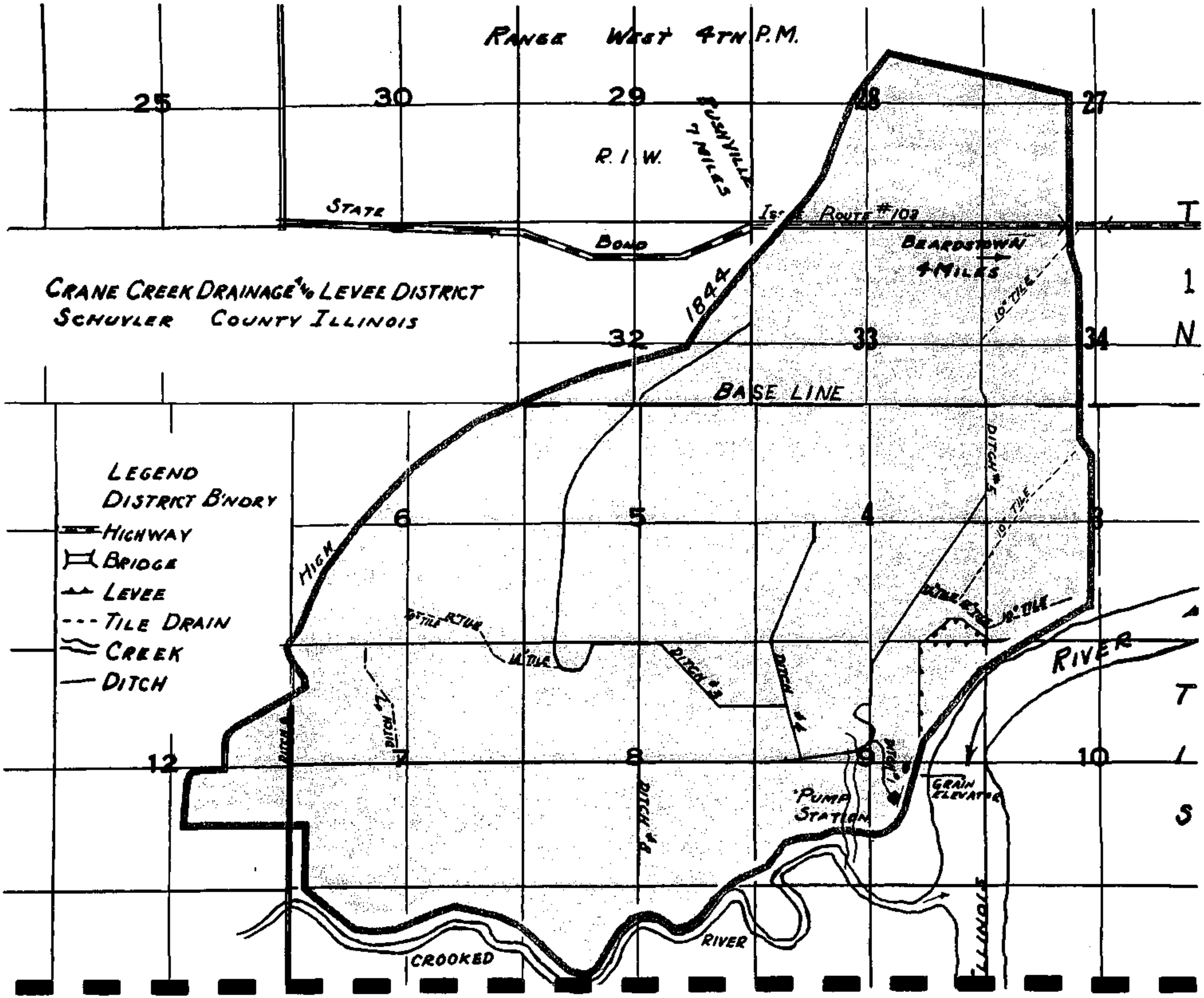


LEGEND
Cities & Towns
Ditches & Creeks
Tile Drains
Levee
River
District Boundary
R3E
R2E
T3N
T2N
HOLMES LANDING
BLUFF CITY
WILSON
CREEK
LATERAL
HIGHWATER LINE 1844
MAIN DITCH
PUMP STA.
TILE
LATERAL
TILE
LATERAL
ILLINOIS
RIVER
MASON CO.
SCHUYLER CO.
FULTON CO.
BIG LAKE D. & L. DISTRICT
SCHUYLER COUNTY ILLINOIS

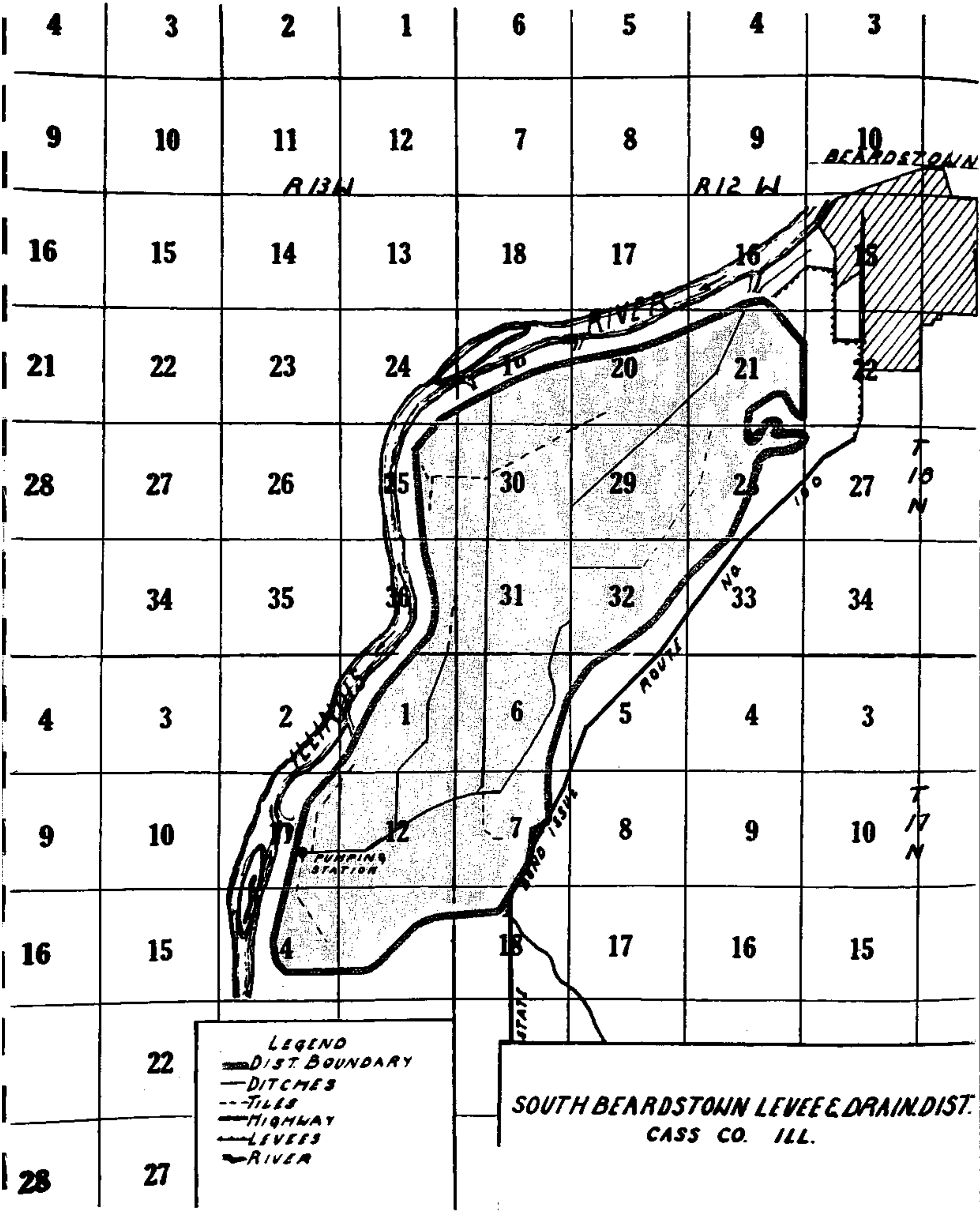


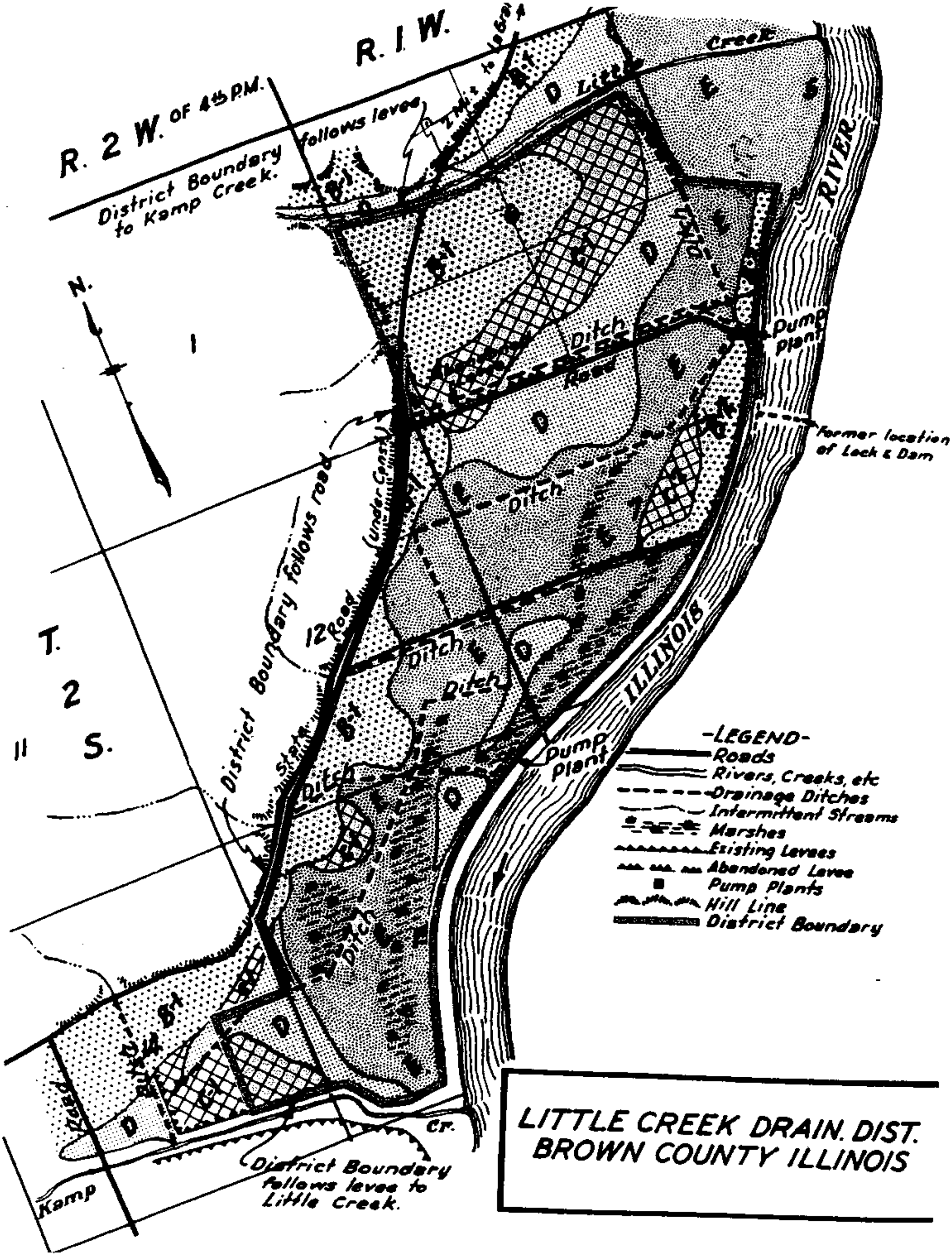
RANGE 2 EAST 4TH. P.M
ELM CREEK
ASTORIA 7 MILES
SHELDONS GROVE
KELLY CREEK
BACK WATER LINE - FLOOD OF 1884
LATERAL
LATERAL
LATERAL
DITCH
MAIN
PUMP STA.
DIVERSION DR. FOR KELLY CR.
ILLINOIS RIVER
T 2 N
6
5
4
3
7
8
10
18
17
16
15
19
20
30
28
27
KELLY LAKE DRAINAGE AND LEVEE DIST.
SCHUYLER COUNTY, ILLINOIS
LEGEND
DISTRICT BOUNDARY
DITCH
RIVER
CREEKS
TOWNS
HIGHWATER LINE
LEVEE

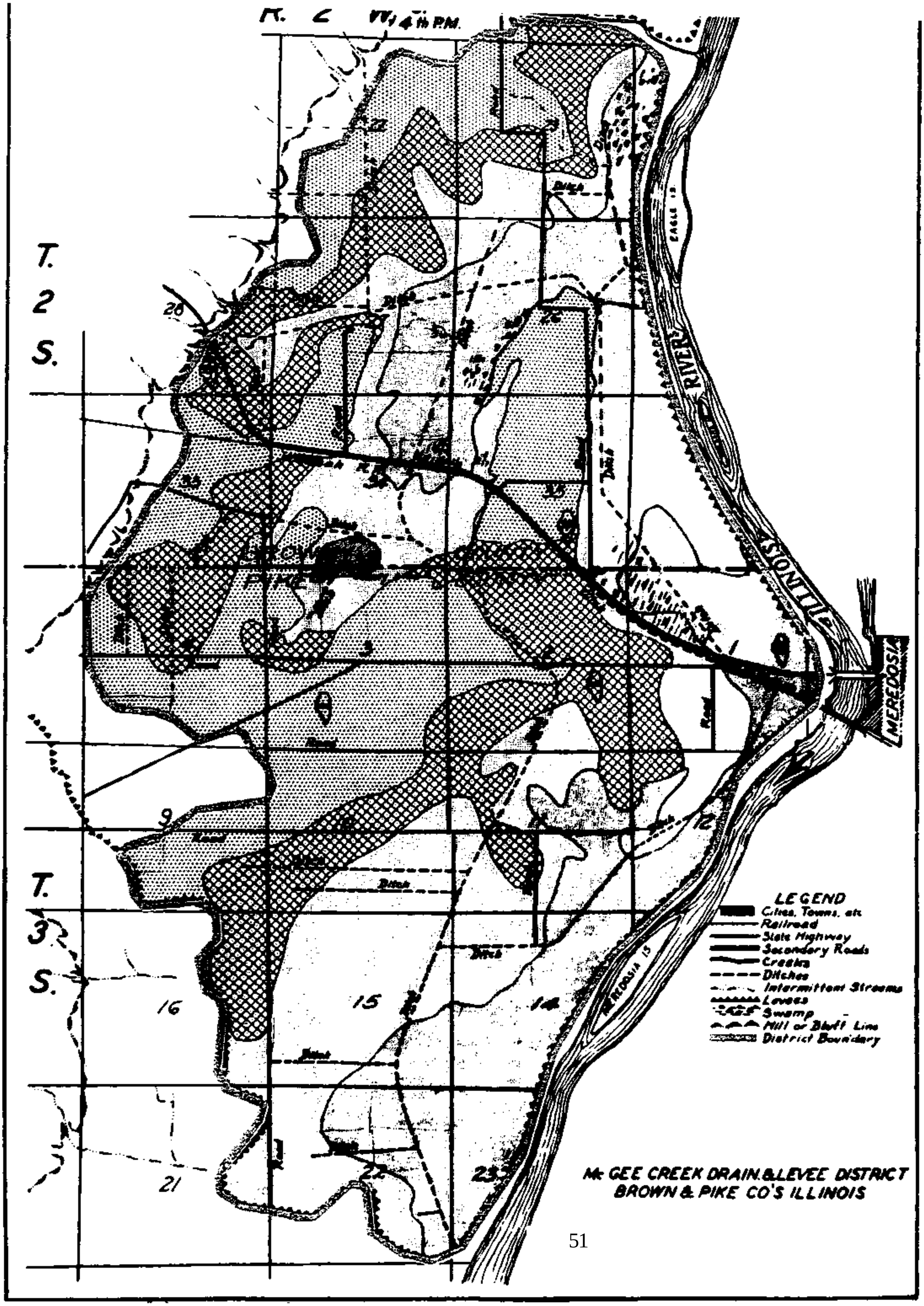

COAL CREEK DRAINAGE AND LEVEE DISTRICT
SCHUYLER COUNTY ILLINOIS
LEGEND
TOWNS
RAILROAD ON LEVEE
LEVEE
HIGHWAYS
RIVER
CREEK
DITCH
BRIDGE
DISTRICT B'NDRY
FREDERICK
R1W R1E
HIGH WATER 1944
COAL CREEK
NEW CHANNEL
4TH PRINCIPLE MERIDIAN
BRANCH A
STATE BOND ISSUE ROUTE #3
TRACK ON LEVEE
RIVER
STATE BOND ISSUE ROUTE #103
DITCH
EAST LATERAL
MAIN DITCH
T1N
BASE LINE
T1S
BEARDSTOWN
PUMP STATION
ILLINOIS
10 11 8
15 13 18 17
34
22 23 24 19
3
26 25 30
10
35 36
15

