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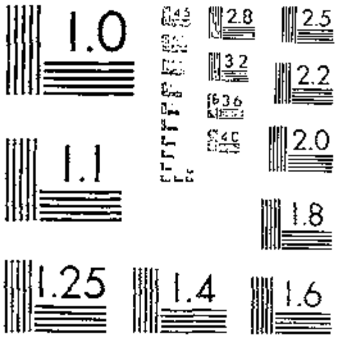

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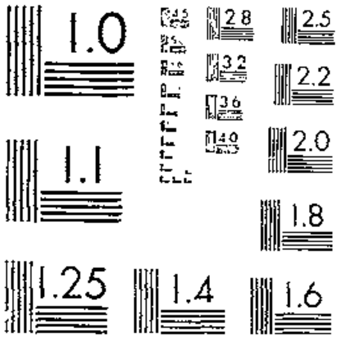

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Technical Bulletin No. 390 November 1933

DESIGN AND OPERATION OF DRAINAGE PUMPING PLANTS IN THE UPPER MISSISSIPPI VALLEY

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INTRODUCTION

A detailed investigation of the design and operating characteristics of pumping plants in the upper Mississippi Valley was made in 1925 to 1930. The investigation developed that the amount of water pumped annually from some drainage districts was more than double that from other all-pumping districts of like size in the same locality, because of differences in seepage from the streams bordering the districts. The amount of seepage was found to be related to the average static lift, to the porosity of subsoil strata, and to the extent that drainage ditches cut into those strata. The data collected show a correlation of plant capacity at maximum lift with average annual amount of pumping which has been expressed in a formula, with constants determined for the upper Mississippi Valley and suggestions as to the modification for application in other regions. Much operating and cost data gathered in that study have already been published (*4*).[1]

In 1930 there were 3,642,495 acres in organized drainage enterprises served by pumps in the United States which had a total capacity of 17,854,824 gallons per minute (*5*). The capacity of the engines and motors driving the pumps was 99,747 horsepower, of which 64.2 percent was electric, 9.8 percent steam, 13.2 percent internal combustion, and 12.8 percent combinations of these kinds of power. The cost of the drainage works of these enterprises was, up to December 31, 1929, $109,823,504.

To assist engineers in designing and operating more efficient low-lift drainage pumping plants this bulletin has been prepared, presenting (1) the elements to be considered in the design of such plants, (2) data for use in determining these elements in the upper Mississippi Valley, (3) a discussion of the selection and design of drainage pumping equipment, and (4) suggestions for the construction and operation of pumping plants.

The facts presented may be used for improving the operating efficiency of existing plants and for designing and constructing plants to replace existing equipment when it is worn out and cannot be operated economically. The low prices of agricultural products during the past few years do not encourage further reclamation by drainage pumping at this time. However, some existing districts may find it essential to install pumping plants to protect parts of the enterprise that could not otherwise be adequately drained, and thereby enable those areas to contribute their proper share to the expenses of those enterprises.

Acknowledgment is made of the cooperation rendered by the commissioners, engineers, attorneys, and plant operators of the several districts in obtaining the records of plant operation. Important data for this report were furnished by the following engineering firms: Bushnell & Bushnell, of Quincy, Ill.; Caldwell Engineering Co., of Jacksonville, Ill.; Central States Engineering Co., of Muscatine, Iowa; Goodell & Millard, of Beardstown, Ill.; Harmon Engineering Co., of Peoria, Ill.; McCann Engineering Co., of Quincy, Ill.; and Randolph-Perkins Engineering Co., of Chicago, Ill. No less important to the success and accuracy of the study have been the cost data supplied by the electric, oil, coal, and machinery companies.

CONDITIONS NECESSITATING PUMPING

The bottom lands along Illinois River below Peoria and along Mississippi River between Rock Island, Ill., and the mouth of the Illinois have been largely subject to overflow by river floods so frequently that farming them was unprofitable or impossible. The construction of dams for navigation and for power has aggravated the conditions in some sections. To protect the lands against high water, more than threescore drainage districts have been organized under State drainage laws, built levees to keep out the floods, and installed ditch systems and pumping plants to drain the lands. The extent of pumping for drainage in that region is shown in figure 1,

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[1] Italic numbers in parentheses refer to Literature Cited, p. 56.

FIGURE 1.—Pumping drainage districts on Illinois River below Peoria and on Mississippi River between Savanna, Ill., and St. Louis, Mo., 1930. (Districts identified by key numbers used in table 7, p. 57.)

and the major design elements of the pumping plants are given in the appendix (table 7).

Those drainage districts usually extend from the river bank to bluffs which rise abruptly to heights of from 50 to 200 feet above the river level. They usually extend along the Mississippi or the Illinois between tributaries which are too large to be diverted or pumped economically. Most districts are 2 to 4 miles in width and 4 to 8 miles in length. About two thirds of the area in the pumping districts is so low in relation to river stages that all drainage water must be pumped. The other third is at such elevations as to require pumping only part of the run-off, and during part of the time obtains gravity drainage through sluices.

RELATION OF PUMPING PLANT TO GENERAL PLAN OF DRAINAGE

Lands that require a pumping plant for drainage always require additional drainage works such as levees, interior ditches, frequently tile drains, and sometimes channels to divert hill streams. All of these works should be coordinated by the designing engineer in order to obtain an adequate and economical plan of drainage for the district. The diversion of hill streams is generally advisable even at considerable expense in order to reduce the costs of pumping and of maintaining the drainage ditches. Where the lands lie well above low-river stages, gravity drainage sometimes can be obtained by means of a sluiceway through the levee. On the other hand, where gravity drainage could be obtained only for occasional short intervals it may be more economical to pump all the drainage than to install the sluiceway.

LOCATION OF PUMPING PLANTS

The location of the pumping plant is determined largely by the topography of the district. It is desirable to install all the pumping equipment at one location if possible to secure lowest construction and maintenance costs. When this is impractical, electric power is more convenient than other types of power to operate small plants. Separate pumping plants are usually less economical for draining isolated areas than deep ditches even though such ditches are costly to maintain. Sometimes it is really impractical to drain all parts of a district to one plant, but division of an area to create a separate district is generally inadvisable. It increases the overhead expenses, some of which are almost as great for a small district as for a large one.

If some choice of location is possible, then the proximity to towns, good roads, railroads, or a navigable waterway may be the controlling factor. Such advantages are especially important for a steam- or an oil-engine plant, to decrease the costs of hauling fuel to the plant and to attract more skilled operators to work at the plant. Similarly, proximity to a power line is a factor in the location of an electric plant, although the cost of extending a power line is not usually sufficient to justify a large increase in the cost of the ditch system. When an old pumping plant is being replaced it is usually impracticable to change materially the location of the plant. When other conditions do not control, the best location may be determined by the pumping lifts and foundation conditions.

DETERMINATION OF STATIC LIFTS

Determination of the maximum, minimum, and average static lifts is a most important part of the design of a drainage pumping plant. The pump manufacturer needs these data in order to supply equipment that will operate efficiently through the controlling range in lifts and be adequate in capacity at the maximum lift. The importance of the variations in lift has not always been recognized. For example, one plant was designed for a head of 17 feet but no provision was made for efficient low-lift pumping although the average lift has been only 5 feet. When new impellers were installed in the pumps and the speed decreased, a saving in electric power was obtained estimated to average $2,000 per year.

The maximum, minimum, and average lifts depend on the operating stages of both the suction and the discharge bays. The elevation of land to be drained, the slope and operating levels of water in the interior ditches, and the elevations of tile outlets determine the operating levels of the suction bay. For this reason the static lifts usually can be determined best after the ditch system has been designed.

The maximum stage in the suction bay usually occurs when the pumps are unable to control the water and low areas of land are flooded. The minimum stage occurs when the ditches are pumped very low in order to extend the time until the pumps will need to be started again. In many cases it is desirable to hold the suction bay at a low stage in the spring in order to lower the water table in the ground. The desirable maximum operating level, or optimum stage of the suction bay, ordinarily is the level that will give drainage to the lowest important areas of cultivated land. The optimum stage may vary with season of year and with weather conditions. The ordinary fluctuations in operating level may depend largely on the amount of storage in the ditch system. Large storage in sloughs or lakes at normal operating levels of the suction bay is advantageous for several reasons. A run-off that temporarily exceeds the plant capacity will not raise the water in the bay and the ditches so much above the "optimum" stage (the stage at which it is desired to hold water in the suction bay), which results in better drainage for the lands of the district. Also, the water in the bay need not be pumped so much below the optimum stage to provide a convenient interval between pumpings.

Pumping the water low in the suction bay increases the static lift and tends to suck air into the pump, which decreases its efficiency. When a district has large storage a very small pump is not so essential for pumping during periods of small run-off. In some cases it may be desirable to leave some low sloughs or lakes unreclaimed, in order to provide storage capacity.

MAXIMUM LIFT

The maximum lift should be taken as the difference between the maximum stage of the discharge bay and the optimum stage of the suction bay. The maximum lift must be determined accurately, because if estimated too low the capacity of the plant may be inadequate during floods. The maximum lift may be determined from maximum recorded flood stages at nearby gages. Figures 2 and 3

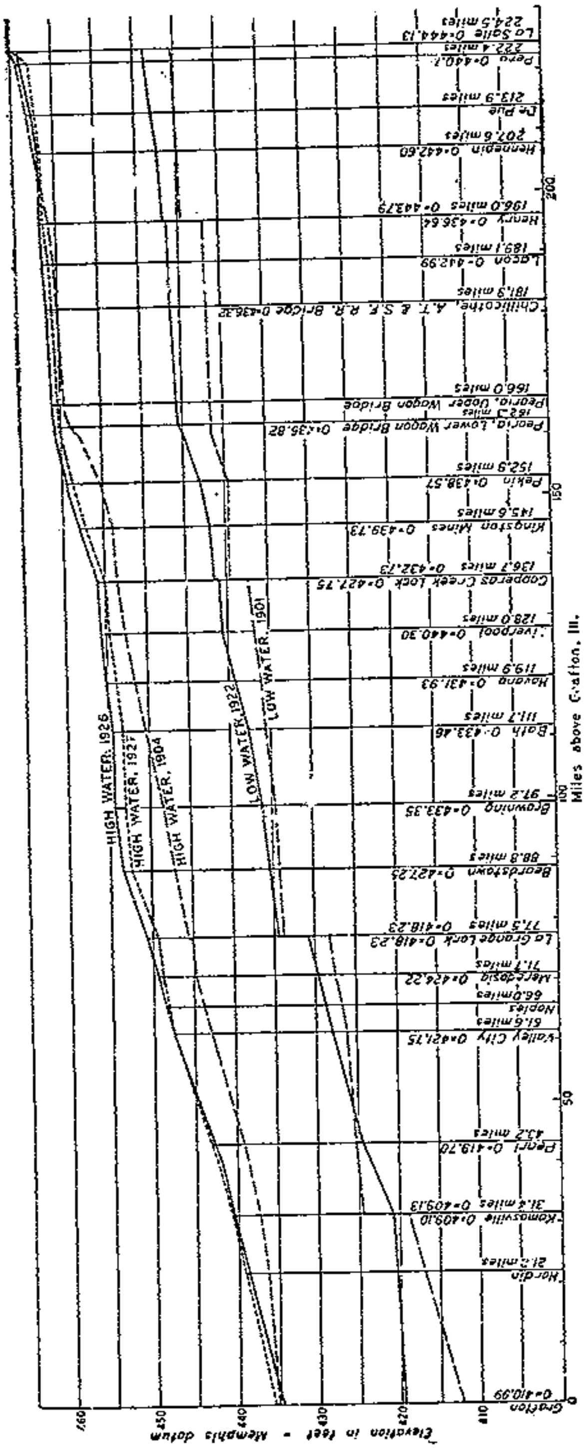


FIGURE 2.—Maximum and minimum recorded stages of Illinois River below La Salle.

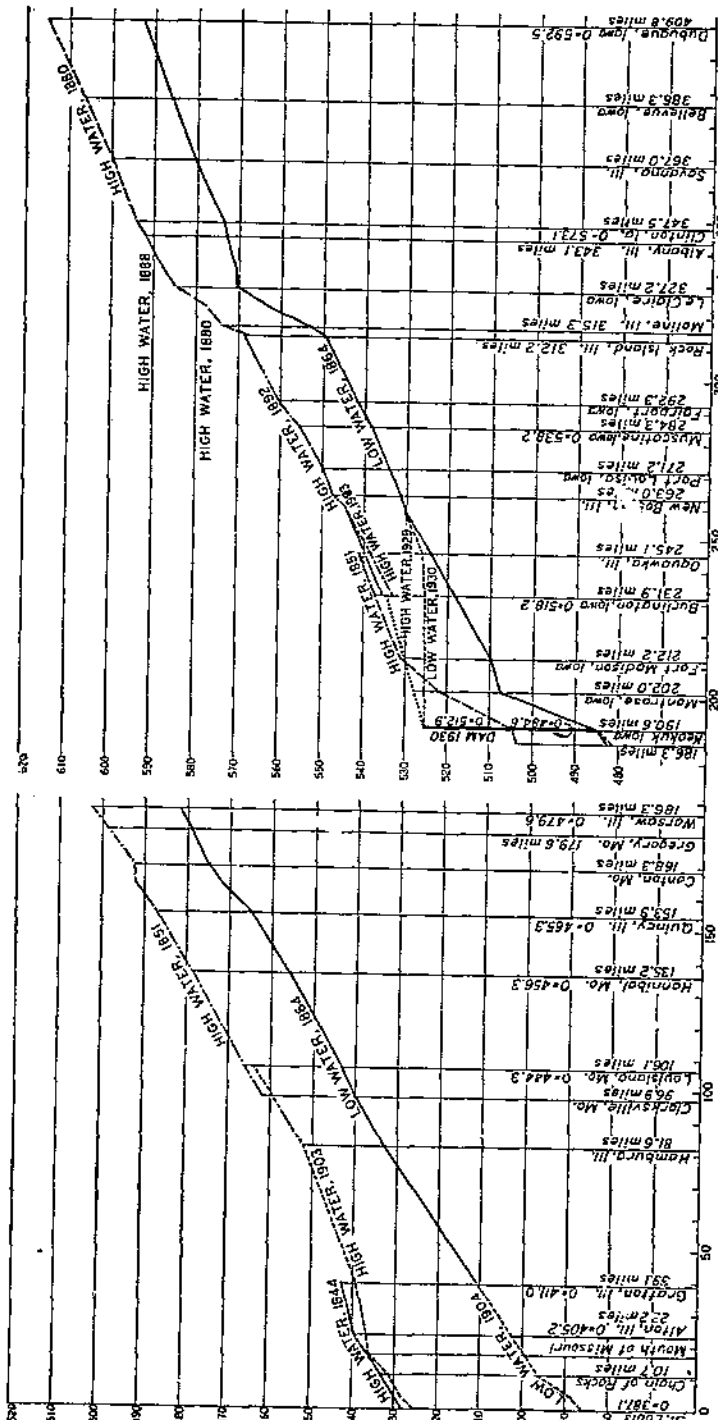


FIGURE 3.—Maximum and minimum recorded stages of Mississippi River between Dubuque and St. Louis.

show profiles of maximum and minimum stages of the Illinois and Mississippi Rivers. Those are valuable for estimating maximum and minimum river stages at points between the gaging stations. The stages of the discharge bay for a plant located on a tributary

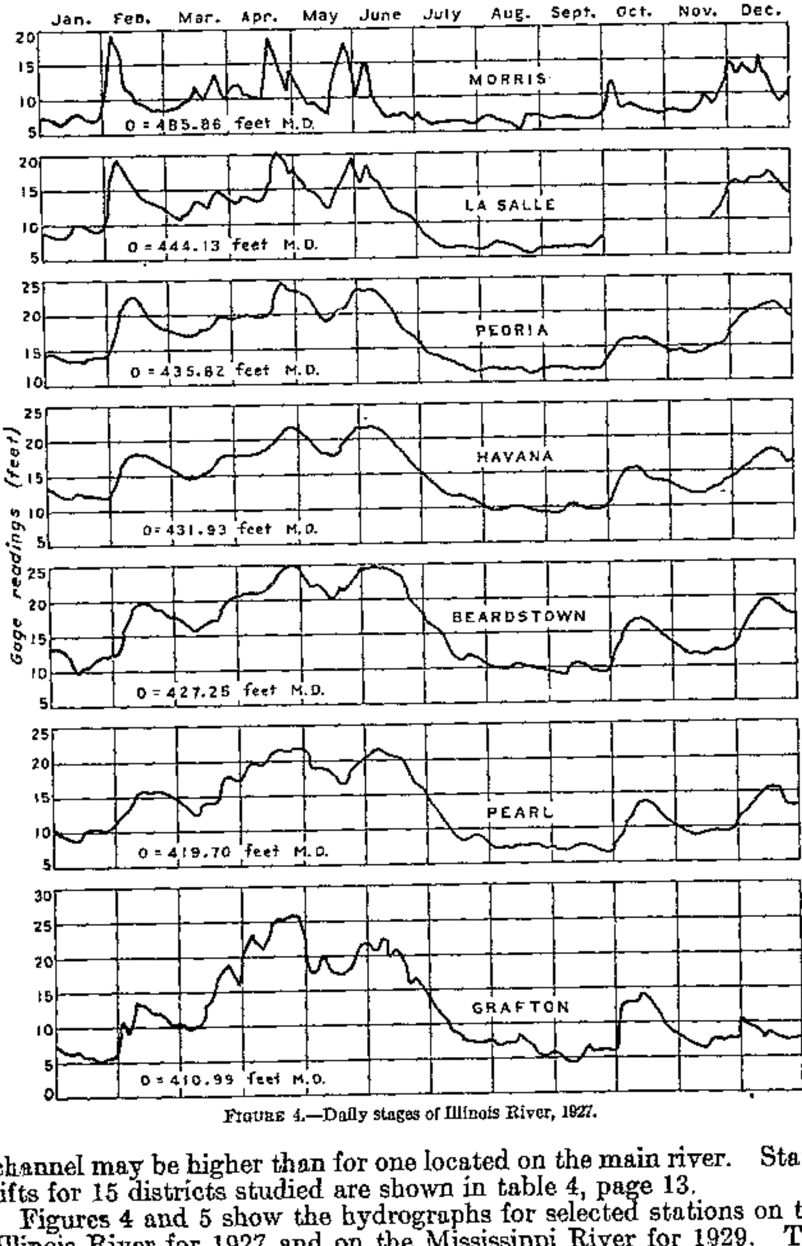


FIGURE 4.—Daily stages of Illinois River, 1927.

channel may be higher than for one located on the main river. Static lifts for 15 districts studied are shown in table 4, page 13.