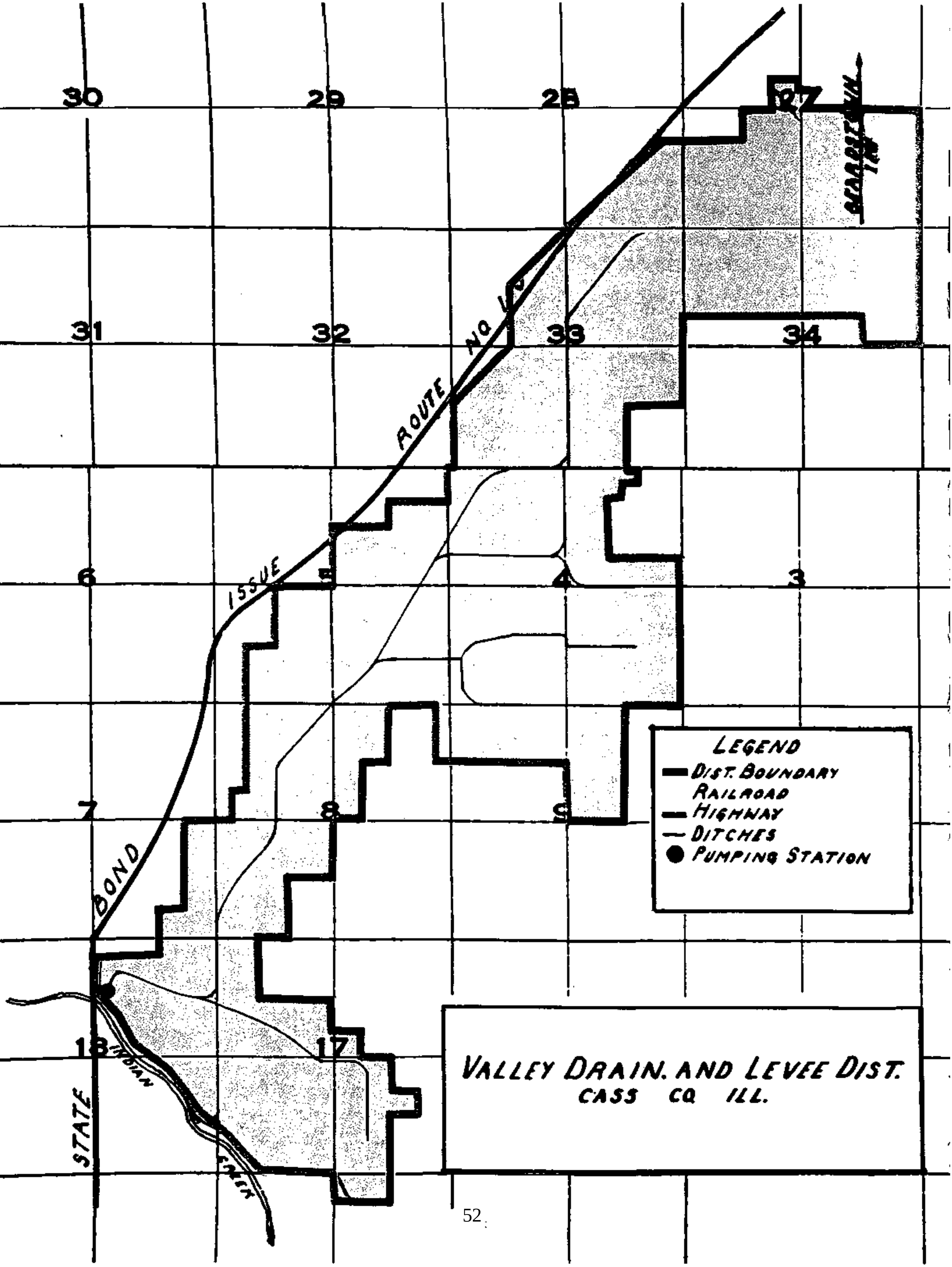


RANGE WEST 4TH P.M.
25
30
29
28
27
R. 1 W.
RUSHVILLE 7 MILES
STATE
STATE ROUTE #103
BOND
BEARDSTOWN 4 MILES
CRANE CREEK DRAINAGE & LEVEE DISTRICT
SCHUYLER COUNTY ILLINOIS
1844
10" TILE
32
33
34
T
1
N
BASE LINE
LEGEND
DISTRICT B'NDRY
HIGHWAY
BRIDGE
LEVEE
TILE DRAIN
CREEK
DITCH
6
5
4
3
HIGH
DITCH #5
10" TILE
DITCH #3
DITCH #4
RIVER
T
1
S
12
7
8
9
10
DITCH #8
PUMP STATION
GRAIN ELEVATOR
CROOKED
RIVER
ILLINOIS

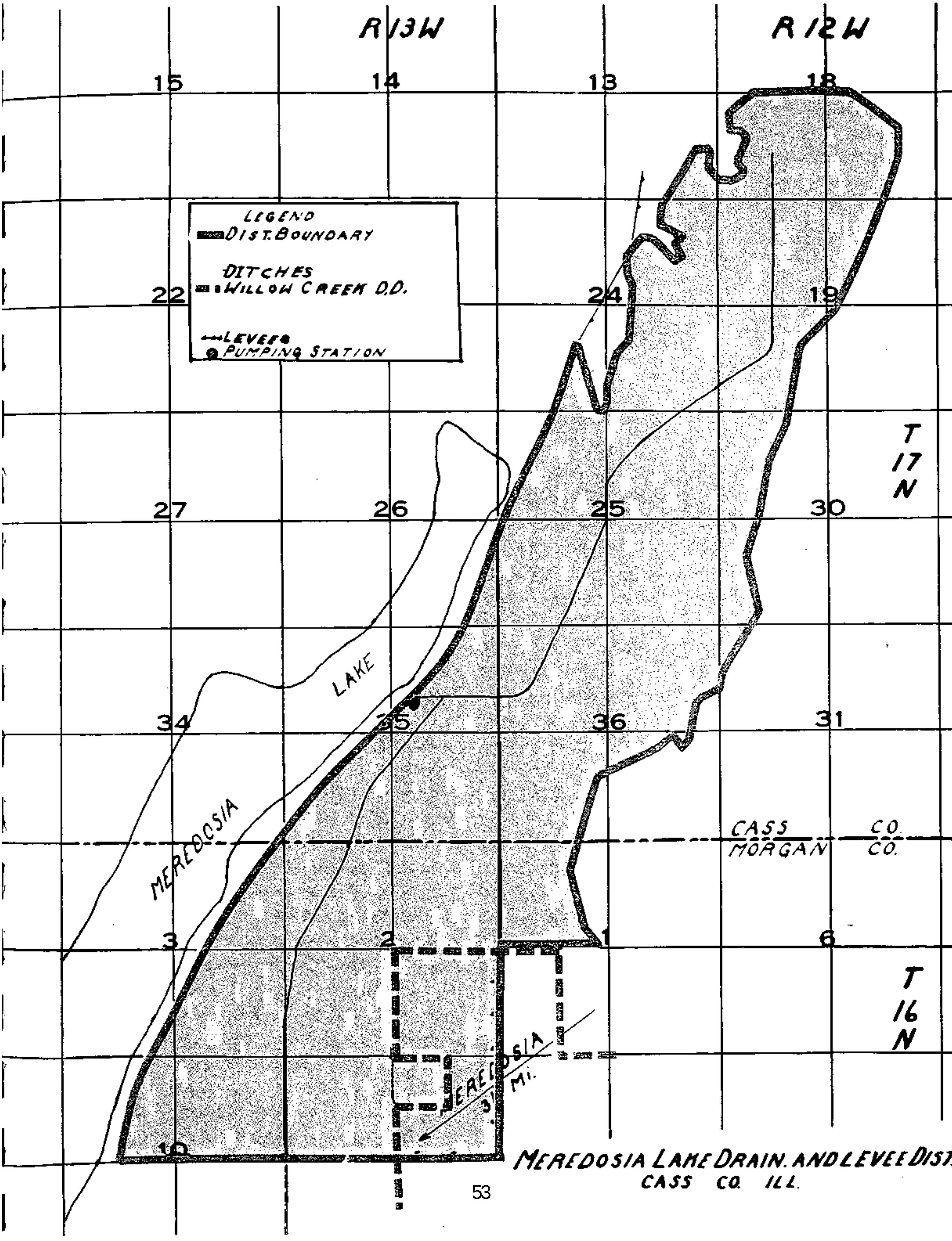

R 13 W
R 12 W
BEARDSTOWN
RIVER
T 18 N
T 17 N
PUMPING STATION
STATE BOND ISSUE ROUTE No 5
LEGEND
DIST. BOUNDARY
DITCHES
TILES
HIGHWAY
LEVEES
RIVER
SOUTH BEARDSTOWN LEVEE & DRAIN. DIST.
CASS CO. ILL.


R. 2 W. OF 4th P.M.
R. I. W.
District Boundary follows levee to Kamp Creek.
N.
T. 2 S.
Little Creek
RIVER
ILLINOIS
Pump plant
Former location of Lock & Dam
Ditch
Road
District Boundary follows road
State Road (under Const.)
12
Pump Plant
-LEGEND-
Roads
Rivers, Creeks, etc
Drainage Ditches
Intermittent Streams
Marshes
Existing Levees
Abandoned Levee
Pump Plants
Hill Line
District Boundary
Kamp Cr.
District Boundary follows levee to Little Creek.
LITTLE CREEK DRAIN. DIST.
BROWN COUNTY ILLINOIS

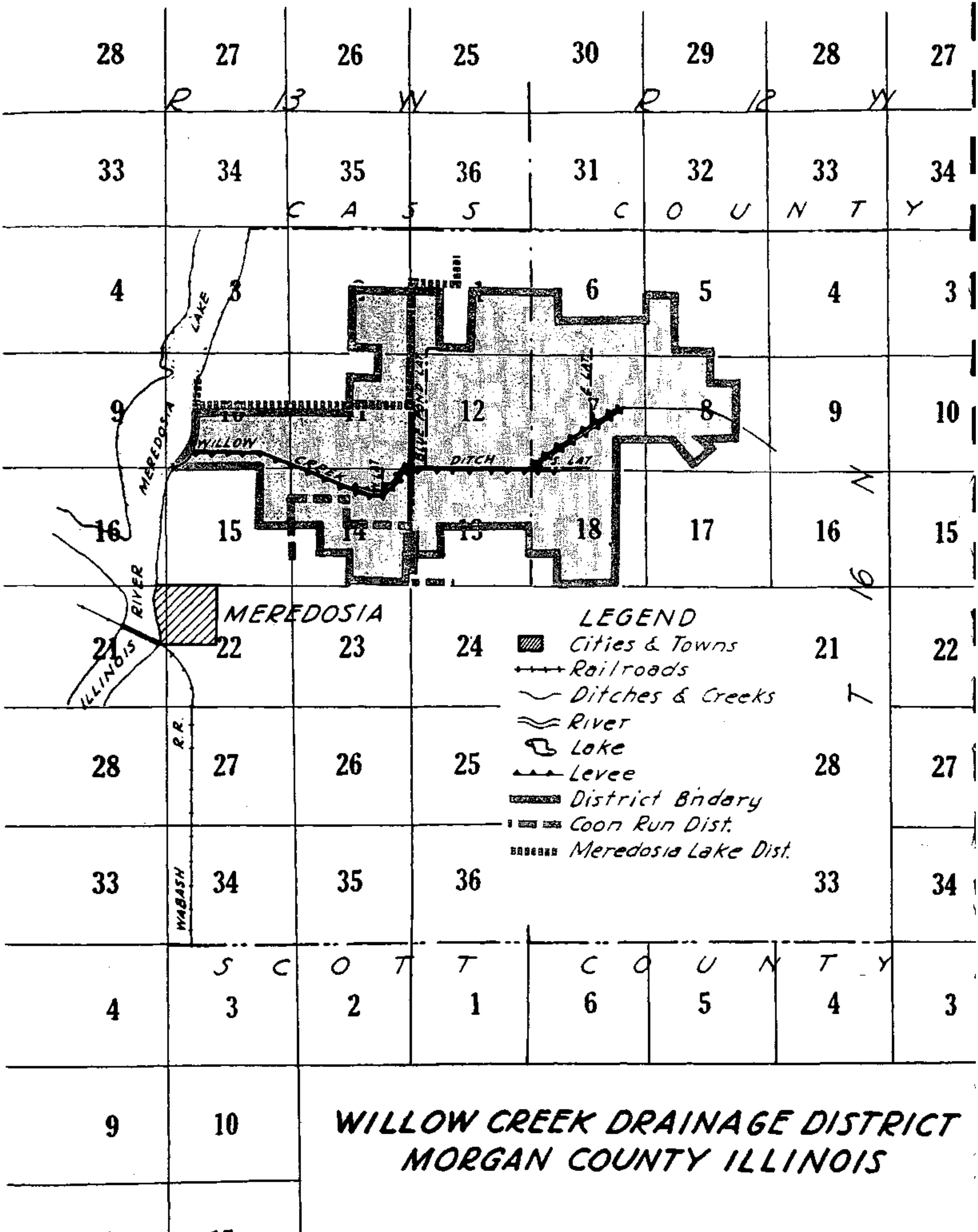

R. 2 W. 4th P.M.
T. 2 S.
T. 3 S.
ILLINOIS RIVER
MEREDOSIA
16
15
21
LEGEND
Cities, Towns, etc.
Railroad
State Highway
Secondary Roads
Creeks
Ditches
Intermittent Streams
Levees
Swamp
Hill or Bluff Line
District Boundary
Mc GEE CREEK DRAIN.& LEVEE DISTRICT
BROWN & PIKE CO'S ILLINOIS



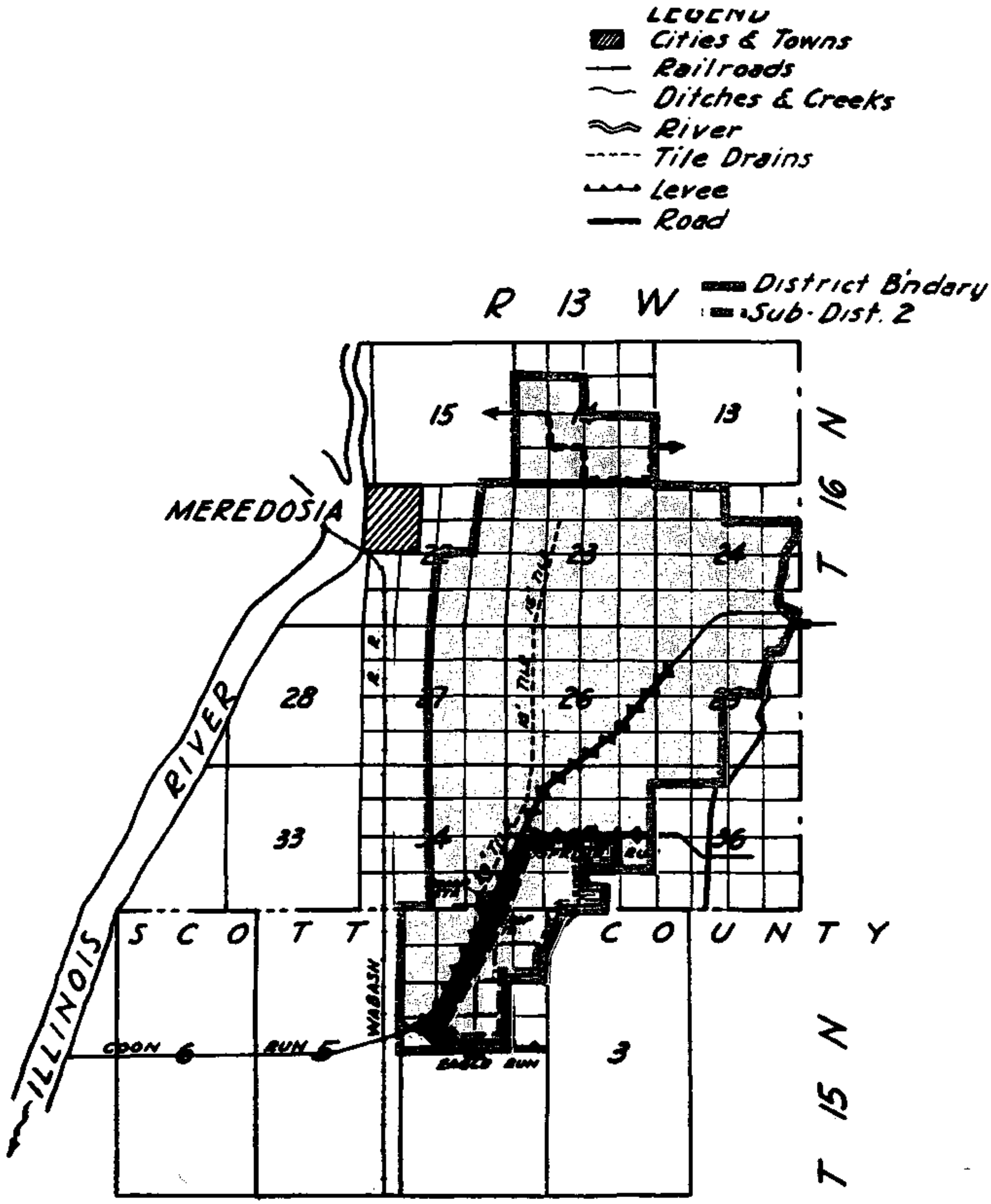
30
29
28
27
31
32
33
34
6
5
4
3
7
8
9
18
17
STATE
BOND
ISSUE
ROUTE NO
INDIAN
CREEK
LEGEND
DIST. BOUNDARY
RAILROAD
HIGHWAY
DITCHES
PUMPING STATION
VALLEY DRAIN. AND LEVEE DIST.
CASS CO. ILL.