Figures 4 and 5 show the hydrographs for selected stations on the Illinois River for 1927 and on the Mississippi River for 1929. The floods during those years are believed to represent maximum conditions that pumping plants along those rivers should be designed to meet, both as to height and as to duration of flood. More extreme conditions probably would require increase in plant capacity as well as in lift. Increasing the lift unnecessarily increases the power required to drive the pumps and, what is more important, tends to

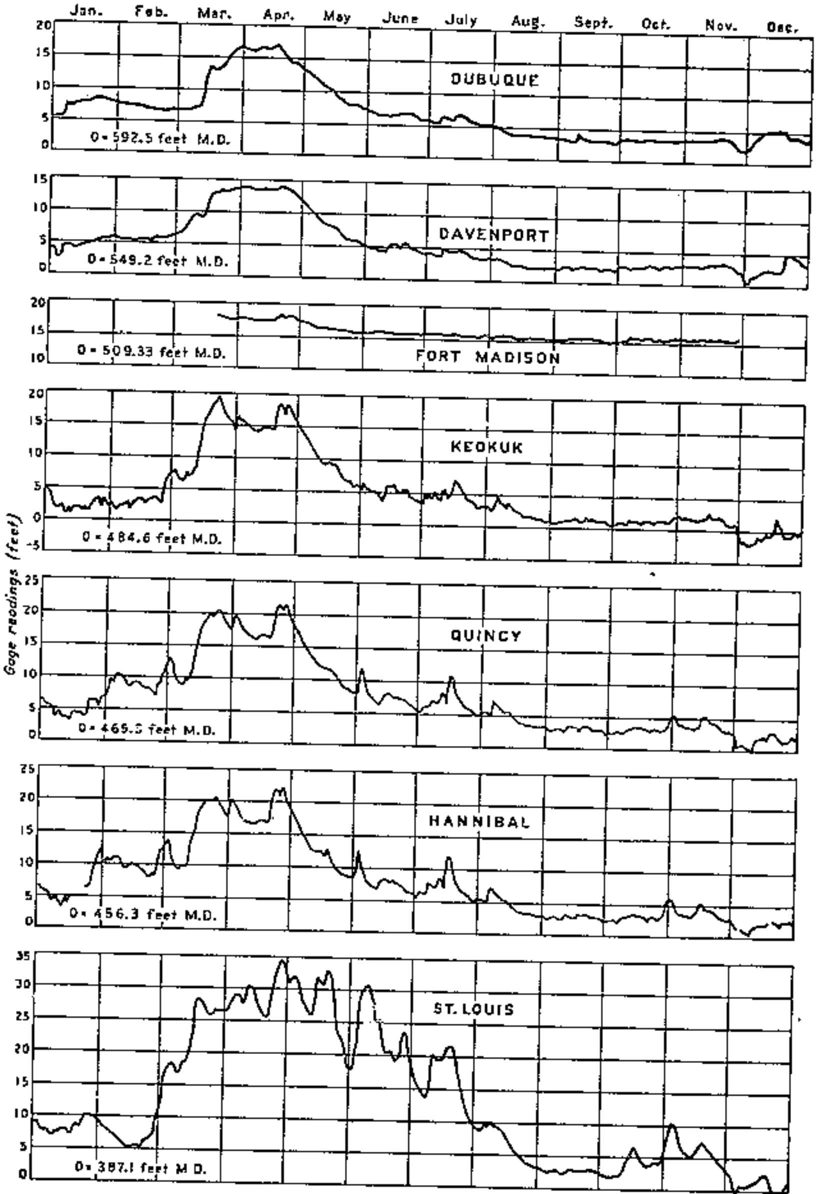


FIGURE 5.—Daily stages of Mississippi River between Dubuque and St. Louis, 1929.

decrease the plant efficiency at lower lifts. The maximum lift for plant design can be decreased to some extent where the river falls very rapidly after reaching a crest. It is usually impracticable to prevent flooding of some low-lying areas by the worst floods.

Study of operating conditions during flood periods (pp. 20 to 24) shows the importance of designing a plant to pump its full capacity at the maximum lift. Full-capacity pumping, as the figure shows, frequently must be begun several days before the maximum river stage occurs and be continued until the flood crest has passed. A considerable period of inundation might cause greater crop losses in a single year than the entire cost of an adequate pumping plant.

MINIMUM LIFT

The minimum lift should be assumed to be difference between the minimum stage of discharge bay and the optimum stage of suction bay. Illinois and upper Mississippi Rivers remain near minimum stage for longer periods than at any other stage. Therefore it is important to have at least one pumping unit that will operate efficiently near the minimum lift.

In many instances the minimum elevation of the discharge bay is above the minimum river stage because the bay is considerably distant from the river or obstructions intervene. The absolute minimum stage of the discharge bay occurs so infrequently that it is not necessary that the unit be designed to pump with highest efficiency against the extreme low lift. Average low-water stages such as given in table 1 are probably better for estimating minimum lifts. For instance, as 8.9 and 8.7 feet for August and September are the average minimum stages for Beardstown, it is desirable to use elevations corresponding to a stage of 9 feet in computing the minimum lift for plants in that vicinity. The minimum stages of the Mississippi occurring in December, January, and February should be disregarded because they occur at a season when very little pumping is necessary. Figures 2 and 3 also can be used in estimating minimum lifts for pumping plants along the upper Mississippi and Illinois Rivers.

TABLE 1.—*Average minimum stages[1] (feet) of Illinois and Mississippi Rivers, 1925 to 1930*

ILLINOIS RIVER

Station	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
Peoria, Ill.	13.6	14.2	15.6	16.0	14.0	12.5	11.7	10.8	10.9	12.0	11.9	13.2
Beardstown, Ill.	10.9	12.2	13.0	14.8	13.0	11.5	10.3	8.0	8.7	10.5	10.1	10.7
Pearl, Ill.	8.4	9.8	10.3	12.1	10.8	8.8	7.4	6.1	6.1	8.0	7.5	7.8

MISSISSIPPI RIVER

Station	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
Dubuque, Iowa	3.9	4.1	5.0	8.2	5.6	4.7	3.8	2.4	3.1	4.1	3.4	2.2
Davenport, Iowa	3.5	3.6	3.6	6.7	4.5	3.5	2.7	1.7	2.3	3.1	2.6	1.2
Keokuk, Iowa	−.5	.3	3.2	6.9	4.2	3.0	1.9	.8	1.2	2.4	1.9	−.9
Quincy, Ill.	2.2	3.5	5.2	8.3	5.8	4.5	3.4	2.1	2.5	3.7	3.6	.9
Hannibal, Mo.	2.8	4.2	5.5	8.6	6.2	4.7	3.6	2.2	2.7	3.8	3.9	1.3
St. Louis, Mo.	4.5	5.7	9.9	15.4	13.1	12.5	8.8	5.7	4.8	6.3	6.2	3.3

[1] Weather Bureau gage readings. See figs. 2 and 3 for elevations, Memphis datum, of zeros of gages.

AVERAGE LIFT

The average lift for use in designing a drainage pumping plant may be determined from the average monthly lifts weighted according to the amounts of run-off pumped in the respective months.

The average stages, annual and monthly, of the Mississippi and the Illinois at the principal cities in the region where drainage pumping is common are given in table 2. The average stages are highest along the Illinois in March, April, and May and along the Mississippi in April, May, and June. Maximum river stages occur somewhat later on the Mississippi because its drainage area lies farther north. From this table the average elevations of the discharge bay can be approximated, by interpolating between the stages shown according to water slopes indicated in figures 2 and 3. Consideration should be given, of course, to any other available data relating to water elevations at the particular location of the proposed pumping plant.

TABLE 2.—*Average stages*[1] *(feet) of Illinois and Mississippi Rivers*

ILLINOIS RIVER

Station	January	February	March	April	May	June	July	August	September	October	November	December	Annual	Period of record (inclusive years)
Peoria, Ill.	12.3	13.5	15.4	15.8	14.1	12.9	11.5	10.5	10.4	10.8	10.9	11.7	12.5	1900 to 1930
Beardstown, Ill.	10.9	11.9	13.5	14.6	13.0	11.9	10.5	9.8	9.5	10.0	9.7	10.4	11.3	[2]1900 to 1930
Pearl, Ill.	8.1	9.1	11.2	13.4	11.2	10.0	8.2	7.0	7.0	7.4	7.2	7.0	9.0	1900 to 1904 1917 to 1930

MISSISSIPPI RIVER

Station	January	February	March	April	May	June	July	August	September	October	November	December	Annual	Period of record (inclusive years)
Davenport, Iowa	4.2	4.9	5.7	7.9	7.8	6.0	5.4	3.3	3.2	3.7	3.4	2.8	4.9	1872 to 1930
Keokuk, Iowa	2.9	4.0	6.3	8.7	8.6	7.8	6.9	3.6	3.3	3.6	3.3	2.2	5.0	[3]1868 to 1930
Quincy, Ill.	3.6	4.7	7.1	10.5	9.0	8.5	6.0	4.3	4.2	4.6	4.1	2.9	5.8	1911 to 1930
Hannibal, Mo.	3.7	5.2	7.6	10.8	9.5	9.3	7.1	4.8	4.4	4.7	4.3	3.2	6.2	[3]1905 to 1930
St. Louis, Mo.	6.9	9.1	13.9	18.6	18.4	19.5	17.3	11.2	9.0	8.3	7.8	5.9	12.2	[3]1861 to 1930

[1] Weather Bureau gage readings. See figs. 2 and 3 for elevations of zeros of gages (Memphis datum).
[2] Records incomplete for 1905 to 1915.
[3] Except when river was frozen.

The average monthly lifts can be estimated from the average elevation of water in the suction bay and the average monthly elevations in the discharge bay. The average amount of pumping done monthly should be estimated as explained under Distribution of Run-off, page 16. The annual average lift determined according to the amounts of pumping done in different months will be more nearly correct than if taken as the difference between the annual average stages of the suction and discharge bays because more pumping is done when the river is high than when it is low.

DETERMINATION OF RUN-OFF

The run-off from a pumping district often includes, in addition to the surface flow, a large amount of seepage from nearby hill lands and from bordering rivers or creeks.

The rate of surface run-off depends on the distribution of rainfall, other climatological conditions, the size and shape of watershed, the ground slopes, the vegetal cover, and the character of soil. The rate of seepage from hill lands is affected by the same conditions but especially by soil and subsoil characteristics. The rate of seepage from adjacent bodies of water depends especially upon the difference in elevation of the water inside and outside the drainage district, the extent of water-bearing gravel and sand under the district, and the length and location of drains touching the water-bearing strata. Because of the large number of influencing factors, the amounts and rates of run-off to be pumped can best be estimated from comparisons with similar districts from which the run-off has been measured. A large amount of such data are available for the districts in the upper Mississippi Valley (*4*, *6*) and will be discussed in the following pages. Considerable data are available also for districts in Louisiana (*1*, *2*).

For the design of a drainage pumping plant careful estimates should be made of (1) the average yearly run-off ,(2) the distribution of that run-off, (3) the maximum daily run-off, and (4) the minimum daily run-off. The yearly run-off must be known in order to estimate the annual cost of pumping and determine the feasibility of the project. The seasonal distribution of the run-off as well as the lifts must be known in order to design an efficient pumping plant. The maximum daily run-off determines the total capacity of the plant, and the minimum run-off is important in determining the size of the smallest pumping unit.