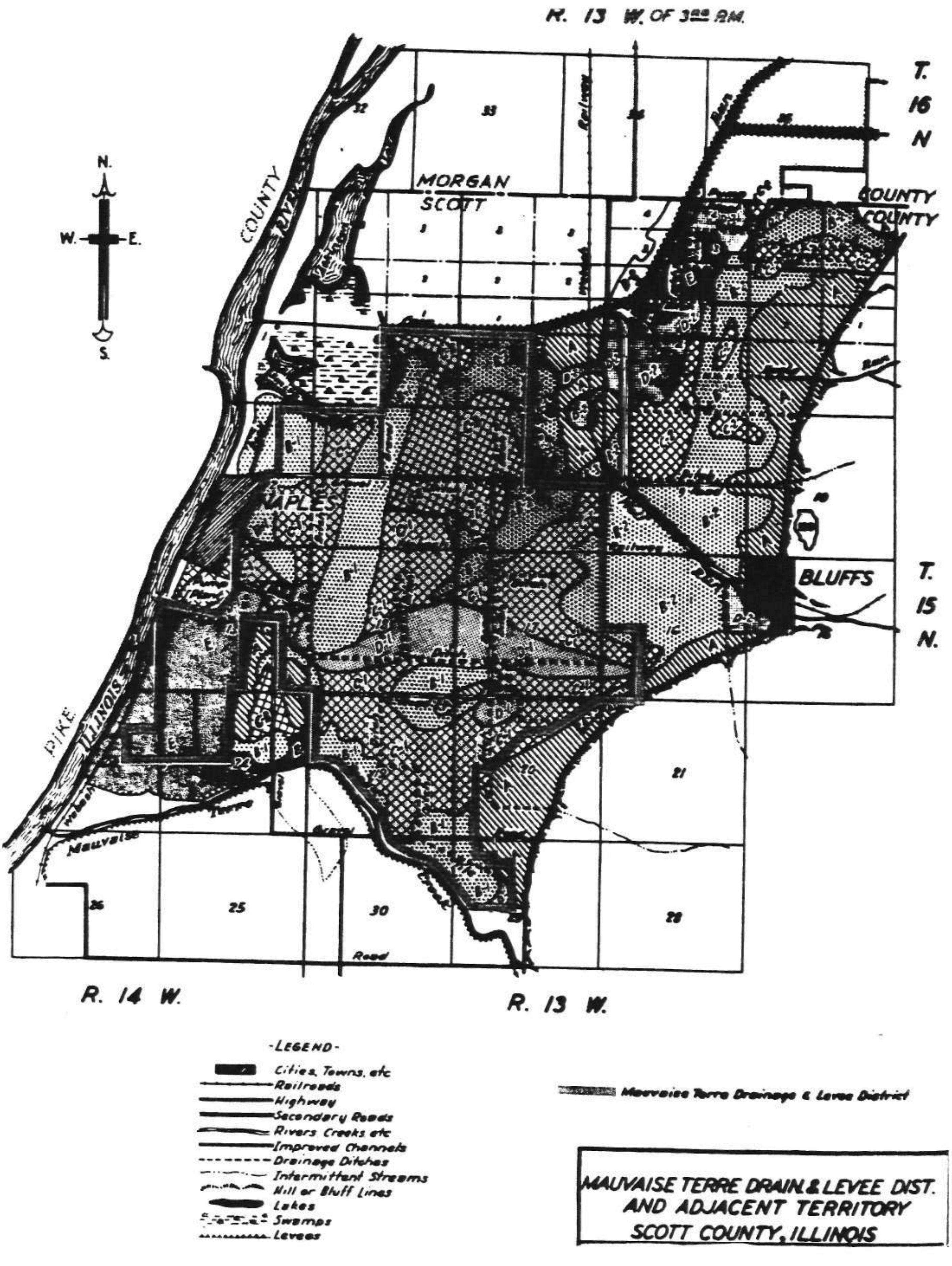
MEREDOSIA LAKE DRAIN. AND LEVEE DIST.
CASS CO. ILL.



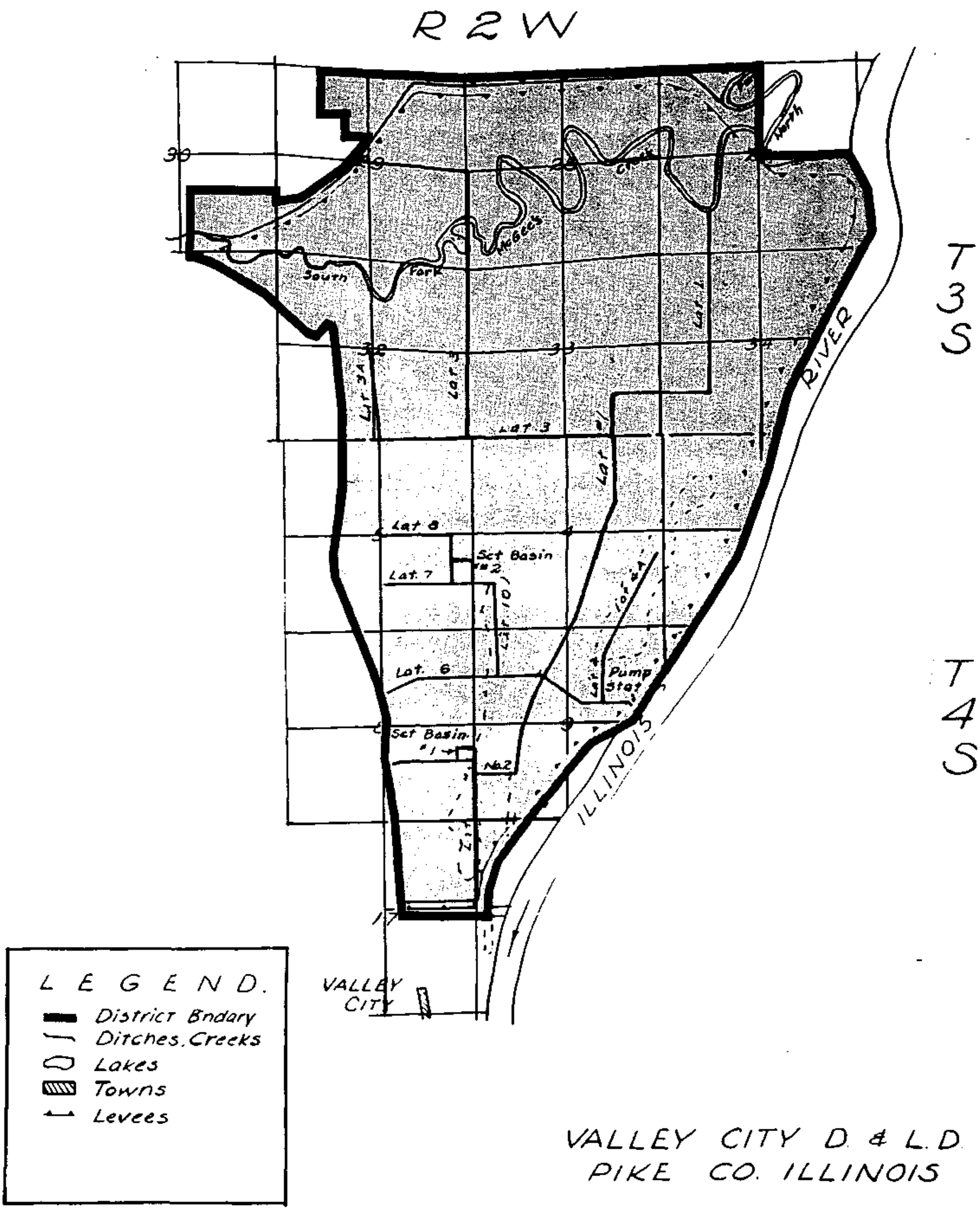
R 13 W
R 12 W
CASS COUNTY
SCOTT COUNTY
T 16 N
MEREDOSIA LAKE
WILLOW CREEK DITCH
ILLINOIS RIVER
MEREDOSIA
R.R.
WABASH
S. LAT.
LEGEND
Cities & Towns
Railroads
Ditches & Creeks
River
Lake
Levee
District Bndary
Coon Run Dist.
Meredosia Lake Dist.
WILLOW CREEK DRAINAGE DISTRICT
MORGAN COUNTY ILLINOIS

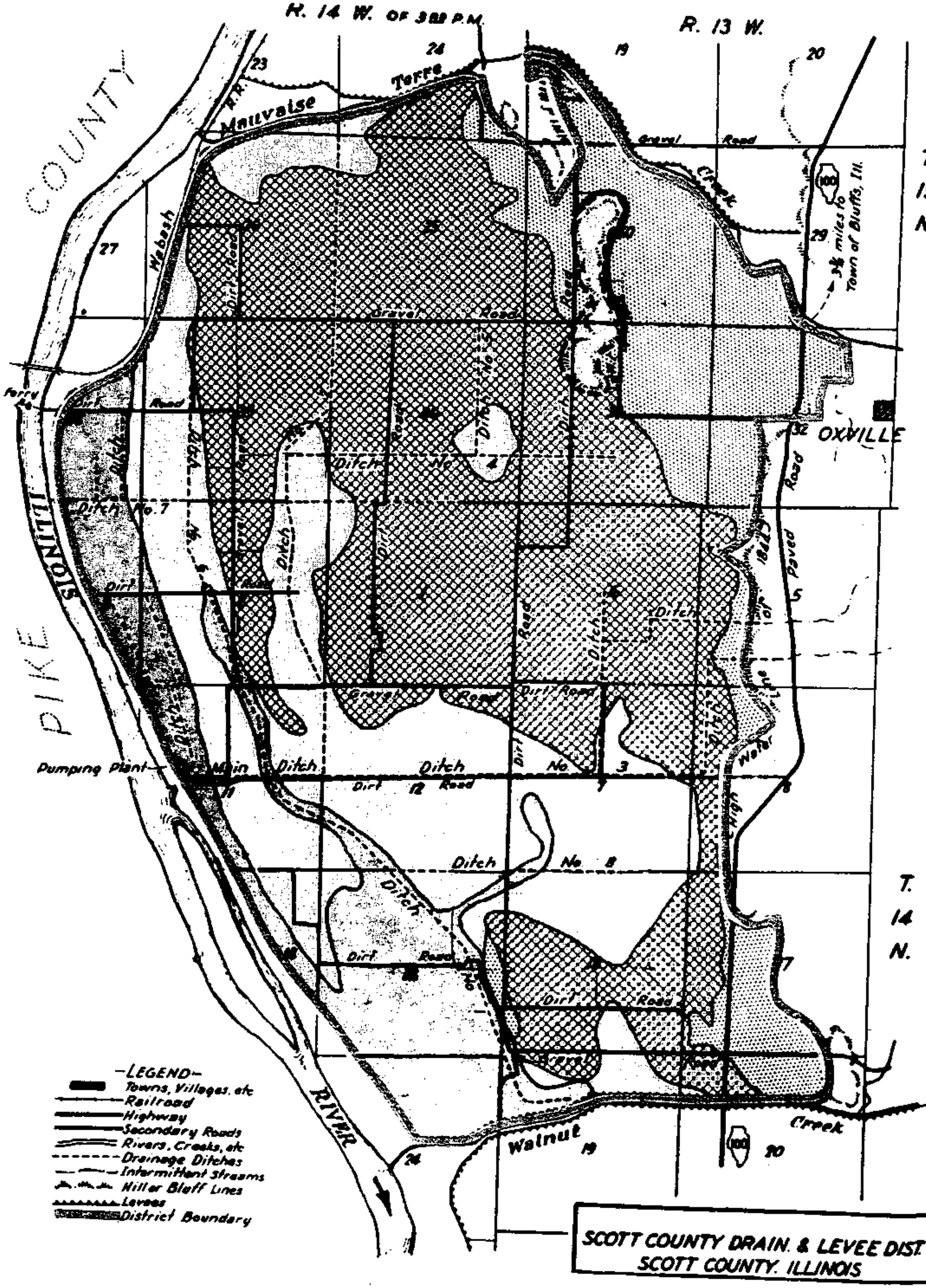


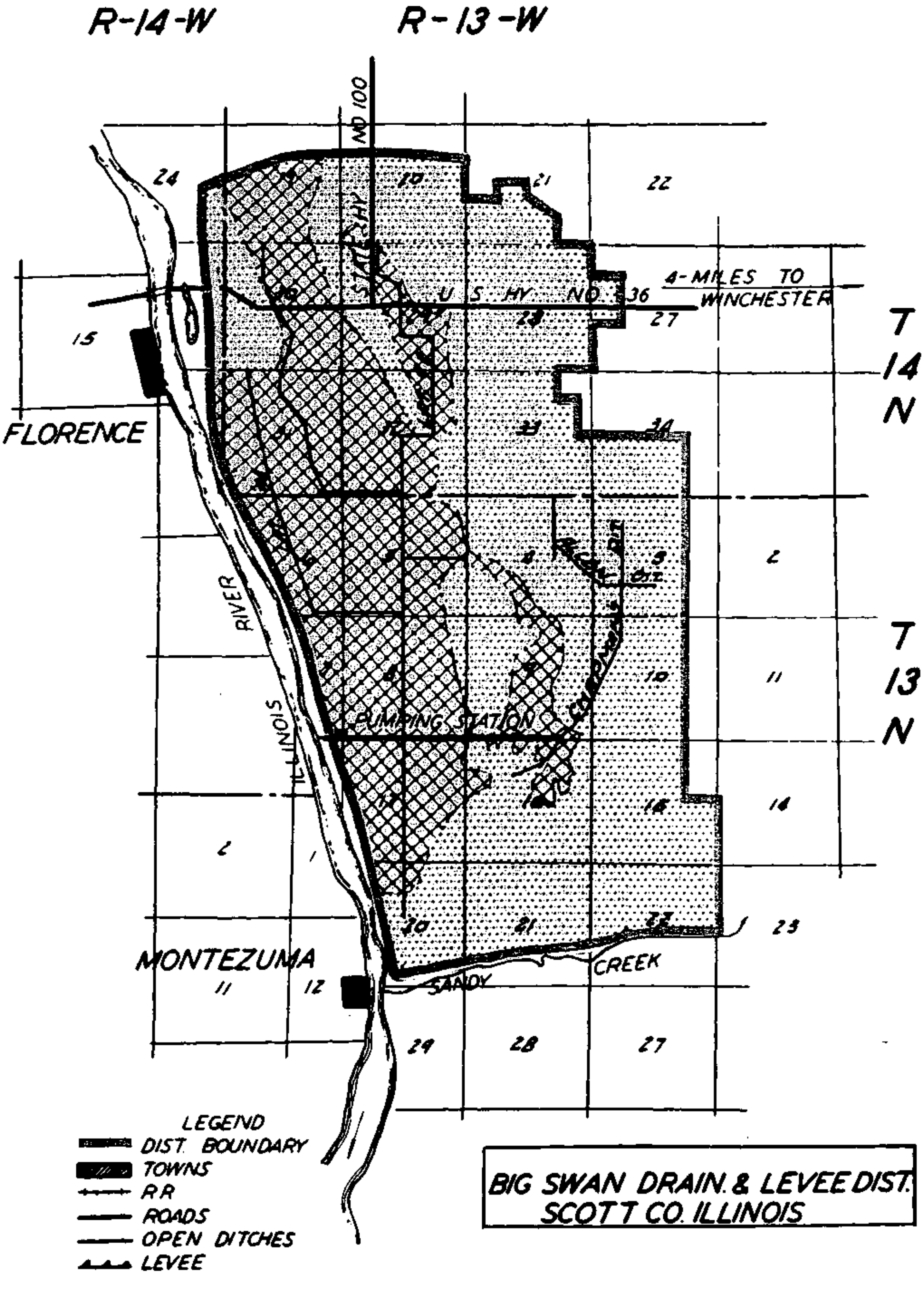
COON RUN DRAINAGE & LEVEE DISTRICT
MORGAN & SCOTT COUNTIES ILLINOIS

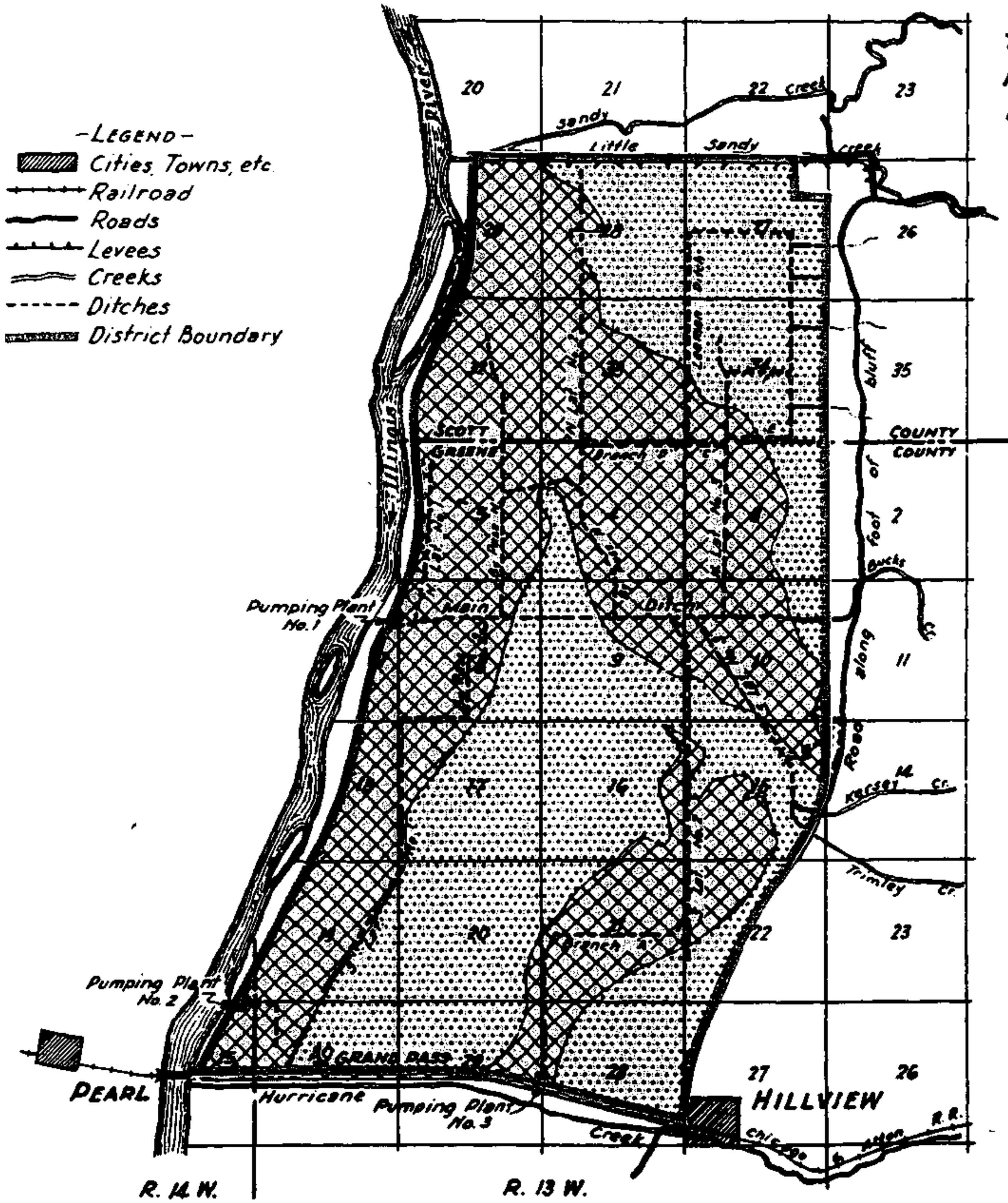


R. 13 W. OF 3RD P.M.
T. 16 N
MORGAN
SCOTT
COUNTY
COUNTY
COUNTY
N.
S.
W.
E.
NAPLES
BLUFFS
T. 15 N.
PIKE
ILLINOIS
Mauvaise
Terre
Road
32
33
21
25
26
30
28
R. 14 W.
R. 13 W.
-LEGEND-
Cities, Towns, etc
Railroads
Highway
Secondary Roads
Rivers Creeks etc
Improved Channels
Drainage Ditches
Intermittent Streams
Hill or Bluff Lines
Lakes
Swamps
Levees
Mauvaise Terre Drainage & Levee District
MAUVAISE TERRE DRAIN.& LEVEE DIST.
AND ADJACENT TERRITORY
SCOTT COUNTY, ILLINOIS

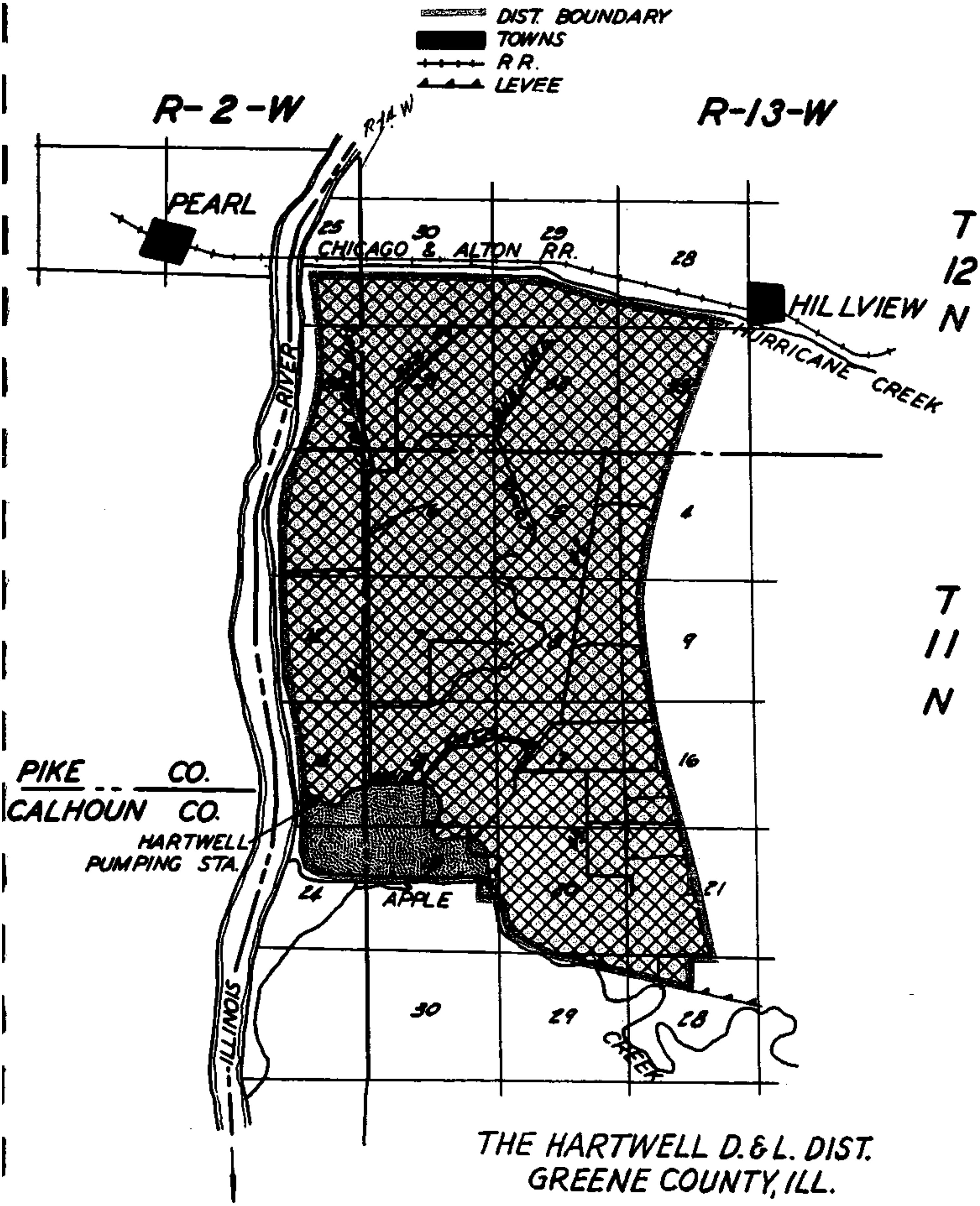

R 2 W
T 3 S
T 4 S
South Fork
McGees Creek
North
RIVER
ILLINOIS
Lat. 3
Lat. 8
Lat. 7
Lat. 6
Set Basin #2
Set Basin #1
No. 2
Pump Stat.
VALLEY CITY
LEGEND.
District Bndary
Ditches, Creeks
Lakes
Towns
Levees
VALLEY CITY D. & L. D.
PIKE CO. ILLINOIS


R. 14 W. OF 3RD P.M.
R. 13 W.
T. 15 N.
T. 14 N.
PIKE COUNTY
ILLINOIS RIVER
Mauvaise Terre
Wabash R.R.
Gravel Road
Creek
3½ miles to Town of Bluffs, Ill.
OXVILLE
Ferry
Dirt Road
Ditch No. 7
Ditch No. 4
Paved Road
Pumping Plant
Main Ditch
Ditch No. 3
Ditch No. 8
High Water Line of 1844
Walnut Creek
-LEGEND-
Towns, Villages, etc
Railroad
Highway
Secondary Roads
Rivers, Creeks, etc
Drainage Ditches
Intermittent Streams
Hill or Bluff Lines
Levees
District Boundary
SCOTT COUNTY DRAIN. & LEVEE DIST.
SCOTT COUNTY, ILLINOIS

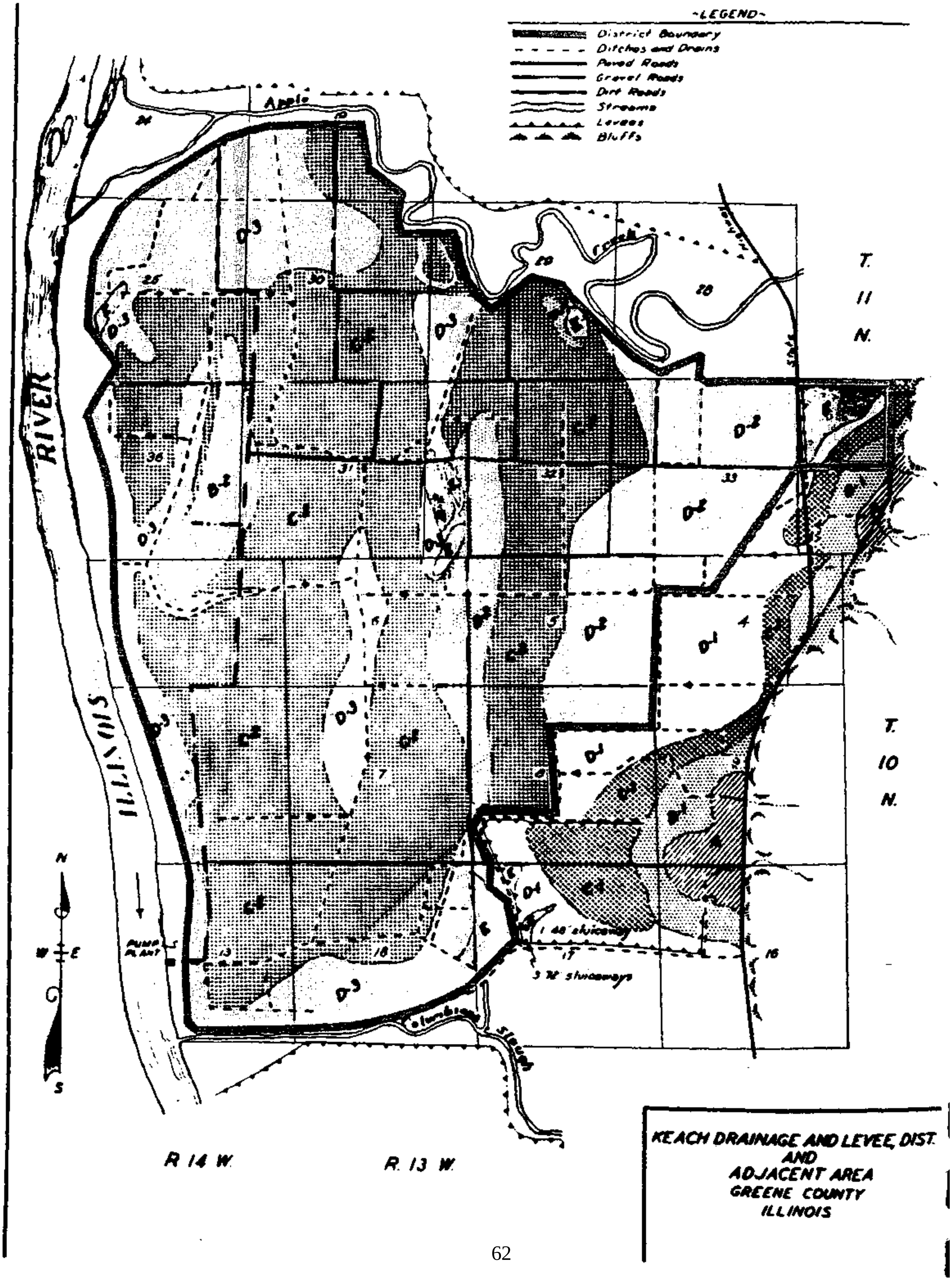

R-14-W
R-13-W
STATE HY. NO 100
U S HY NO 36
4-MILES TO WINCHESTER
T 14 N
T 13 N
FLORENCE
MONTEZUMA
ILLINOIS RIVER
PUMPING STATION
SANDY CREEK
LEGEND
DIST. BOUNDARY
TOWNS
RR
ROADS
OPEN DITCHES
LEVEE
BIG SWAN DRAIN. & LEVEE DIST.
SCOTT CO. ILLINOIS



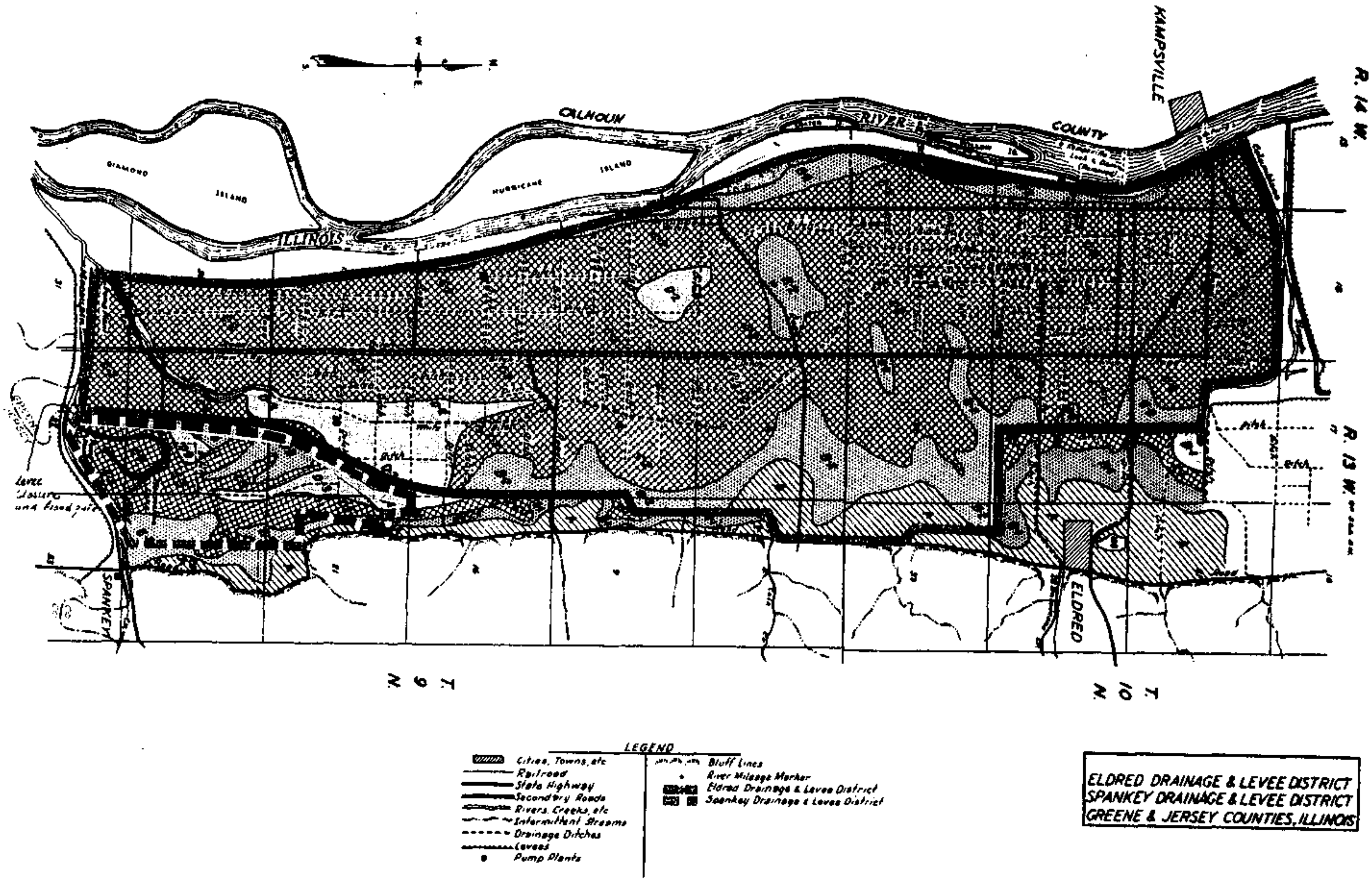
THE HILLVIEW D. & L. DIST.
GREENE & SCOTT COS., ILL.



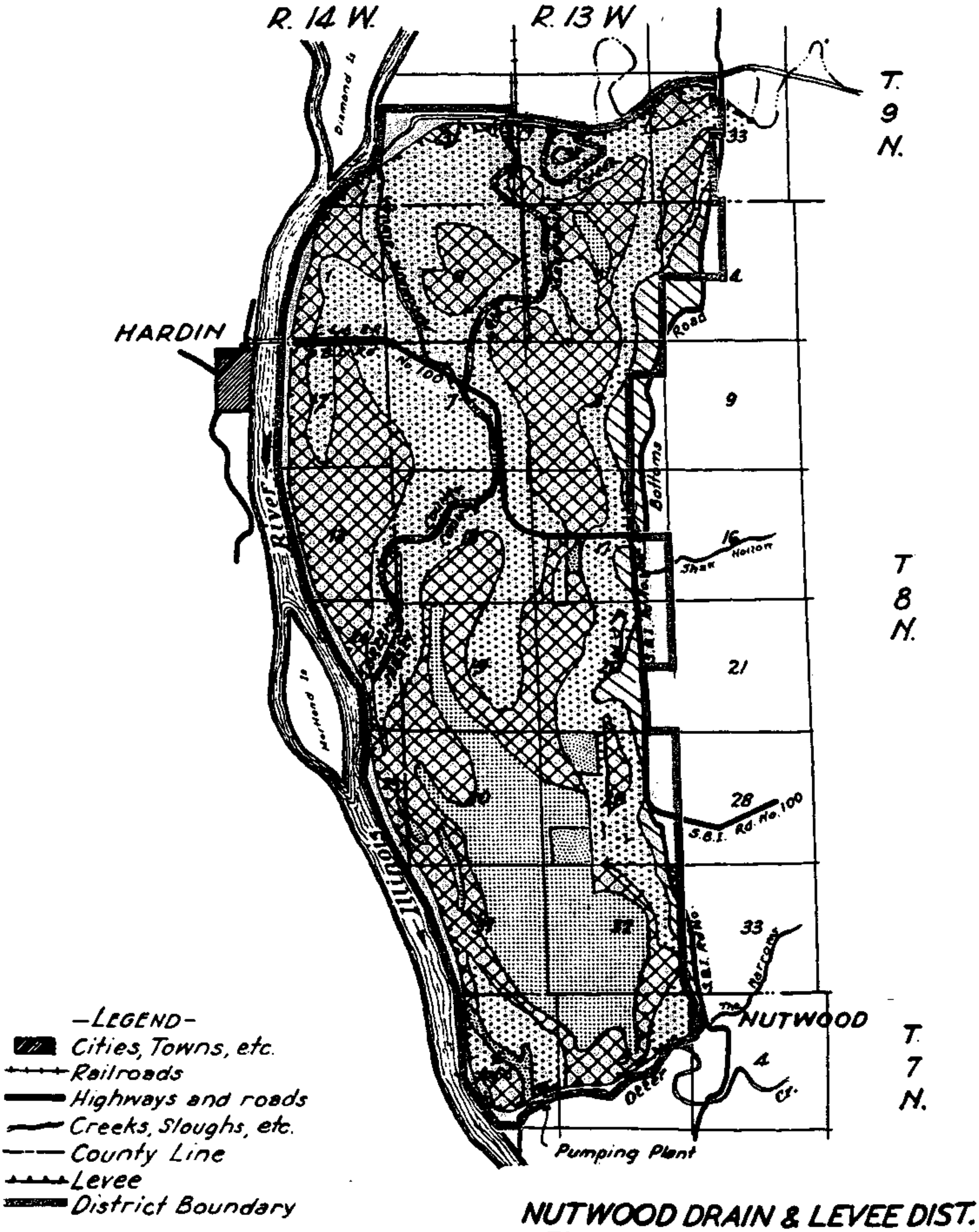
THE HARTWELL D.&L. DIST.
GREENE COUNTY, ILL.