RAINFALL IN THE UPPER MISSISSIPPI VALLEY

The normal annual rainfall at 14 stations on Illinois and upper Mississippi Rivers, as determined by the Weather Bureau varies from 32.14 inches at Davenport, Iowa, to 37.44 inches at St. Louis, Mo., and the normal monthly precipitation from 1.03 to 4.95 inches, as shown in table 3.

TABLE 3.—*Normal monthly and annual precipitation (inches) along Illinois and Mississippi Rivers* [1]

Station	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
ILLINOIS RIVER													
Peoria, Ill	1.78	2.01	2.73	3.38	4.06	3.77	3.58	3.12	4.03	2.29	2.37	1.77	34.89
Havana, Ill	2.03	1.71	2.85	3.21	3.70	3.82	3.82	2.97	3.97	2.09	2.06	1.87	34.10
Beardstown, Ill	1.63	1.62	3.15	3.32	4.21	3.56	3.00	3.59	3.92	2.67	1.89	1.65	34.01
Pearl, Ill	1.03	1.04	3.47	4.25	3.35	3.19	2.20	3.25	4.95	3.93	2.25	1.75	34.06
Grafton, Ill	2.22	1.97	3.39	3.77	3.94	3.80	3.23	3.50	3.75	2.79	2.39	2.43	37.18
MISSISSIPPI RIVER													
Dubuque, Iowa	1.30	1.38	2.03	2.85	4.22	4.31	3.94	3.24	4.01	2.48	1.70	1.44	32.00
Davenport, Iowa	1.42	1.58	2.31	2.69	3.94	4.11	3.33	3.40	3.58	2.39	1.82	1.48	32.14
Muscatine, Iowa	1.84	1.92	2.60	3.28	4.42	4.43	3.71	4.14	3.60	2.65	2.30	1.96	36.85
Burlington, Iowa	1.70	1.81	2.51	3.10	4.53	4.29	3.41	3.73	3.92	2.28	1.86	1.60	34.80
Keokuk, Iowa	1.56	1.55	2.38	2.99	3.93	4.13	3.41	3.20	3.84	2.26	1.94	1.45	32.64
Quincy, Ill	1.68	1.57	2.35	3.10	4.22	4.11	3.47	3.13	4.68	1.96	1.87	1.51	33.65
Hannibal, Mo	1.72	1.86	2.00	3.19	4.35	3.90	3.00	3.34	3.97	2.39	2.06	1.54	34.01
Louisiana, Mo	2.02	2.35	2.90	3.38	4.56	3.95	3.12	3.73	3.95	2.63	2.28	1.69	36.56
St. Louis, Mo	2.34	2.56	3.38	3.81	4.34	3.82	2.98	2.99	3.46	2.72	2.83	2.21	37.44
Average for 14 stations	1.74	1.77	2.76	3.31	4.13	3.94	3.31	3.39	3.97	2.63	2.12	1.74	34.71

[1] U.S. Weather Bureau records.

The maximum 24-hour rainfalls that have been recorded at those stations range from 4.80 inches at Keokuk to 7.02 inches at St. Louis. The maximum rainfalls have occurred during the growing season when evaporation and transpiration are large and the ground is likely to absorb a large portion of a heavy rainfall. Rains exceeding 5 inches in 24 hours have not been recorded at any of the stations during October to April, inclusive. A total rainfall of 15.56 inches recorded at the Scott County plant from August 30 to September 9, 1926, started one of the highest and most prolonged floods known on Illinois River. Recorded precipitations were 5.18 inches on September 4 and 5.95 inches on September 8.

ANNUAL RUN-OFF

The best way of determining the average annual run-off from districts where no records are available is by comparison with districts whose topographical, soil, drainage, and static lift conditions are similar. The estimated average annual run-off for certain districts in the upper Mississippi Valley, with the static lifts and the amounts that backwater from power and navigation dams have increased the minimum lifts, are shown in table 4.

TABLE 4.—*Estimated annual run-off to be pumped, static lifts, and backwater effects for typical districts*

Key No.[1]	District	Average run-off depth pumped[2]	Static lift: Average annual	Static lift: Maximum	Static lift: Minimum monthly	Backwater effect[3]
		Inches	*Feet*	*Feet*	*Feet*	*Feet*
GROUP 1.—ILLINOIS RIVER DISTRICTS PUMPING CONSIDERABLE SEEPAGE						
22	South Beardstown	35	14	24	9	10
18	Coal Creek	24	12	22	8	10
35	Eldred	20	7	18	1	[4] 11
19	Crane Creek	16	11	21	7	10
GROUP 2. ILLINOIS RIVER DISTRICTS PUMPING LITTLE SEEPAGE						
28	Valley City	16	11	20	6	6
6	Banner Special	15	12	23	8	9
30	Scott County	14	12	22	7	6
27	McGee Creek	14	10	20	4	5
33	Hartwell	14	11	18	4	9
GROUP 3. MISSISSIPPI RIVER DISTRICTS PUMPING CONSIDERABLE SEEPAGE						
52	Green Bay	32	11	13	10	10
61	Henderson County	27	7	13	4	6
GROUP 4. MISSISSIPPI RIVER DISTRICTS PUMPING LITTLE SEEPAGE						
43	Bay Island	15	7	17	1	0
57	Indian Grave	12	5	16	1	0
GROUP 5. DISTRICTS HAVING SOME GRAVITY DRAINAGE						
59	Fabius	7	5	13	1	0
29	Mauvaisterre	5	4	12	1	6

[1] For identification in figures 1 and 10 and table 7.
[2] The figure for Coal Creek district is the average for 16 years; the figures for South Beardstown, Green Bay, Banner Special, Hartwell, and Indian Grave districts were determined from long records of power used and records of pumping for shorter periods; the estimates for the other districts then were based largely on comparisons of amounts of pumping recorded by those districts with the amounts by the districts just named (*4, p. 81*).
[3] Height that minimum river stages have been raised by dams.
[4] Along upper part of district only. Backwater increases seepage, but does not affect lift.

The average annual lifts were estimated from records of lift published elsewhere (*4, p. 81*), long-time records of river stages, the distribution of run-off and the optimum suction-bay stages. The maximum lifts were determined from river-stage records and optimum suction-bay stages. The minimum lifts represent the least monthly average recorded with the suction bay at optimum stage. The amount that the minimum river stages were raised by backwater from dams was determined from profiles of low water before and after the dams were installed in the rivers.

The districts have been grouped according to the amount that backwater from dams increased the run-off to be pumped. Districts having an average annual run-off of 20 inches or greater evidently were materially affected by backwater. This represents an increase of at least 6 to 8 inches in annual run-off due to the dams.

The Crane Creek district has been included in the same group because its run-off has also been increased greatly due to the dam. A large proportion of the district is relatively high, and if it were not for the dam, this district could obtain gravity drainage a large part of the time and would pump probably 8 inches less depth than now is necessary. The Valley City district, on the other hand, is so low it could obtain very little gravity drainage even if not affected by backwater, and existence of the dam probably does not increase the average run-off to be pumped by more than 2 or 3 inches per year.

The estimated average annual run-off of districts of group 1, the districts that pump a considerable amount of seepage from Illinois River, ranged from 16 to 35 inches depth over the watershed area. The South Beardstown, Crane Creek, and Coal Creek districts are above the dam at La Grange, Ill. Only the upper part of the Eldred district is affected by backwater, but the run-off pumped is 20 inches per year. The lifts are not affected by backwater because the pumping plant is below the dam.

The average run-off pumped from the Illinois River districts of group 2, (table 4) varied only from 14 to 16 inches. Apparently there was little seepage into these districts and the run-off was little affected by backwater from dams except that no gravity drainage could be obtained. Although the dams increased the low-water stages at these districts from 5 to 9 feet, part of this increase was without effect because the low-water stages of the Illinois prior to the time that the dams were put in, were, in some cases, below the normal stages of the suction bay. The districts of group 2 have tight soils which reduce seepage to a minimum, and the records indicate that backwater from dams may be of little importance if the subsoils are sufficiently tight.

Group 3 comprises two districts on the Mississippi affected by backwater from the Keokuk dam. The dam holds the water at almost constant elevation at the Green Bay district, where the minimum lift is 10 feet and the maximum 13 feet.

Group 4 comprises two Mississippi River districts not affected by backwater from dams, but which obtain no gravity drainage. The run-off from these districts varies from 12 to 15 inches, which is considered typical for districts in this territory that have to pump all run-off but into which there is little seepage.

Group 5 comprises two districts having some gravity drainage which pumped average run-off depths of 5 to 7 inches.

These data indicate that the run-off from districts not affected greatly by backwater from dams ranged from 12 to 16 inches, whereas the districts that were affected by seepage pumped from 16 to 35 inches. The difference in amount of seepage at equal lifts is largely the result of differences in porosity of subsoils and the extent to which the drainage ditches cut into the porous strata.

The ratio of pumped run-off to rainfall increases with the static lift, due partly to greater seepage, and partly to the natural occurrence of high average river stages in the years of large precipitation and consequent high ratio of surface run-off to rainfall. Figure 6 shows how this ratio of pumped run-off to rainfall increased with the static lift for those districts for which adequate records were available. The differences in the position of the curves doubtless are largely the result of differences in amounts of seepage.