KEACH DRAINAGE AND LEVEE DIST.
AND
ADJACENT AREA
GREENE COUNTY
ILLINOIS



ELDRED DRAINAGE & LEVEE DISTRICT
SPANKEY DRAINAGE & LEVEE DISTRICT
GREENE & JERSEY COUNTIES, ILLINOIS



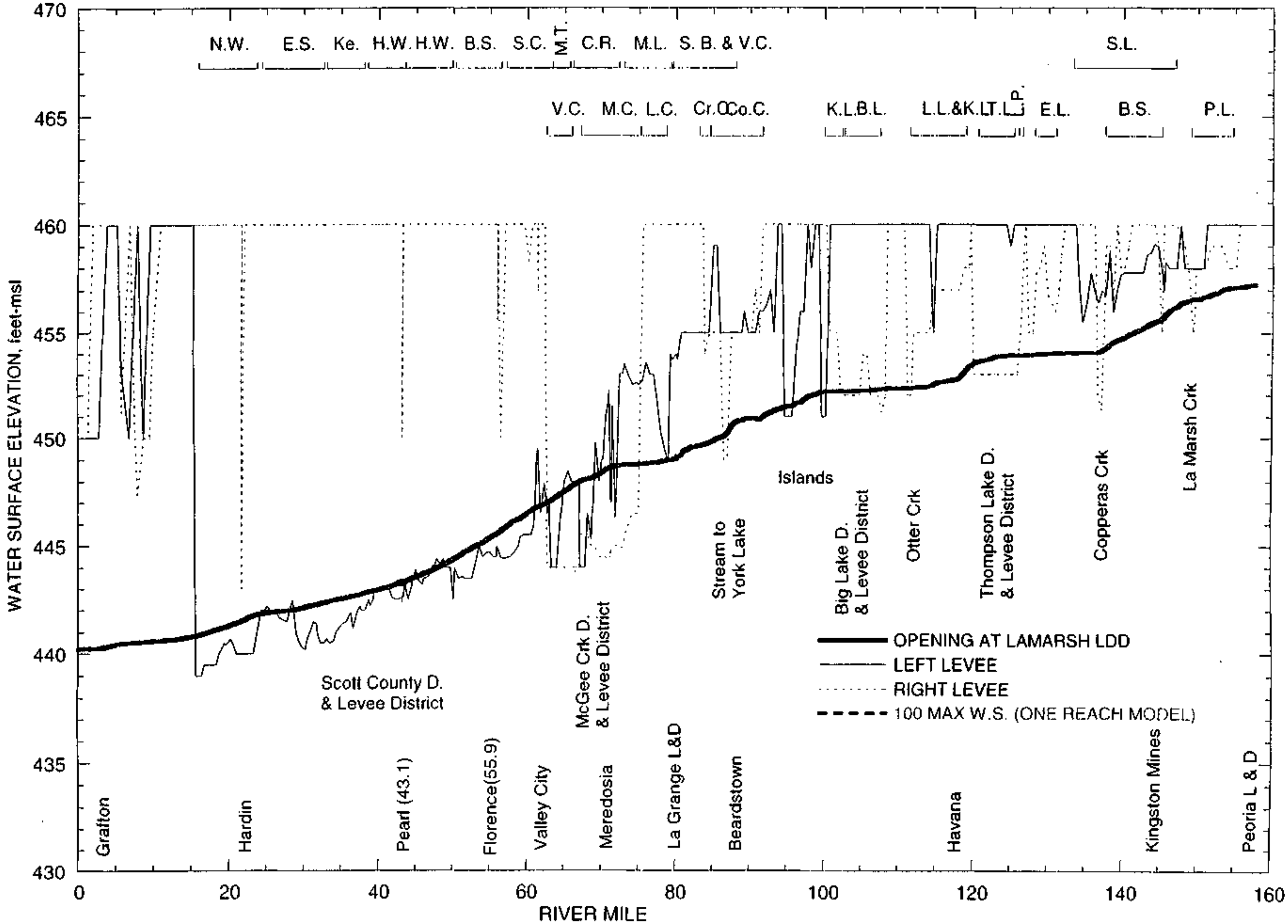
NUTWOOD DRAIN & LEVEE DIST.
JERSEY & GREENE COS., ILLINOIS

APPENDIX II.

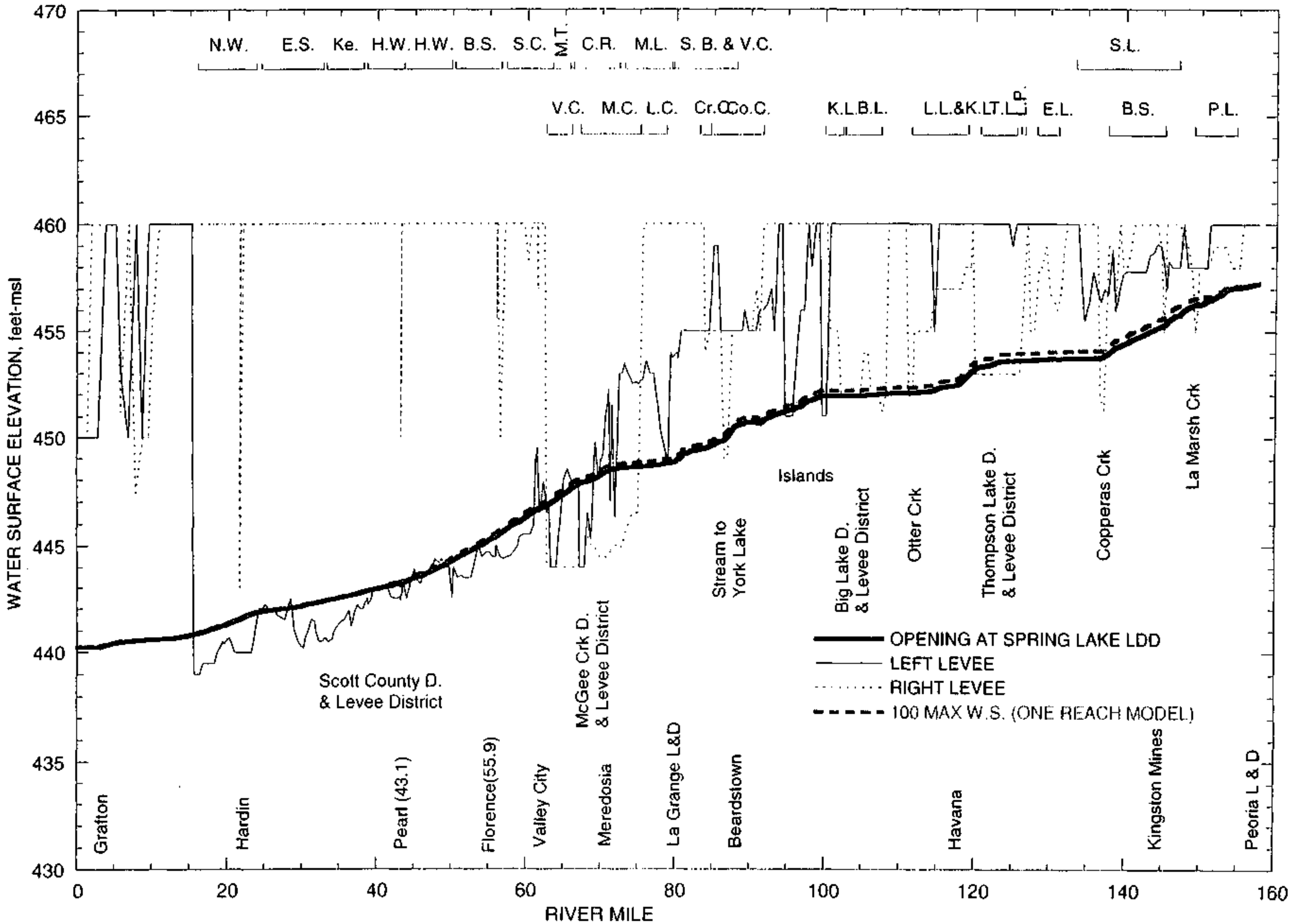
PEAK WATER SURFACE ELEVATION PROFILES WITH AND WITHOUT AN 1000 BY 6 FEET OPENING ON THE LEVEE OF EACH LEVEE AND DRAINAGE DISTRICT

Acronyms for Levee and Drainage Districts Used in Appendix II

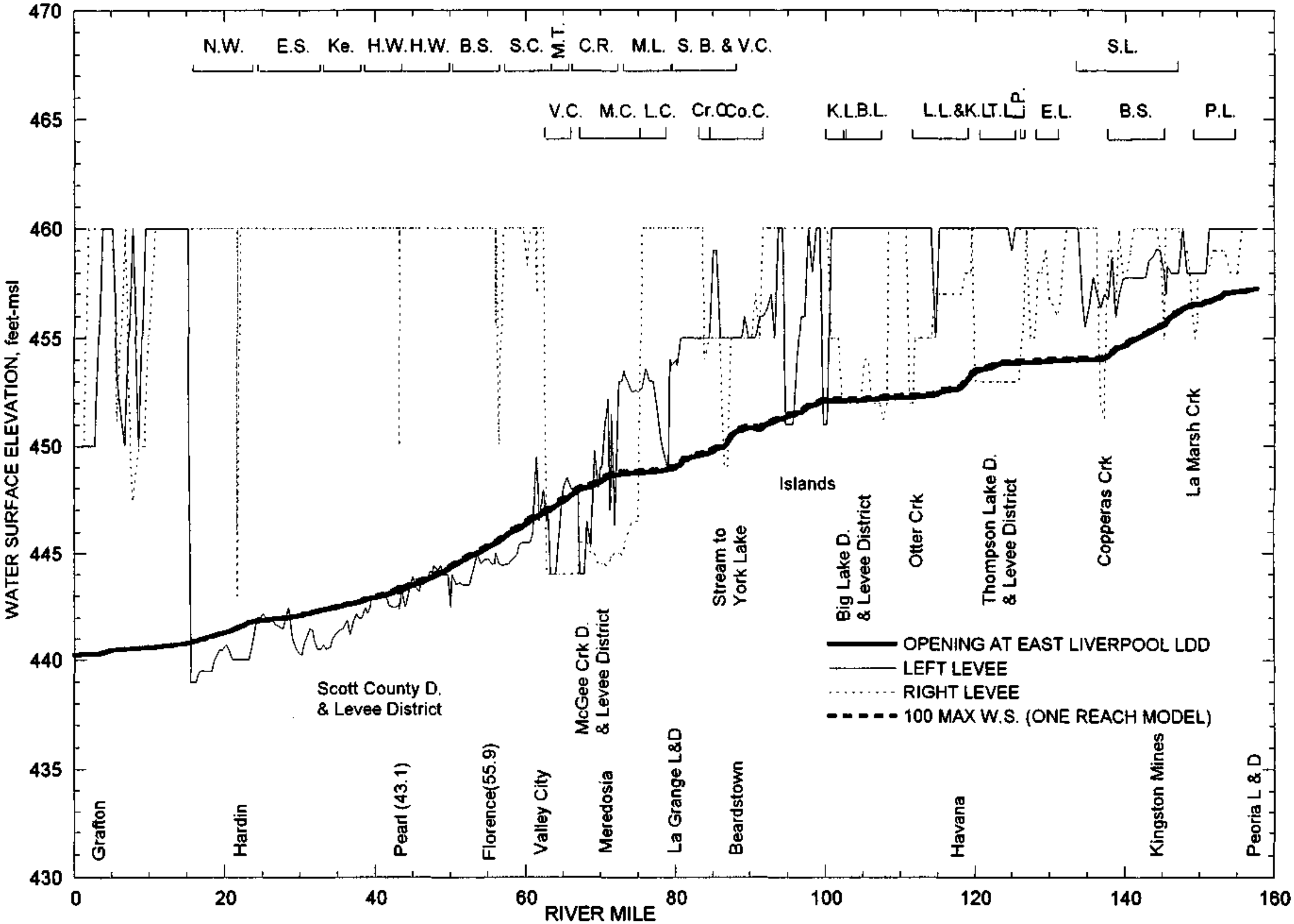
Levee and Drainage District	Abbreviation
Pekin & LaMarsh	P.L.
Spring Lake	S.L.
Benner Special	B.S.
East Liverpool	E.L
Liverpool	L.P.
Thompson Lake	T.L.
Lacey, Langellier, W. Matanzas and Kerton	L.L.&K.
Big Lake	B.L
Kelly Lake	K.L.
Coal Creek	C.C.
S. Beardstown and Valley	
Crane Creek	Cr.C.
Meredosia Lake and Willow Creek	
Little Creek	LC.
McGee Creek	M.C.
Valley City	V.C.
Mauvaise Terre	M.T.
Scott County	S.C.
Big Swan	B.S.
Hillview	H.W.
Hartwell	H.W.
Keach	
Eldred & Spanky	E.S.
Nutwood	N.W.



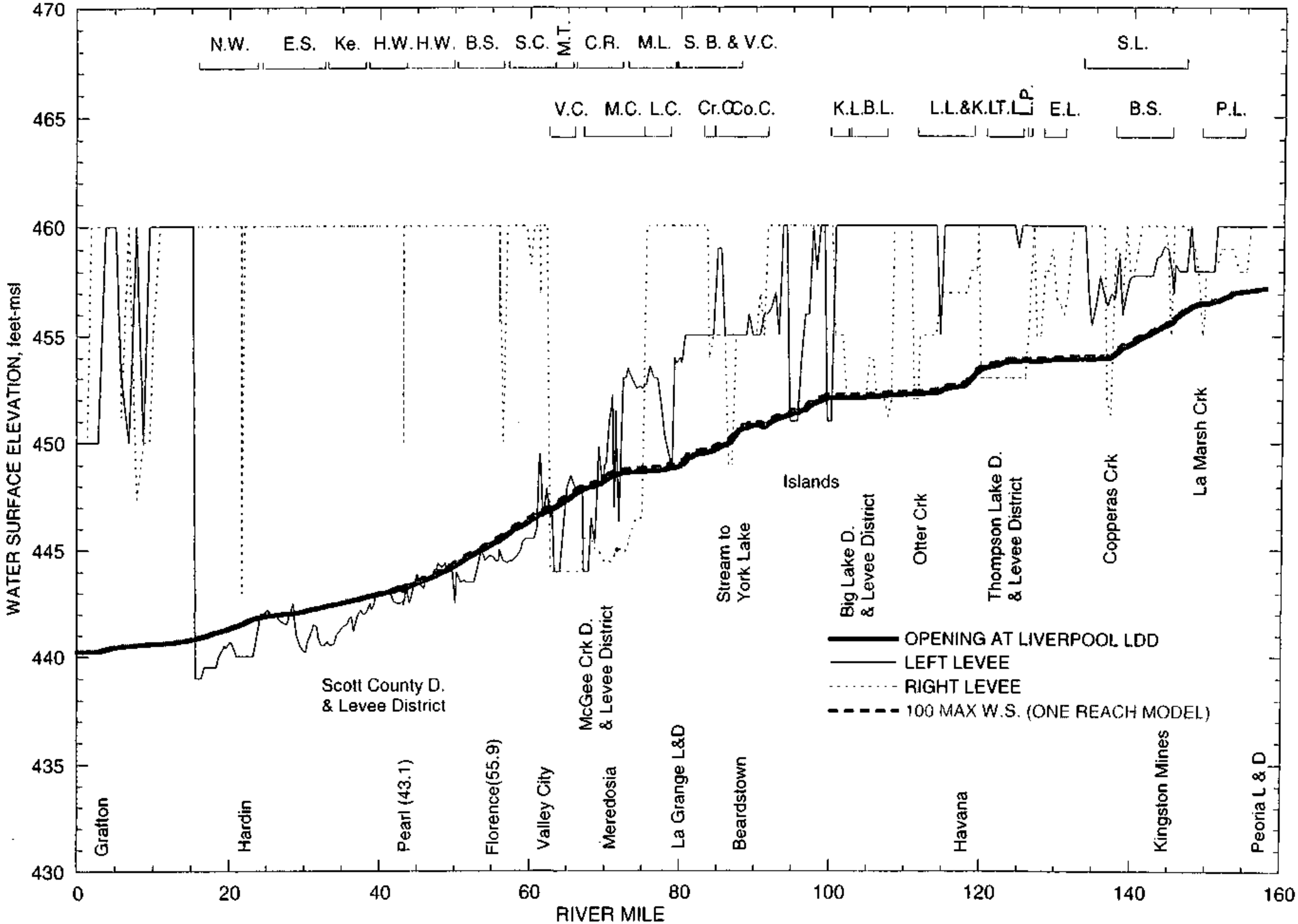
Peak WSR profiles with and without an opening at LaMarsh LDD



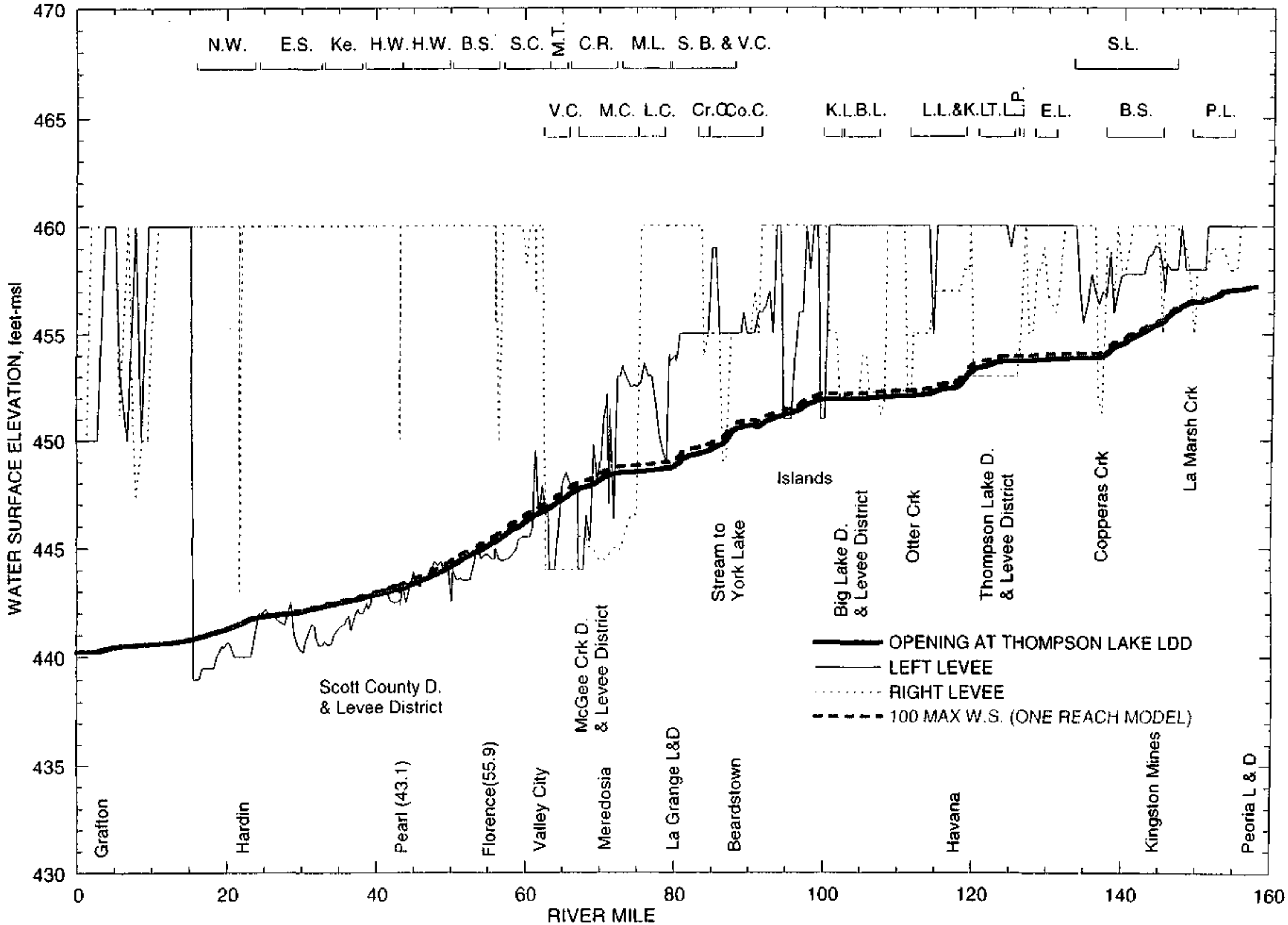
Peak WSE profiles with and without an opening at Spring Lake LDD