The Henderson County district had a very high ratio of run-off to rainfall although its average lift ranged only from 5 to 8 feet. It is

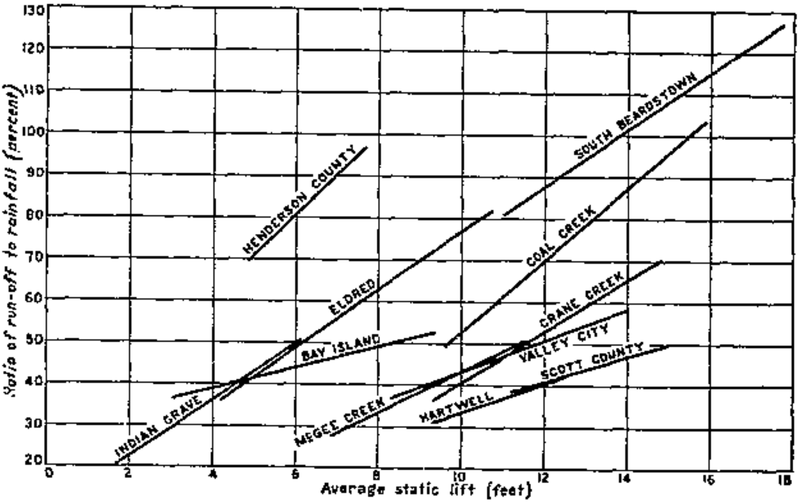


FIGURE 6.—Observed relation between the ratio of pumped run-off to rainfall and the static lift for certain drainage districts.

almost entirely underlain by water-bearing sand strata which are tapped by the drainage ditches and form the bottom of a lake of about 20 acres at the pumping plant. The ratio of river frontage to area protected also is high. About half the South Beardstown district consists of low lake beds that are underlain by sand strata and badly seeped during river floods. Much of the remainder of this district is low but has tight subsoil. The Eldred and the Indian Grave districts are high with respect to the river and contain coarser soils than the South Beardstown. Probably half of each shows seepage and the remainder of each district is too high to be so affected. The flatter slope of the Bay Island district curve is believed due to the fact that half the area drained is hill land which is not affected by seepage, and the remainder is similar in character and situation to the Indian Grave district. Approximately one fifth of the Coal Creek district may be classed as seeped, but the remainder has tight subsoil. The other five districts represented in figure 6 have less than 10 percent of their areas badly affected by seepage. The greater portions of the Crane Creek, McGee Creek, Hartwell, and Scott County districts are high in relation to river stages. The Valley City district, like the Banner Special district which is not shown in that figure, has extensive low-lying lake beds but the drainage ditches are largely in silt or clay soil and seepage is not great.

The Coal Creek and the South Beardstown data are the best available for districts that drain low lake beds. The average run-off from the Coal Creek district for 16 years was about 70 percent of the rainfall, at an average lift of about 12 feet. The South Beardstown plant operates against unusually high lifts, and a large part of the district is composed of a low lake bed underlain by sand strata, conditions that are conducive to large seepage. The average depth of run-off pumped by the South Beardstown plant in the 6 years, 1925 to 1930, was 7 percent greater than the rainfall, and the average lift was 15 feet. The estimated annual run-off is 35 inches, at an average lift of 14 feet (table 4). The amount of gravity drainage from the Eldred district is so small (*4*) that it does not materially affect the accuracy of the curve shown in figure 6.

The run-off to be pumped from a district that will obtain some gravity drainage can best be estimated from records of similar districts, considering the effects of differences in lifts and in porosity of subsoils. Estimates can be made also from records of low-lift districts such as Indian Grave or Bay Island, deducting the probable run-off during months when gravity drainage is expected.

DISTRIBUTION OF RUN-OFF

The principal factors that cause seasonal variations in amount of run-off are (1) variations in amount of rainfall, (2) variations in seepage due to fluctuation of river stage, (3) transpiration by plants, (4) evaporation, and (5) degree of saturation of the ground. The combined effect of these factors upon the average monthly run-off pumped by a number of typical districts is shown in figures 7 to 9 with the average monthly rainfall and static lift. The run-off usually varied more nearly with the fluctuations in lift than with the variations in rainfall, both because the seepage increases as the lift increases and because generally the run-off from the district and that from the watershed of the river are influenced by the same weather conditions.

Plants that pump large quantities of seepage, such as the South Beardstown, Henderson County, and Coal Creek which operate against lifts of 6 to 8 feet during low stages of the river, usually pump more or less continuously during the dry months. The dry-season run-off was comparatively small from the other districts shown in figures 7 to 9.

Run-off distribution for a proposed plant will be similar to that of nearby plants on the same river, though in making comparisons the effects of differences in static lifts, in soils, and in plans of drainage should not be disregarded. The difference in static lifts will be fairly uniform throughout the year. It can be estimated by comparing lifts for the existing and the proposed plants through a period of several days.

If the run-off distribution at an existing electric pumping plant is not known, that can be determined with fair accuracy from the power-consumption records. The power consumption in kilowatt-hours per acre-foot pumped for various lifts can be determined from tests made, or can be approximated from the pump manufacturer's rating curves with suitable allowance for reduction in pump efficiency

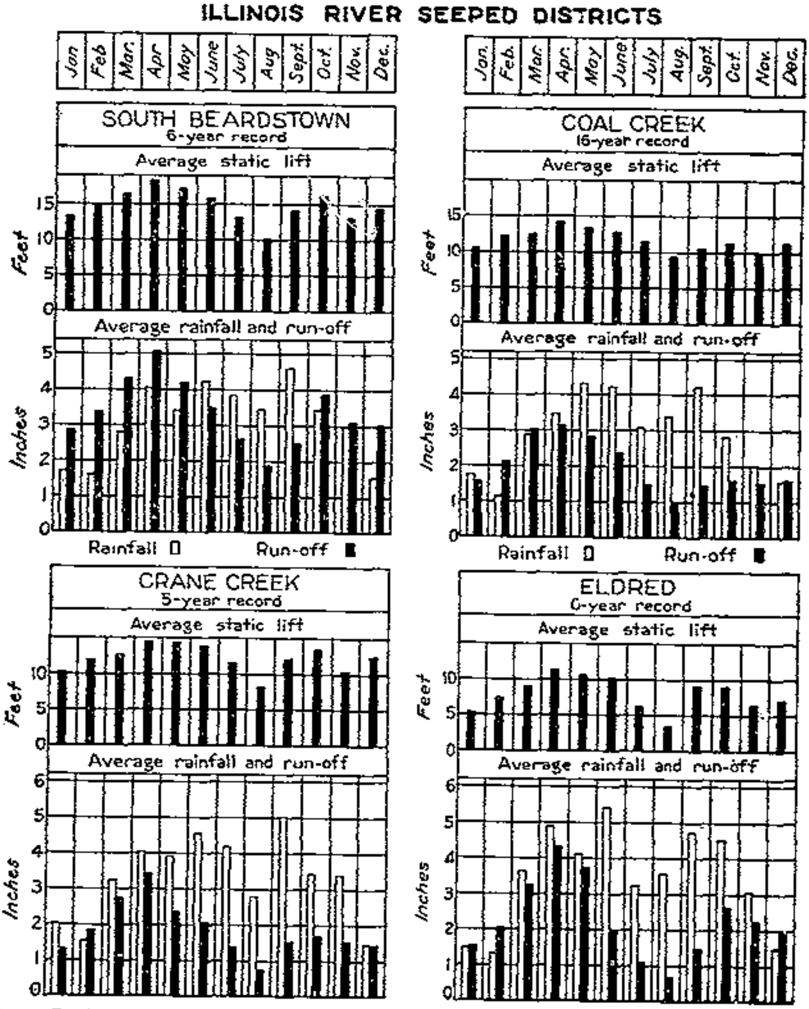


FIGURE 7.—Average operating conditions for typical districts on Illinois River pumping much seepage.

on account of age and condition of equipment. Power consumption in indicated horsepower-hours per acre-foot by a number of plants has been published (*4, p. 81*). From the power consumption and average lift for each month, and the power consumption per acre-foot pumped at that lift, the amount that was pumped each month can be calculated.

MAXIMUM DAILY RUN-OFF

The first drainage pumping plants along the Illinois and upper Mississippi were almost invariably of inadequate capacity. The plants built since about 1910 have been larger, with capacities usually of one fourth to one half inch run-off depth per 24 hours. A few plants apparently have unnecessarily large capacity, but about half

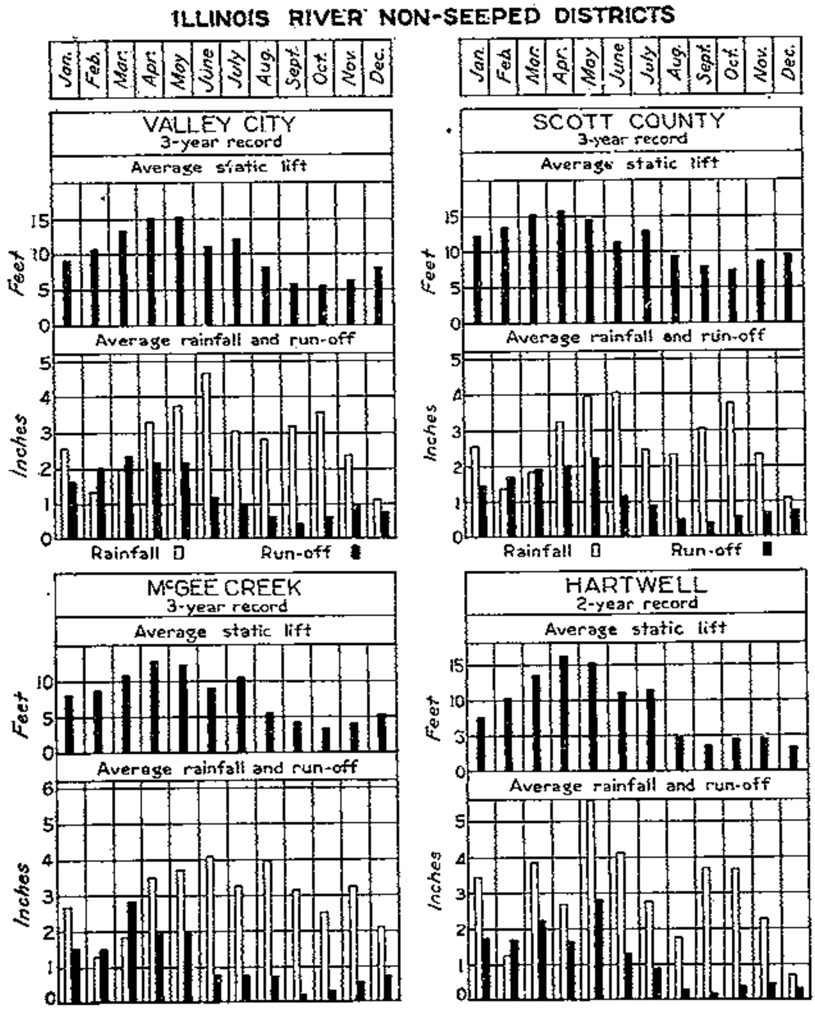


FIGURE 8.—Average operating conditions for typical districts on Illinois River pumping little see page.

of those studied were inadequate to hold down the water properly in the suction bay and prevent heavy crop losses during the worst storms. However, an economic limit to size of plant is reached when provision of additional capacity costs more than the damage to be prevented. Proper determination of the maximum rate of run-off to be pumped is the most important problem in designing a plant.