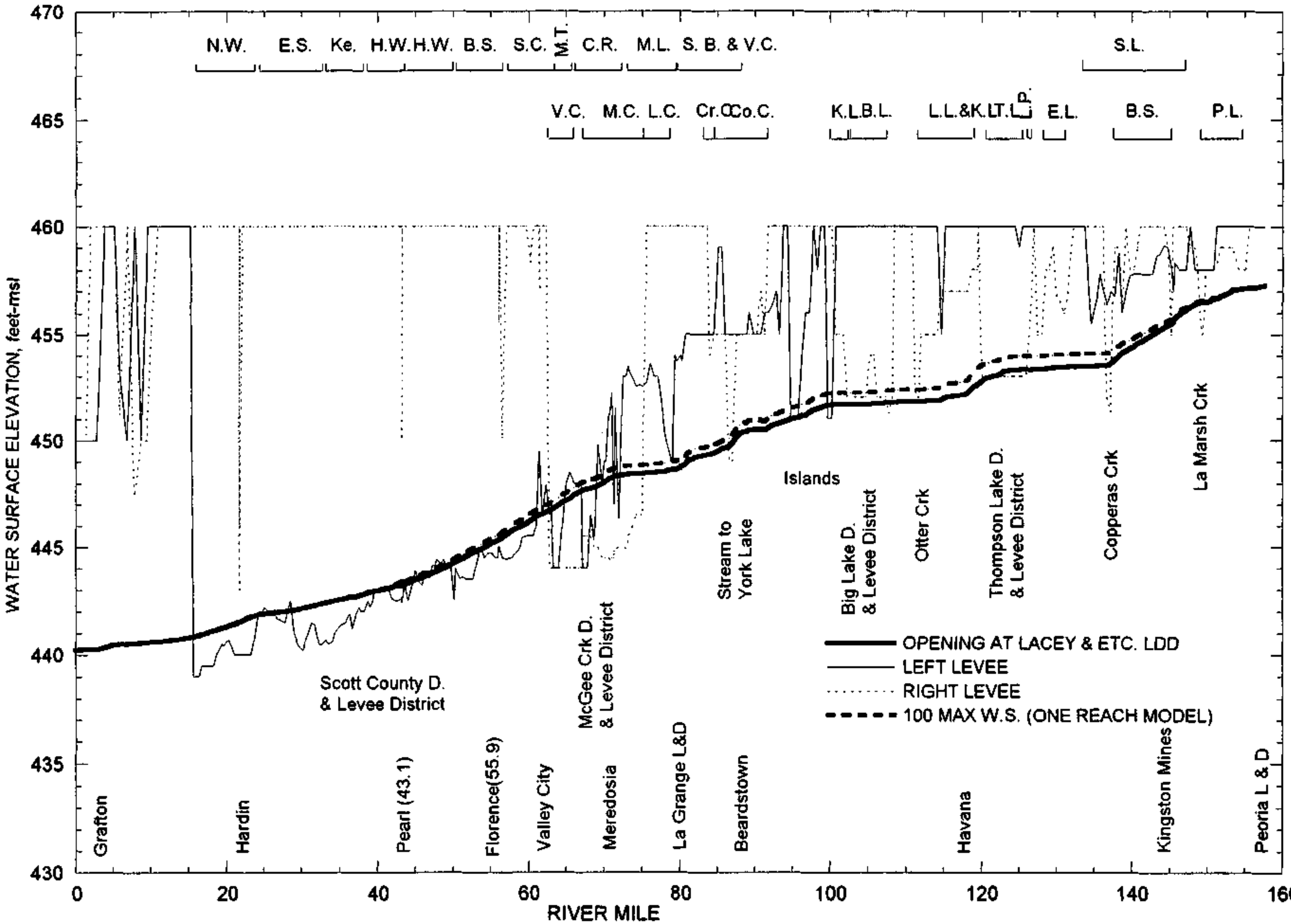
. Peak WSE profiles with and without an opening at East Liverpool LDD



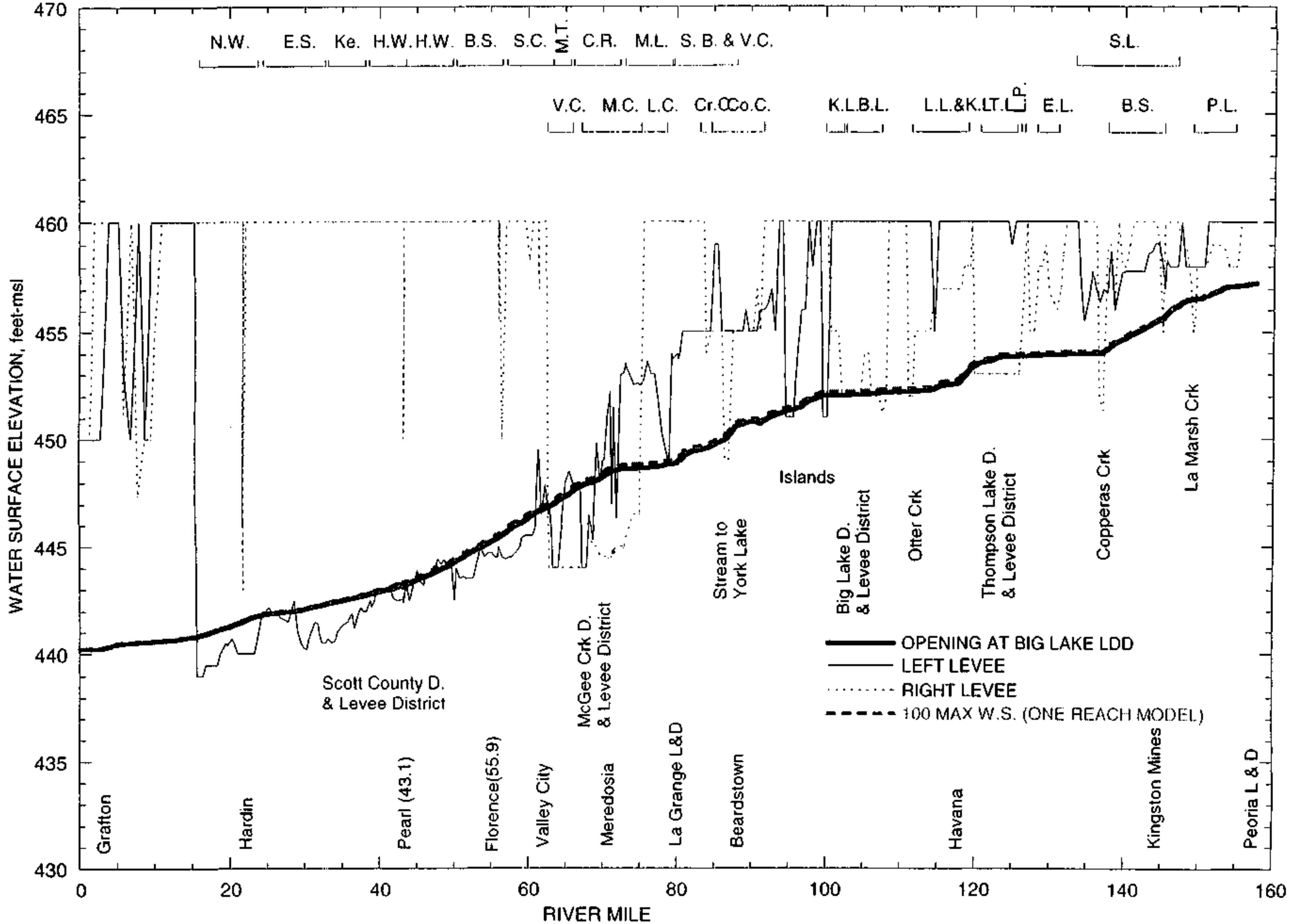
Peak WSE profiles with and without an opening at Liverpool LDD



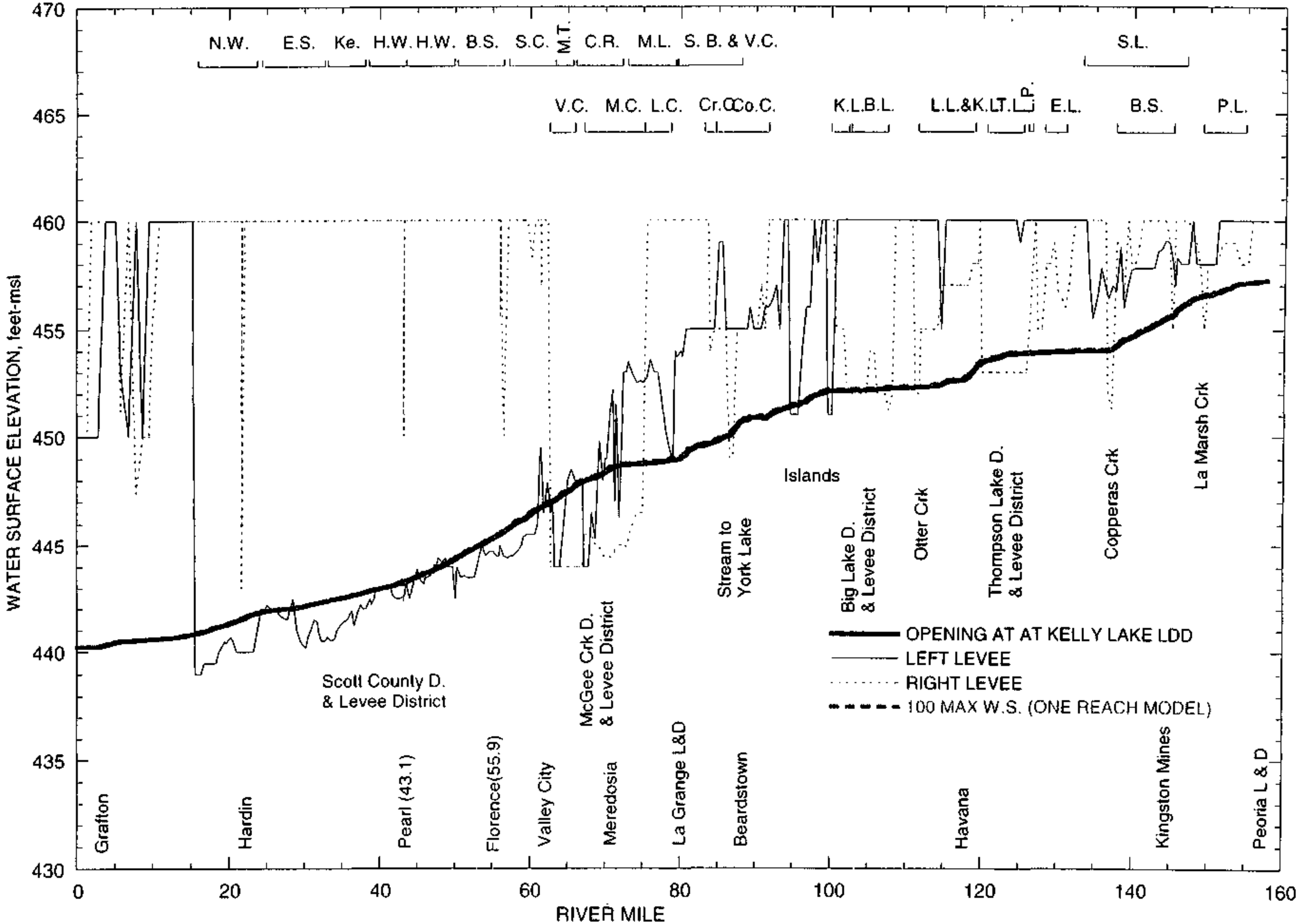
Peak WSE profiles with and without an opening at Thompson Lake LDD



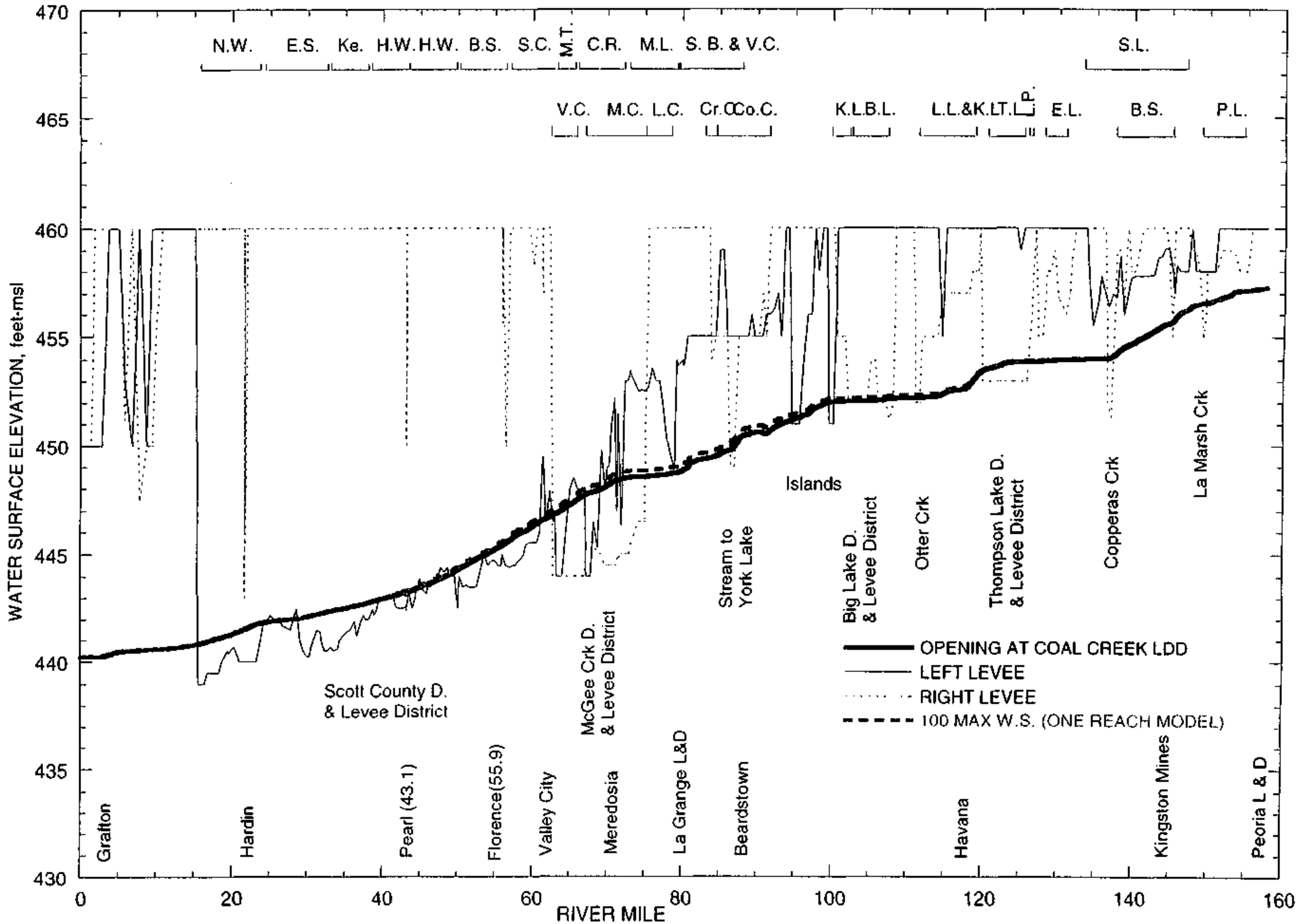
Peak WSE profiles with and without an opening at Lacey, Langellier, W. Matanzas & Kerton LDD



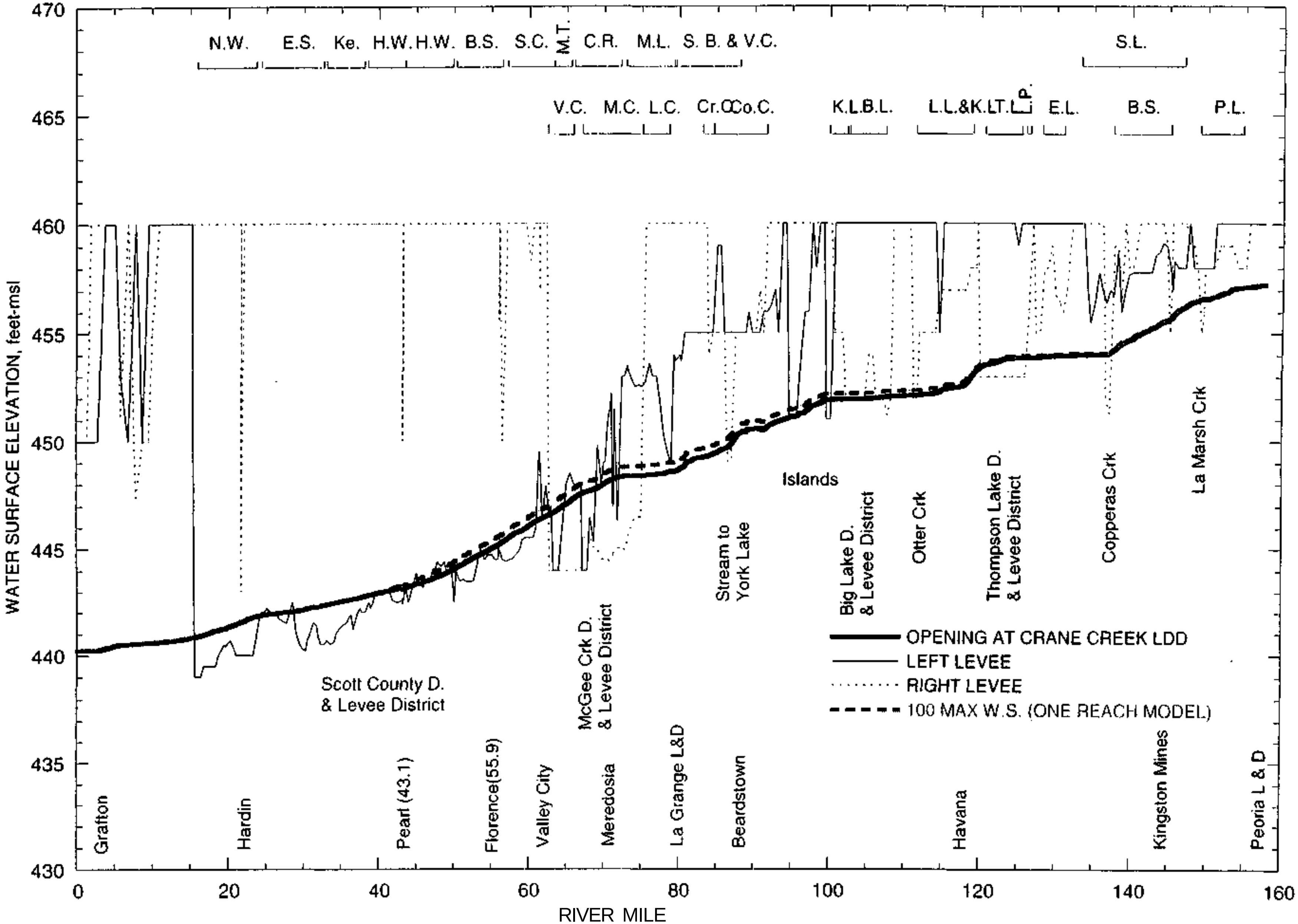
Peak WSE profiles with without an opening at Big Lake LDD