A proposed plant must be large enough to provide adequate drainage, yet unnecessary capacity is a waste of money because the first cost of a plant is almost proportional to its size.

Table 5 gives the design factors actually used in designing the pumping plants covered in this investigation, the design factors

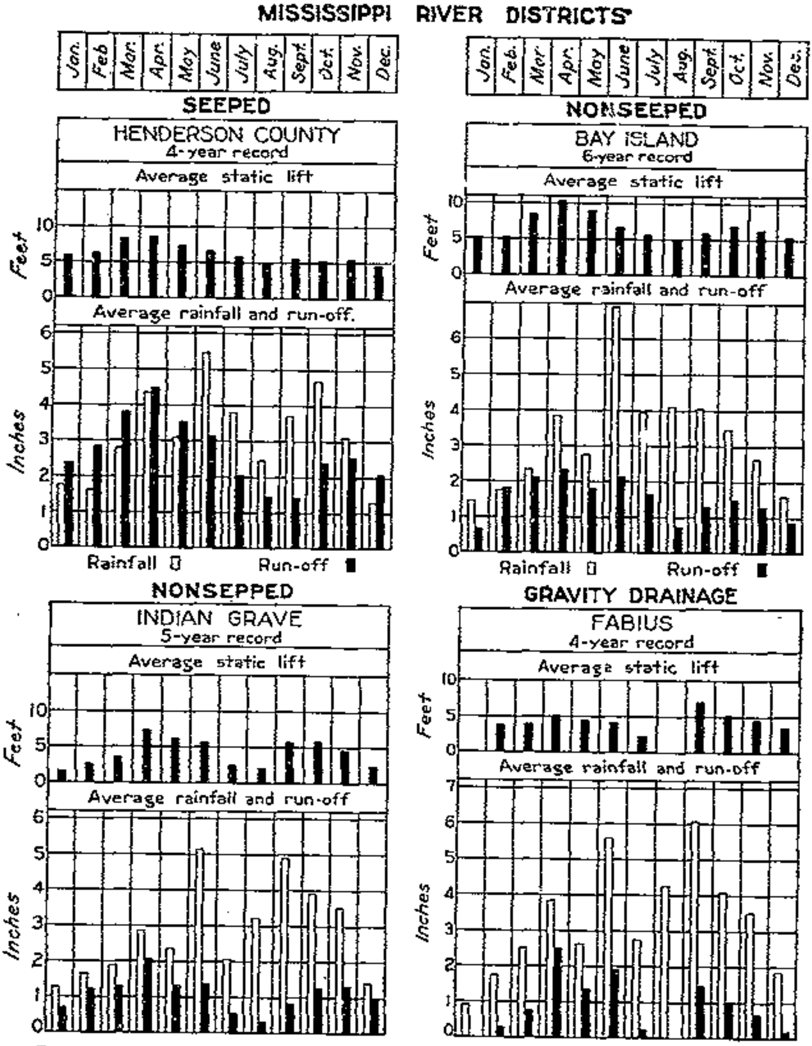


FIGURE 9.—Average operating conditions for typical pumping districts on Mississippi River.

recommended for those plants as a result of the study presented in the succeeding pages, and the actual plant capacities at the recommended maximum lifts as determined from rating curves prepared from tests of the plants.

TABLE 5.—*Design factors for certain pumping drainage districts*

District		Actual design factors		Recommended design factors			Actual 24-hour capacity at maximum lift recommended
Key no.[1]	Name	Maximum 24-hour run-off	Total head on pumps	Average annual run-off[2]	Maximum 24-hour run-off[3]	Maximum static lift	
		Inches	*Feet*	*Inches*	*Inches*	*Feet*	*Inches*
22	South Beardstown	0.38	27	35	0.43	24	0.44
52	Green Bay	.36	26	32	.41	13	.48
51	Henderson County	.26	15	27	.38	13	.26
18	Coal Creek	.26	18	24	.36	22	.23
35	Eldred	.52	14	20	.34	18	.24
19	Crane Creek	.26	24	16	.32	21	.22
28	Valley City	.26	21	16	.32	20	.26
43	Bay Island	.23	15	15	[4].26	17	.12
6	Banner Special	.37	20	15	.31	23	.31
30	Scott County	.35	22	14	.30	22	.30
27	McGee Creek	.40	16	14	.30	20	.34
33	Hartwell	.55	21	14	.30	18	.44
57	Indian Grave	.27	17	12	.29	16	.21
59	Fabius	.31	16	7	.26	13	.27
29	Mauvaisterre	.15	17	5	.25	12	.15

[1] For identification in figures 1 and 10 and table 7.
[2] See table 4.
[3] Computed as $C=1.00\ (0.22+0.006\ r)$.
[4] Computed as $C=0.85\ (0.22+0.006\ r)$, because of large area drained.

To determine the proper capacity of pumping plants for economical drainage in this region, careful study was made of the pumping records obtained from the 15 plants listed in table 4. Various methods of comparing these data for calculating economical plant capacities were

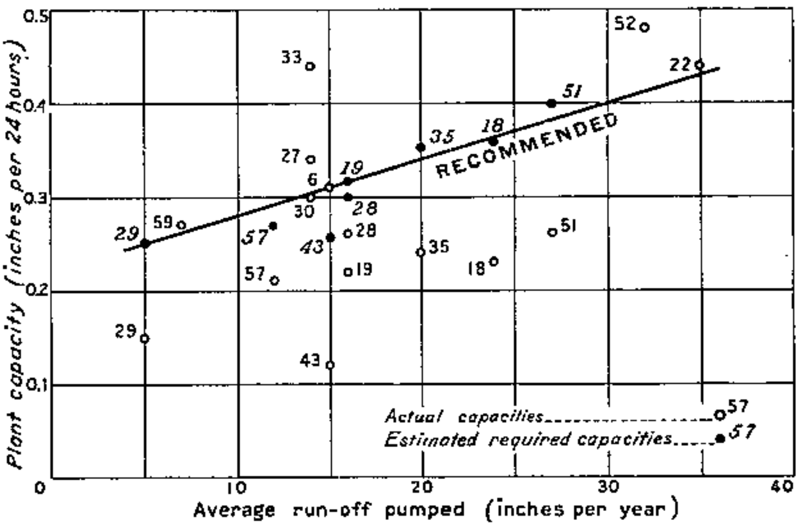


FIGURE 10.—Actual and recommended plant capacities at maximum lifts for drainage pumping plants in upper Mississippi Valley. (Numbers identify data in tables 4 and 5.)

examined, but because the capacity of a drainage pump decreases as the lift increases it was concluded that capacity at maximum lift is the most satisfactory basis for design. Comparison of these plants was greatly simplified by reason of the drainage areas being similar in topography, crops, weather, river fluctuations, and generally in size.

After investigating the experiences of the drainage districts, the probable frequency of river floods, and the damage to crops resulting from lack of drainage it was concluded that for average agricultural lands in the upper Mississippi Valley a drainage pumping plant should be able to prevent flooding of large areas for periods longer than 3 or

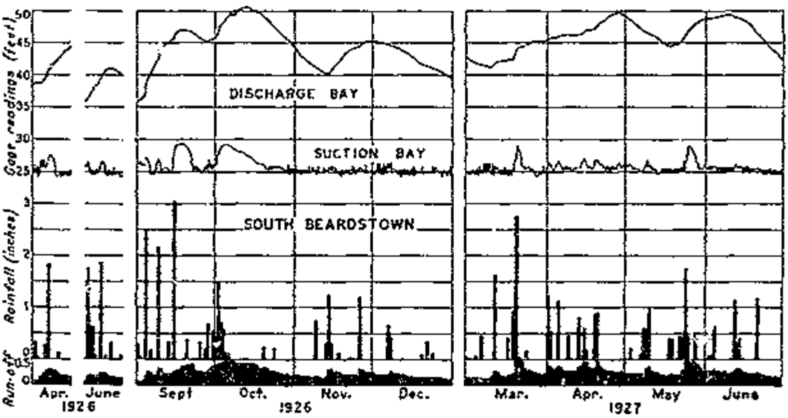


FIGURE 11.—Pumping during flood periods by plant having adequate capacity. South Beardstown plant, 1926 to June 1927.

4 days during flood conditions as severe as those that occurred in 1926, 1927, or 1929. The required capacities for the different plants were found to vary considerably because of large differences in the amounts of seepage. The rate of seepage at any time is not determinable, but it appears that the required capacity of a plant is related

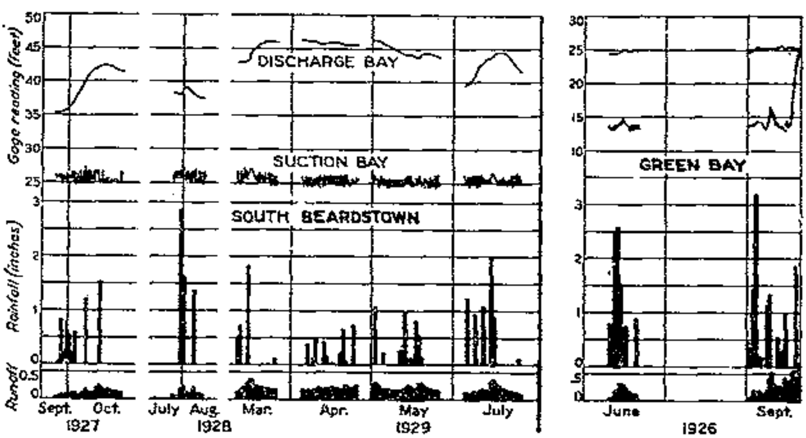


FIGURE 12.—Pumping during flood periods by plant having adequate capacity. South Beardstown plant, September 1927 to 1929, and Green Bay plant.

to the total run-off to be pumped annually. Both plant capacity and annual run-off pumped are integrations, for different periods, of the effects of the same factors—rainfall, topography, vegetation temperature, river stages, and nature of soil and subsoil.

In figure 10 the actual plant capacity at maximum lift, C, has been platted against the average annual run-off to be pumped, r, for each plant. Those plant capacities were determined from rating curves prepared from tests of the pumps, and the amounts of run-off are those shown in table 4. In figures 11 to 18 are shown the daily

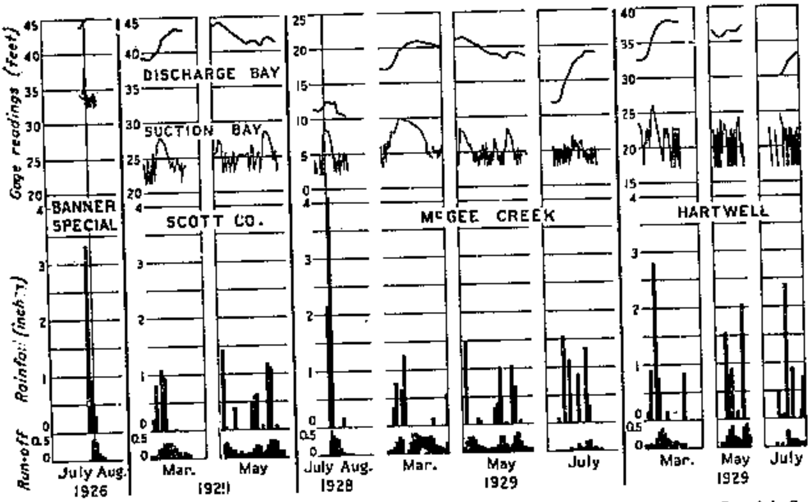


FIGURE 13.—Pumping during flood periods by plant having adequate capacity. Banner Special, Scott County, McGee Creek, and Hartwell plants.

records of rainfall, run-off pumped, and stages of the suction and discharge bays during flood periods, for 14 of the plants. Those records and the records of the Fabius plant indicate that, at maxi-

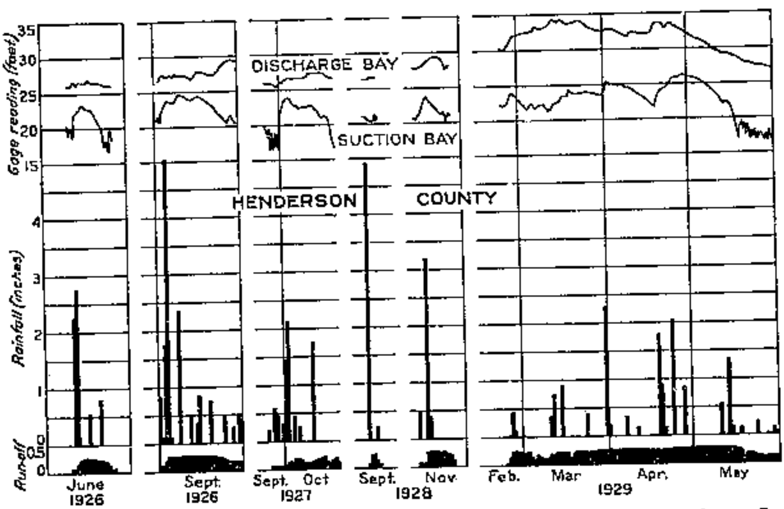


FIGURE 14.—Pumping during flood periods by plant having inadequate capacity. Henderson County plant.

mum lift, 4 of the 15 plants studied had capacity just about equal to that necessary for maintaining proper drainage in the respective districts, that 3 plants had capacity greater than necessary, and that the other 8 plants were not adequate.

The adequate plants lowered the suction bays nearly to the optimum stages within a few days even during extreme floods. With the inadequate plants, however, the suction bay stages remained 4 to 6 feet above the optimum for periods of 2 weeks or longer, although pumping

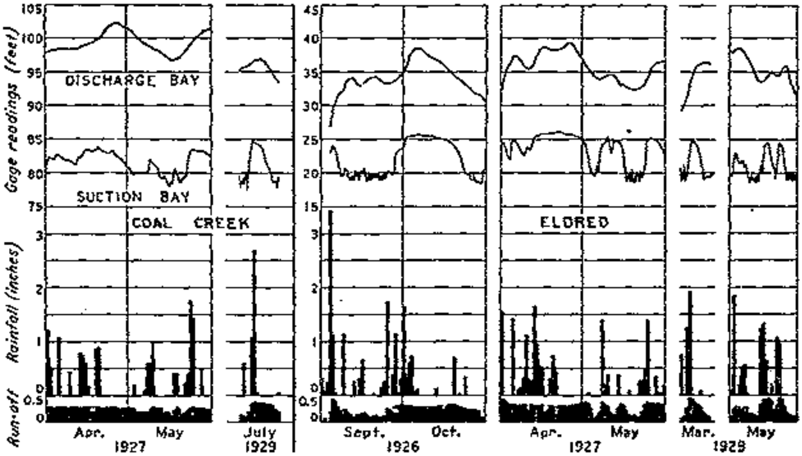


FIGURE 15.—Pumping during flood periods by plant having inadequate capacity. Coal Creek and Eldred plants.

at full capacity was continuous. Flooding of Green Bay district in September 1926 (fig. 12) was due to a break in the levee, and flooding of McGee Creek district in March 1929 (fig. 13) was caused by lack of fuel for 3 days. The height and rapidity of the rises in the suction

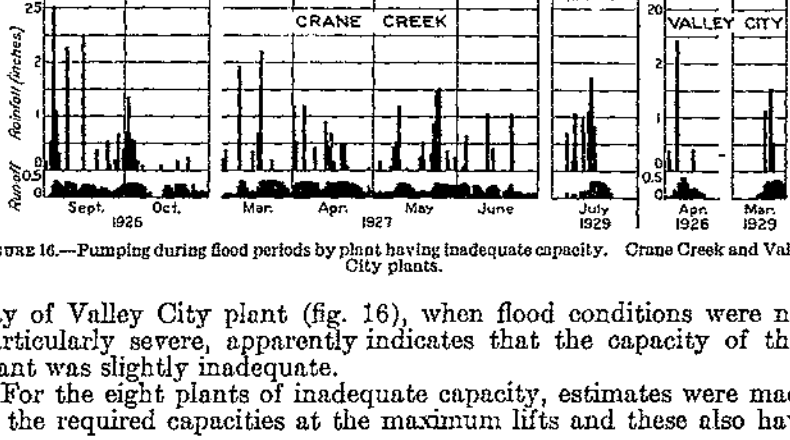


FIGURE 16.—Pumping during flood periods by plant having inadequate capacity. Crane Creek and Valley City plants.

bay of Valley City plant (fig. 16), when flood conditions were not particularly severe, apparently indicates that the capacity of that plant was slightly inadequate.

For the eight plants of inadequate capacity, estimates were made of the required capacities at the maximum lifts and these also have been plotted in figure 10. For all except the Mauvaisterre and Henderson County plants (nos. 29 and 51, respectively) those estimates are actual quantities pumped during floods, but at low lifts

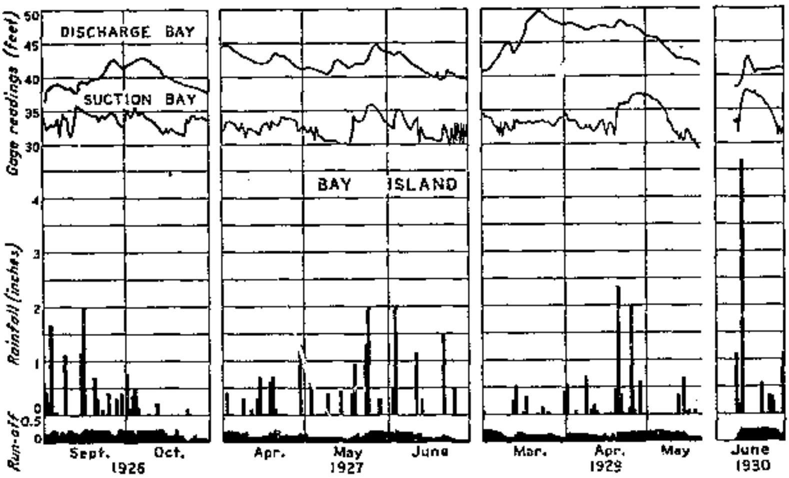


FIGURE 17.—Pumping during flood periods by plant having inadequate capacity. Bay Island plant.

resulting from high suction-bay stages, when the water in the suction bays was being lowered so rapidly as to indicate adequate rates of pumping. The two plants named could not pump at any lift a

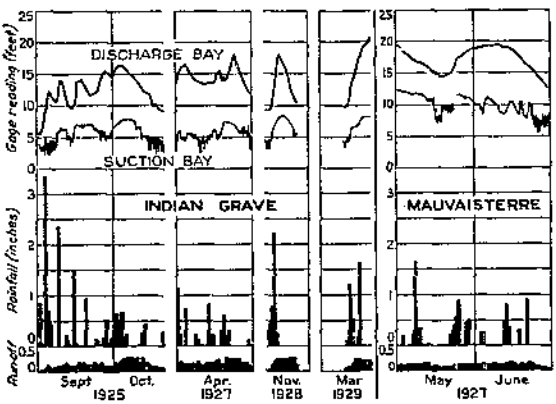


FIGURE 18.—Pumping during flood periods by plant having inadequate capacity. Indian Grave and Mauvaisterre plants.

quantity that would be necessary at maximum lift, so the required maximum-lift capacities were estimated from consideration of the performance of those plants and the evident requirements for similar plants.

FORMULAS FOR PLANT CAPACITY

In order to determine a formula for computing the required capacities of drainage pumping plants in the upper Mississippi Valley, a straight line was drawn in figure 10 as nearly as possible through the 4 points representing plants of adequate but not excessive capacity and 7 of the points representing estimated requirements of inadequate plants. The Bay Island plant (no. 43 in fig. 10) drains a much larger area than any of the other plants, and its required capacity in run-off depth is much less than that of the other plants.