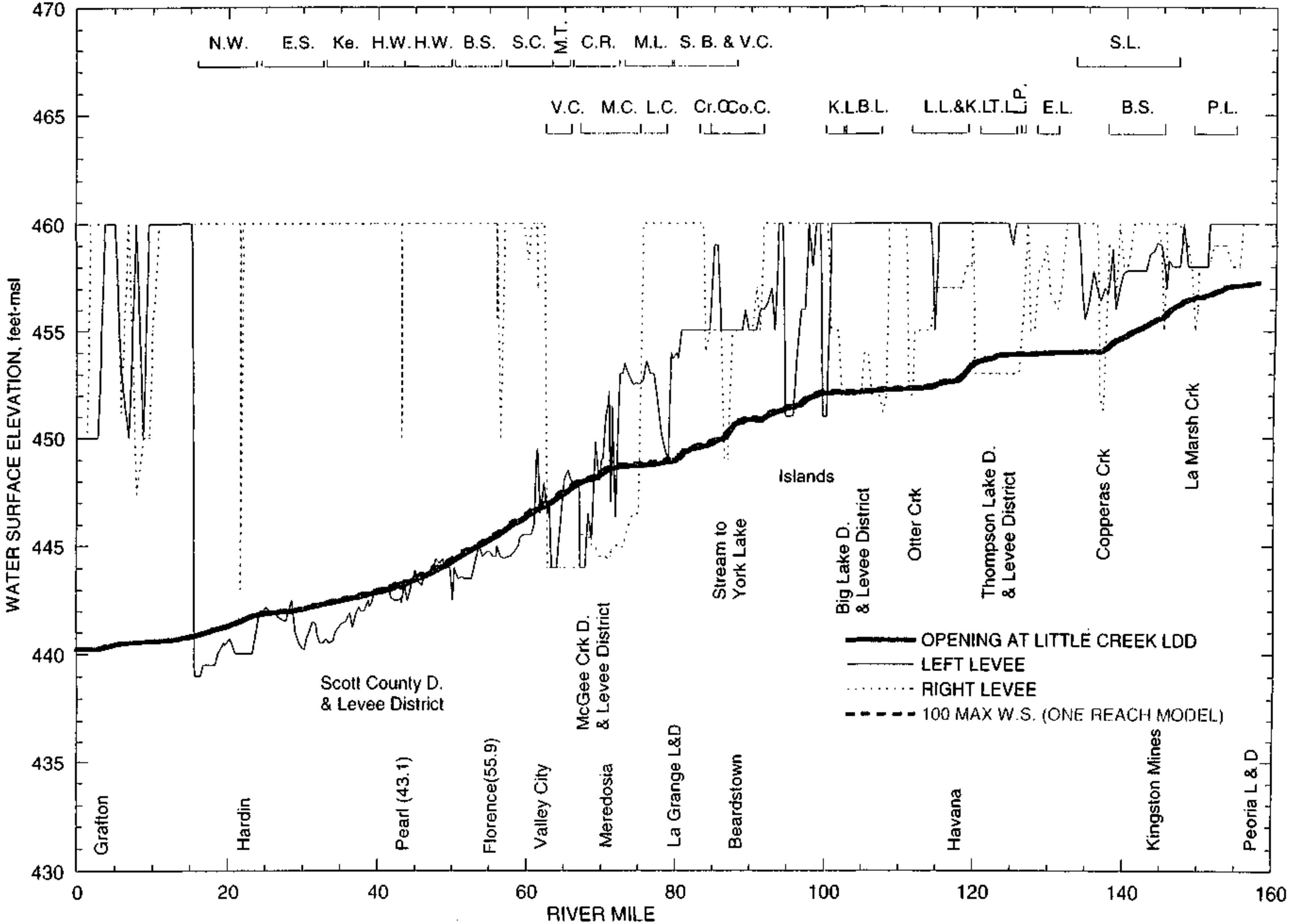
Peak WSE profiles with and without an opening at Kelly Lake LDD



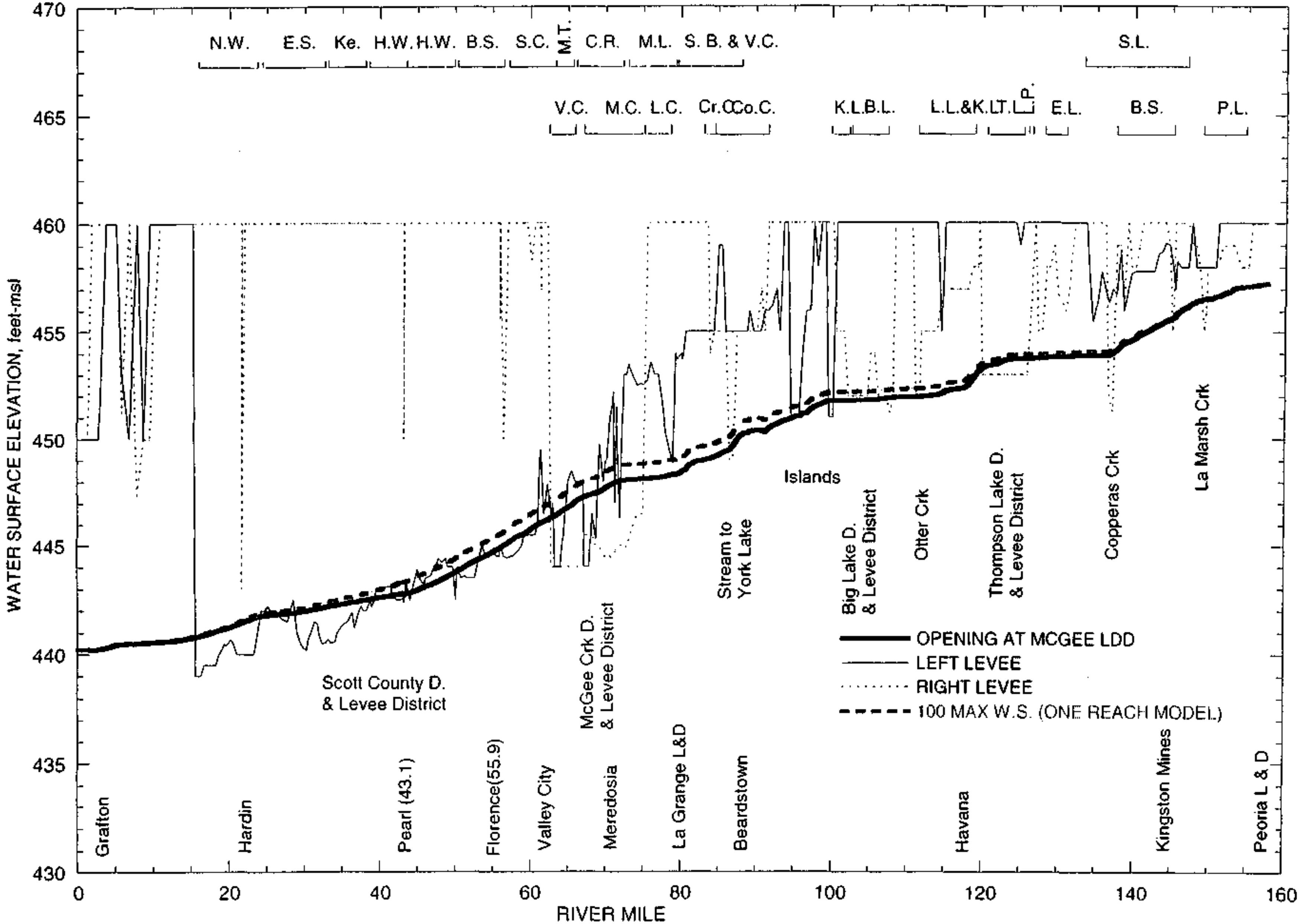
Peak WSE profiles with and without an opening at Coal Creek LDD



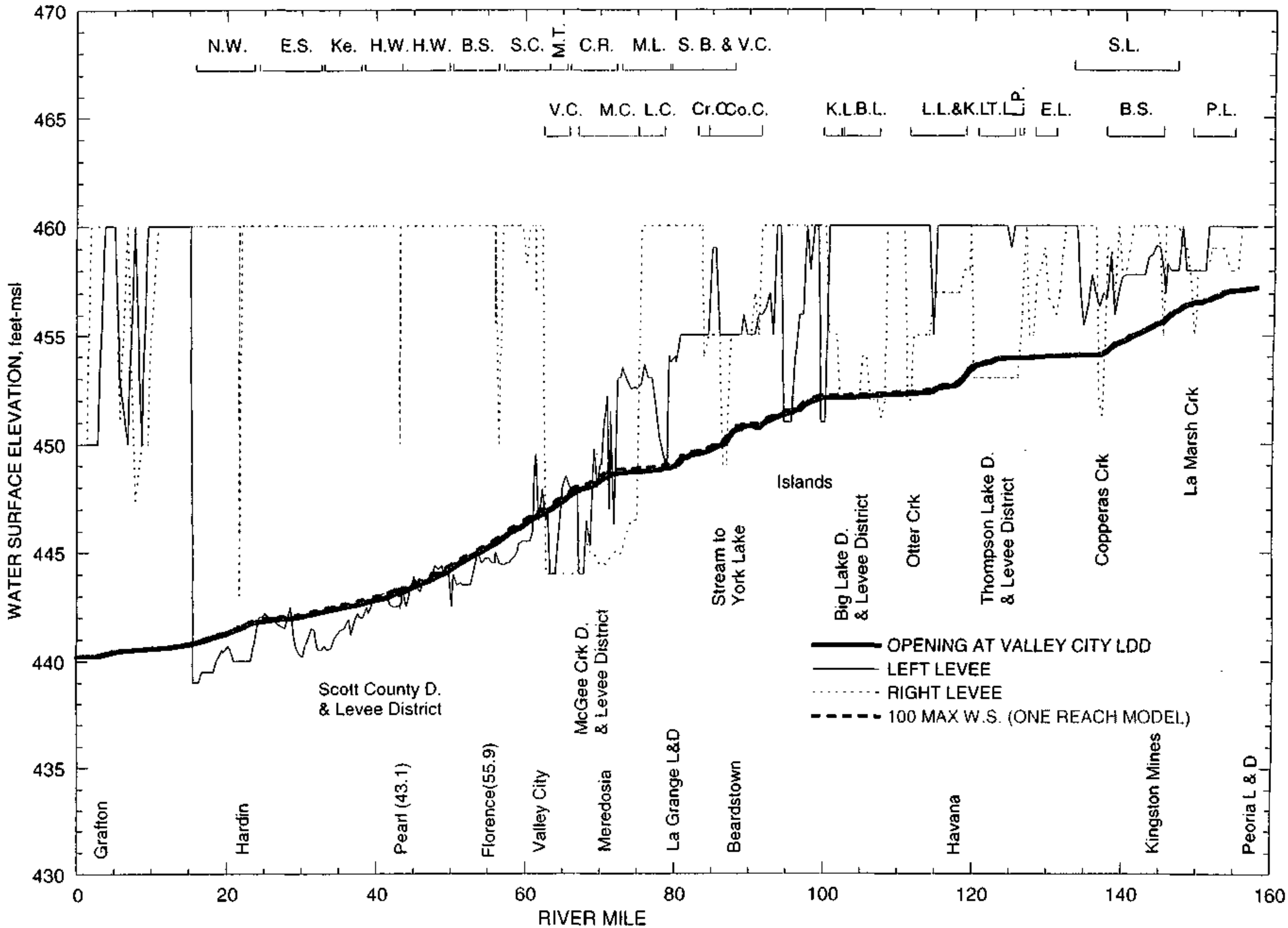
Peak WSE profiles with and without an opening at Crane Creek LDD



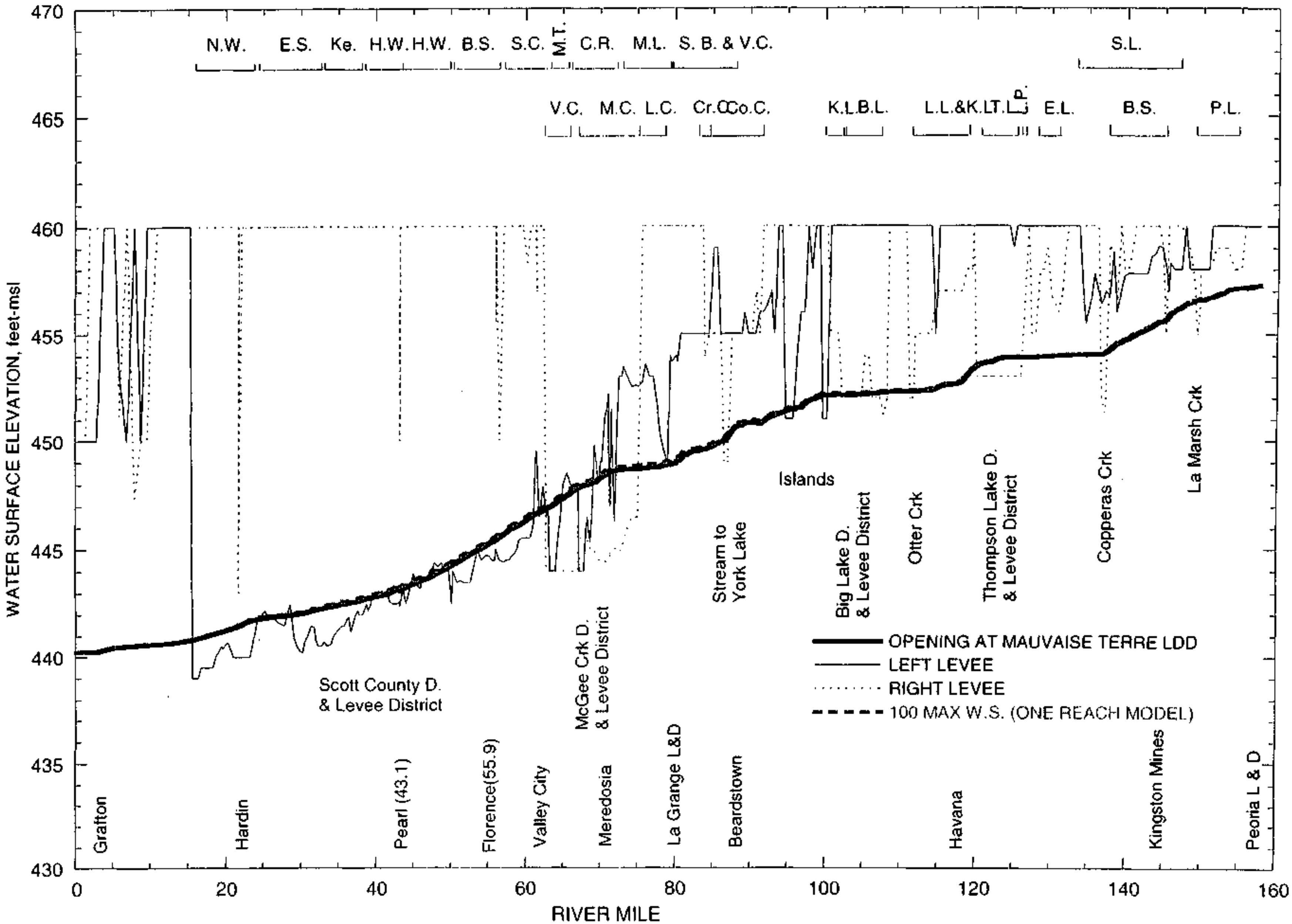
, Peak WSE profiles with and without an opening at Little Creek LDD



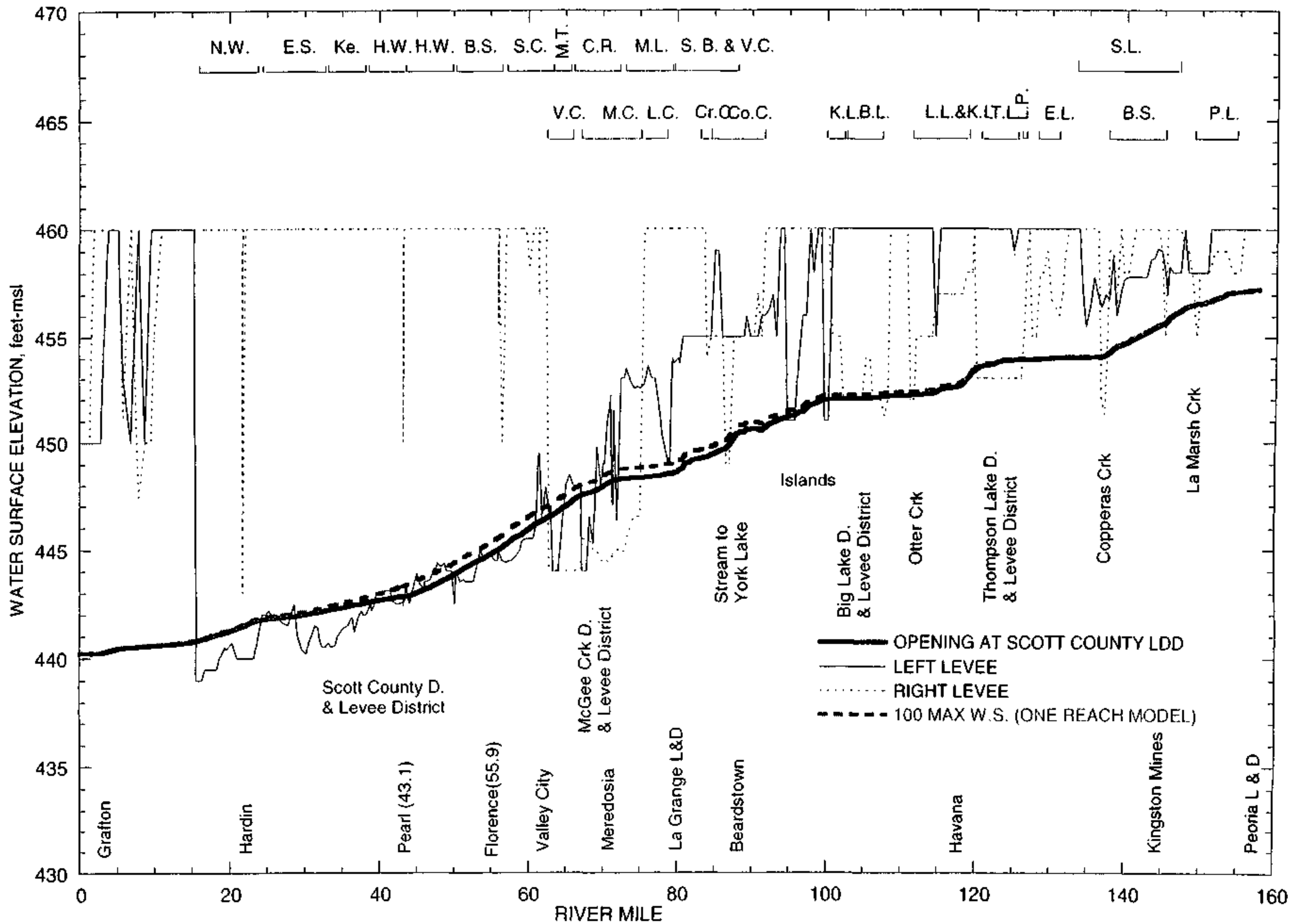
Peak WSE profiles with and without an opening at McGee LDD



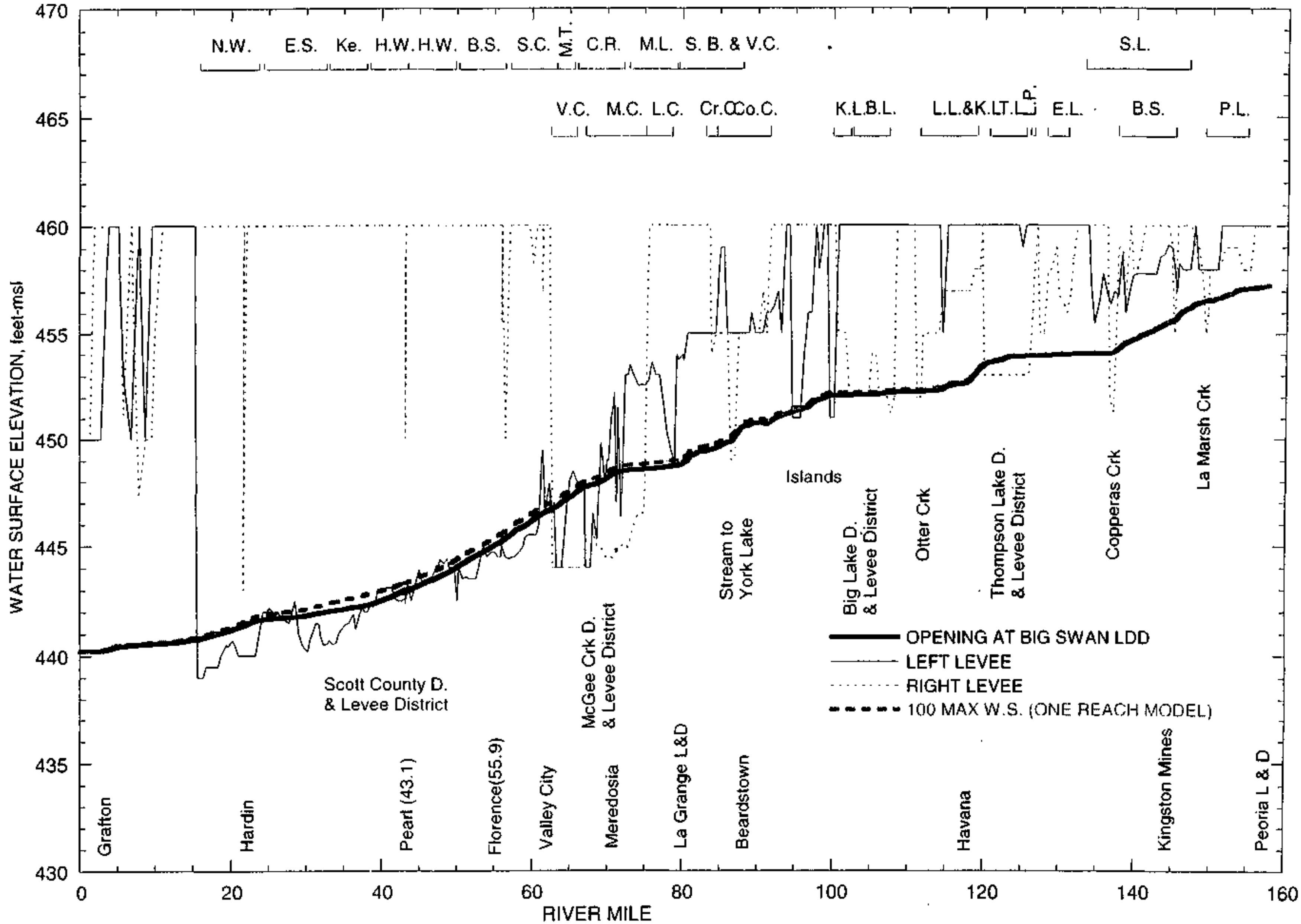
Peak WSE profiles with and without an opening at Valley City LDD



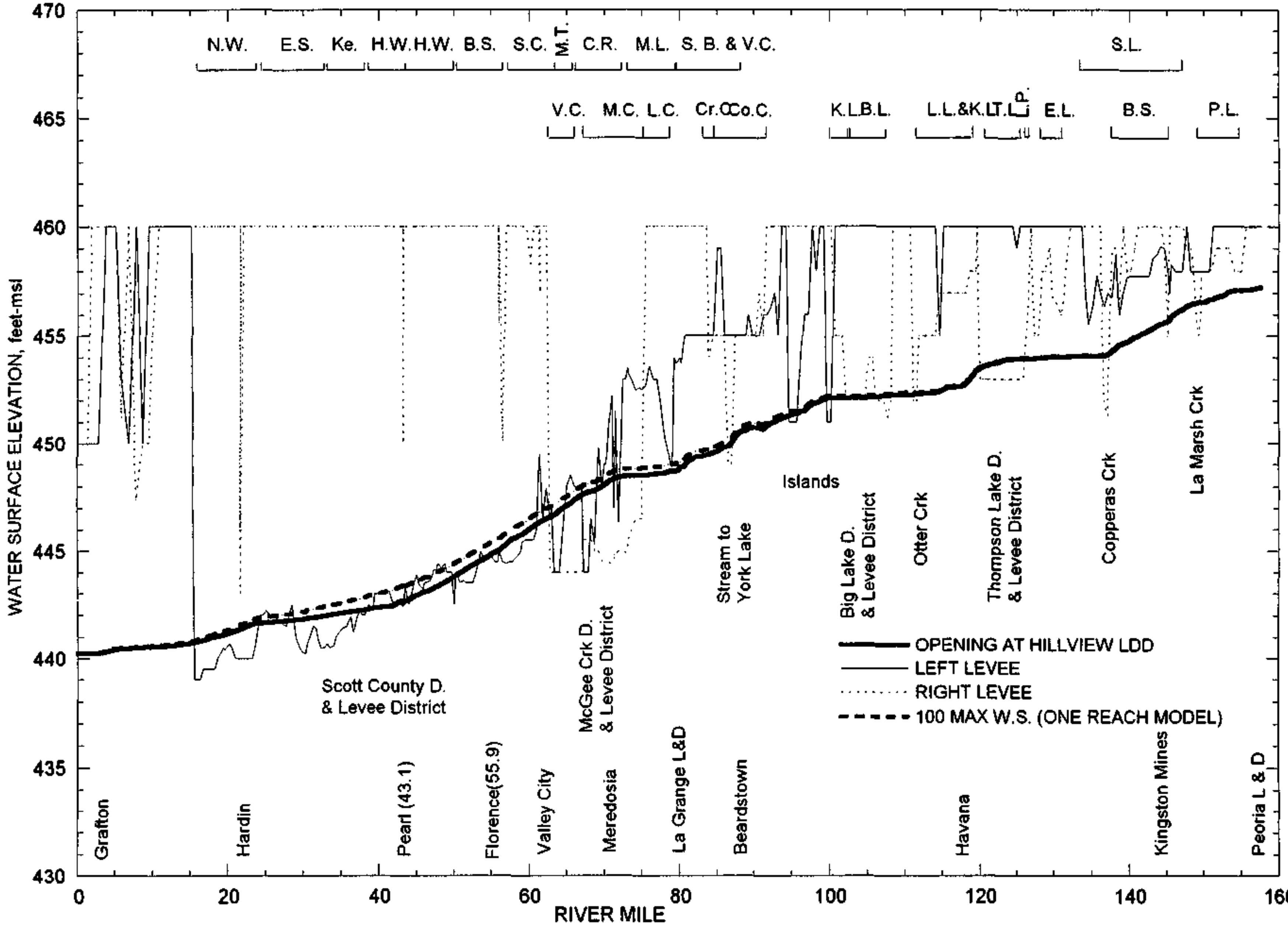
Peak WSE profiles with and without an opening at Mauvaise Terre LDD



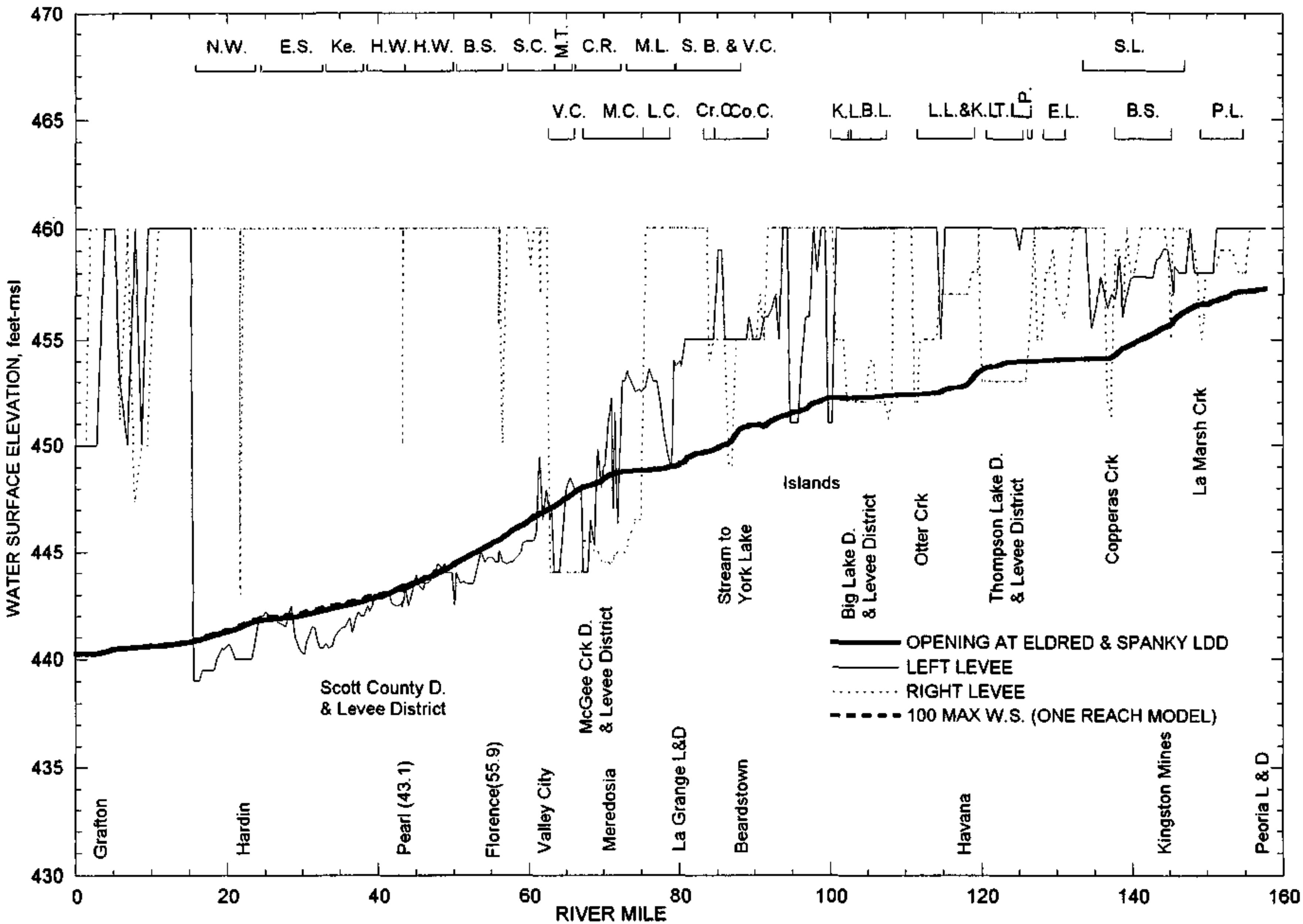
Peak WSE profiles with and without an opening at Scott County LDD



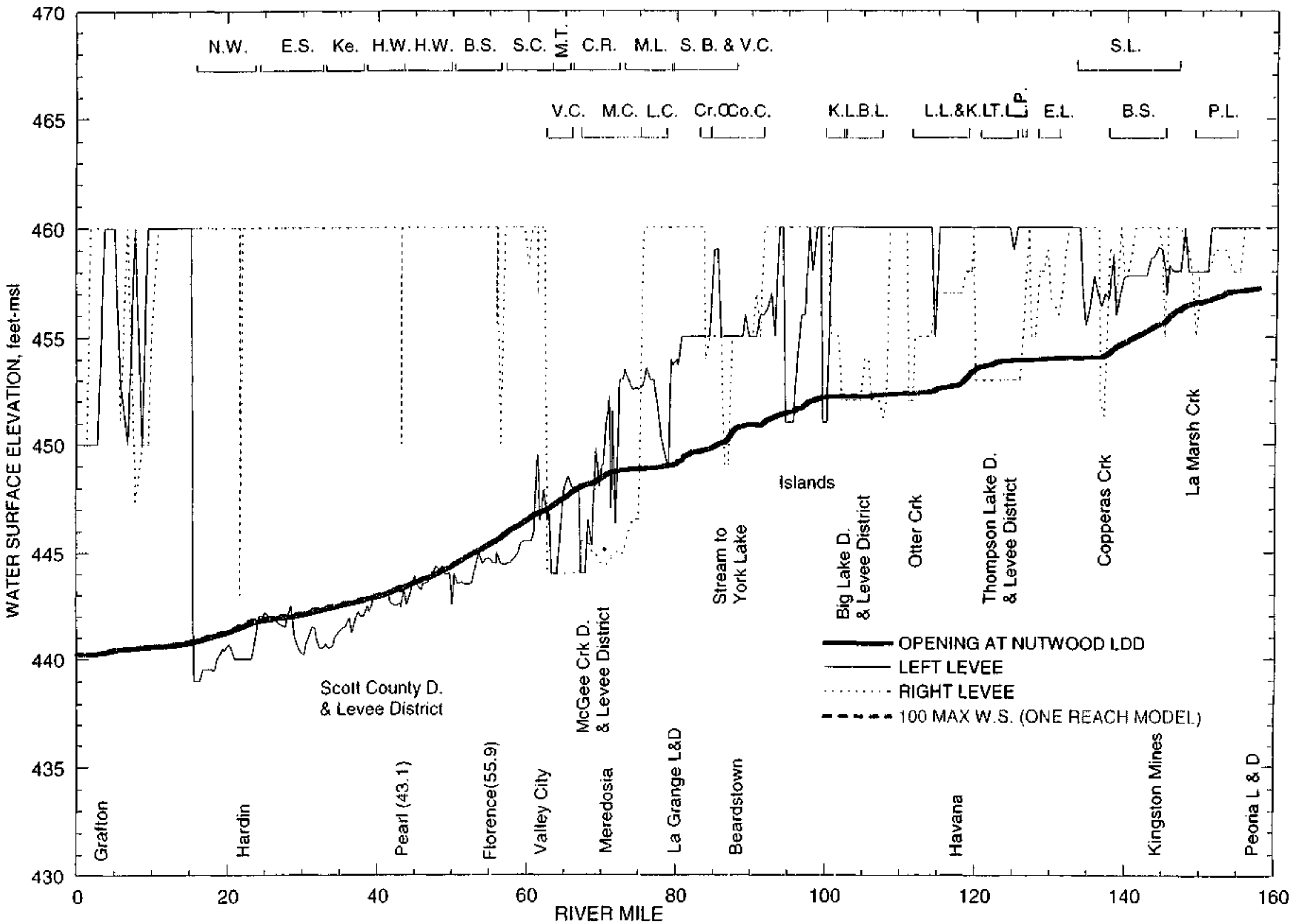
Peak WSE profiles with and without an opening at Big Swan LDD



Peak WSE profiles with and without an opening at Hillview LDD



Peak WSE profiles with and without an opening at Eldred & Spanky LDD



Peak WSE profiles with and without an opening at Nutwood LDD

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