The equation of the line of recommended capacities in figure 10 is

$$C=0.22+0.006r \tag{1}$$

The equation seems to express the proper relation of plant capacity to total annual run-off pumped for all except the Bay Island plant, which drains 52,000 acres whereas most of the plants drain less than 16,000 acres. That plant apparently had sufficient capacity when pumping about 0.26 inch of run-off per day at high stages of the suction bay (fig. 17). The equation would apply reasonably well to the Bay Island plant if the factor 0.85 were introduced, which suggests that a general formula should include a coefficient related to the size of drainage area. The estimated requirement for the Indian Grave plant (no. 57) shows the next greatest departure from the line of recommended capacities. The formula would fit that plant, which drains 21,000 acres, with a coefficient of about 0.95. Because of these variations, formula 1 is recommended as applying to plants pumping from areas of 3,000 to 15,000 acres.

It is also desirable to have a coefficient by which the capacity of plant can be varied according to the degree of protection required for the property in the district. It is believed expedient, however, to use one coefficient for both size and adequacy, which would be the product of the two separate factors as determined by the designing engineer. Thus for drainage pumping plants in the upper Mississippi Valley the formula would be

$$C=K_1(0.22+0.006r) \tag{2}$$

in which K_1 would be the size and adequacy coefficient.

The maximum daily run-off logically comprises run-off from rainfall upon the drainage area, just as for all-gravity drainage systems, and seepage from the high water bordering the district. The value of 0.22 in equation 1 is slightly less than the 0.25-inch run-off depth generally accepted as the necessary capacity of all-gravity drainage systems in the region where these pumping studies were made. In figure 10 the line of equation 1 cuts the line of 0.25-inch daily run-off at $r=5$, which is the lower limit of the range of pumping studies and also the probable minimum annual run-off for which a pumping plant would be installed. Therefore, formula 2 might be modified to read

$$C=K_1[0.25+0.006(r-5)] \tag{3}$$

which would, with proper values of K_1, fit all the plants studied in the upper Mississippi Valley. In general terms, this might be written

$$C=K_1[K_2+K_3(r-a)] \tag{4}$$

in which

C=plant capacity at maximum lift, in inches depth of run-off for 24 hours
K_1=size and adequacy coefficient, depending upon area drained and adequacy of drainage
K_2=rainfall-run-off coefficient, equal to the run-off capacity for all-gravity systems draining similar areas and topography
K_3=seepage coefficient, indicated by the slope of the line of recommended capacities
r=total annual run-off to be pumped, in inches depth over the drainage area
a=number to be determined empirically for each particular region.

Although a straight line seems to represent the relation of plant capacity to total run-off pumped within the limits of the data obtained, it is not improbable that this relationship would frequently be a regular curve which could be expressed by formula 4 with some exponent other than unity, or a varying exponent, for one of the terms in the formula.

APPLICATION OF FORMULAS

To compute maximum daily run-off for design of upper Mississippi Valley pumping plants it is recommended that formula 2 be used as follows:

$$C=K_1(0.22+0.006r),$$

with $K=1.0$ for agricultural districts growing corn and grain crops, ranging in area from 3,000 to 15,000 acres. This coefficient should be decreased with increase in drainage area, by about 15 percent for areas of 50,000 acres. Where more complete protection is required, as where valuable town property is involved, the coefficient should be increased from that determined for an equal area of farm lands.

The average annual run-off, r, may range from 5 to 12 inches for districts obtaining considerable gravity drainage, from 12 to 16 inches for nonseeped districts pumping all run-off, and from 16 to 35 inches for heavily seeped districts. The amount of this run-off can best be estimated by comparing the drainage area under consideration with those described on pages 13 to 16 in regard to soils and elevation with respect to river stages and then applying the conditions of static lift for the proposed installation (determined as explained on pages 5 to 11) to the appropriate curve in figure 6.

When there is large storage capacity for run-off outside the drainage ditches, a deduction may be made from the run-off coefficient determined by formula 2. The data presented in this report relate to districts in which there were not more than a few acres of storage available in lakes and sloughs. As the total storage available must be utilized through a period of 2 weeks or more, it will not materially affect the required capacity of the plant unless it amounts to as much as half the maximum daily pumping capacity.

The general application of formula 4 in regions where the values of the coefficients K_1, K_2, K_3, and a are not indicated by existing pumping records will be attended with some uncertainty. It is believed, however, that with this formula, or the simpler formula 2 applicable in the upper Mississippi Valley, more certain results can be obtained than by estimating plant capacity without some such guide.

It is believed that in using the formula

$$C=K_1[K_2+K_3(r-a)]$$

K_1 will not vary with respect to geographic location of the drainage area. K_2, K_3, and a are believed to be regional coefficients, varying only with large differences in area, topography, drainage, farming, and climatic conditions. In any particular region the rainfall-run-off coefficient, K_2, would be taken as the capacity required for gravity drainage districts, determined from local records and experience or calculated by the rational method of computing run-off. The seepage coefficient, K_3, and a can be determined accurately only from extensive pumping records in the region. It is believed that the values applicable in the upper Mississippi Valley may be used if the soils and pumping lifts are at all comparable. The amount of annual run-off to be pumped, r, may be estimated by a comparison of soils, ground elevations, and static lifts with the same elements for the districts described on page 14 and graphed in figure 6.

As an example of the application of formula 4, the required capacity may be calculated for a pumping plant to drain about 10,000 acres of river-bottom land, including some village property, in a region where the average annual rainfall is 45 inches but no drainage pumping records are available. A study of river stages and ground-surface elevations indicates that the maximum static lift will be about 24 feet, the minimum monthly lift about 8 feet, and the average annual lift approximately 14 feet. No considerable storage will be available outside the drainage ditches. Seepage of the land and subsoil borings show that about 20 percent of the area is underlain with sand strata that will be tapped by the ditches.

By comparison of these data with upper Mississippi Valley conditions, the size and adequacy coefficient is estimated as 1.1 because of the village property to be protected. The value of K_2 is taken as 0.35 inch, the daily run-off capacity of gravity drainage systems in the locality. The values of K_3 and a in formula 3 are used. The seepage and soil conditions are similar to those of the Coal Creek district (p. 15), therefore the curve for that district in figure 6 is used to determine the relation of run-off to rainfall. For a static lift of 14 feet that curve shows the run-off to be pumped annually as 87 percent of the rainfall, which would be 39 inches depth over the proposed district. Substitution of these values in formula 4 gives

$$C=1.10[0.35+0.006(39-5)]=0.61.$$

Thus the capacity of plant is determined as 0.61 inch depth per 24 hours on 10,000 acres, or approximately 115,000 gallons per minute. This capacity should be provided at 24-foot lift.

MINIMUM RUN-OFF

The minimum rate of run-off is influenced by the elevation, size, and slope of ditches leading to the pumping plant or the available storage in lakes or sloughs near the plant. An analysis of these factors should be made to determine the minimum size of pump needed.

SELECTION OF PUMPS

Centrifugal, screw, and mixed-flow pumps are the types most suited for drainage pumping. Centrifugal pumps are best adapted for pumping against heads exceeding 12 feet; screw and mixed-flow pumps are more efficient at heads less than 10 feet. The advantages of using more than one type of pump should be considered in proposed installations, a centrifugal pump for use at high heads and a screw or mixed flow pump for efficient pumping at low and medium lifts. In some plants it may be desirable to install one or more pumps that are especially efficient, to do most of the pumping.

The screw pump and the mixed-flow pump operate at higher speeds for similar conditions than the centrifugal pump. This is an advantage, particularly for electrically driven units, because high-speed motors cost much less than low-speed motors and a high-speed pump can be direct-connected to a cheaper motor.

CENTRIFUGAL PUMPS

The double-suction volute centrifugal pump was used almost exclusively in drainage plants in the upper Mississippi Valley for

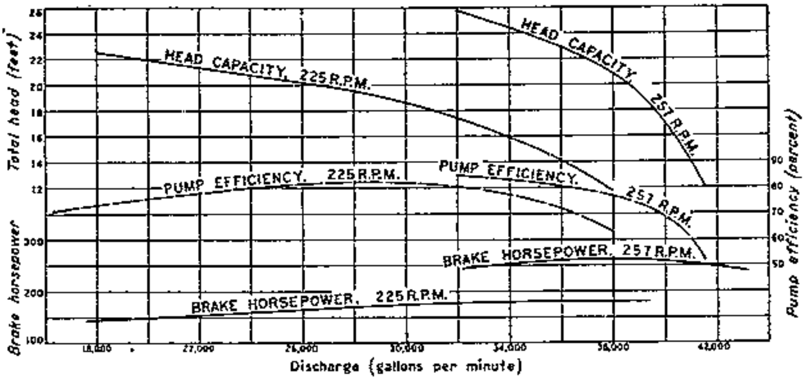


FIGURE 19.—Characteristic curves of typical modern centrifugal pump (field tests of 36-inch pump, Hunt plant).

about 20 years prior to 1928. Single-suction centrifugal pumps were used in many earlier plants, but such are used now only in very small sizes (pl. 1, A).

Considerable development of centrifugal pumps has taken place since 1915. Characteristic curves are shown in figure 19. An increase of almost 20 percent in efficiency has been obtained by improved design. However, the efficiencies of old low-speed, steam-driven pumps in the Bay Island and Henderson County plants were approximately as great at low and medium lifts as the efficiencies of recent higher-speed pumps. Such low-speed pumps would not be economical now because difficult to adapt to electric or oil-engine operation and more costly because of greater weight. At the present time, efficiencies of 80 to 85 percent are guaranteed and obtained on centrifugal pumps at heads above 18 feet. The Francis-type impeller, with specially curved vanes, is particularly efficient for drainage pumping.

Centrifugal pumps have a long life and are dependable. They usually have a greater capacity than the same size screw or mixed-flow pumps, especially against the higher heads. They often have a slight price advantage in comparison based on capacity rather than size. However, a small difference in first cost is less important than an appreciable difference in average operating efficiency.

SCREW PUMPS

The true screw pump is a patented type, and is especially adapted for low-head pumping (pl. 1, B). The impeller has many blades, somewhat similar to those of a ship propeller, set on the shaft at angles determined according to the head and speed. The direction of flow through the pump does not change as in a centrifugal pump. A spiral motion of the water results from the screw action, but is corrected by diffusion vanes. The type has been in successful operation in Louisiana for many years but only since 1928 have such pumps been

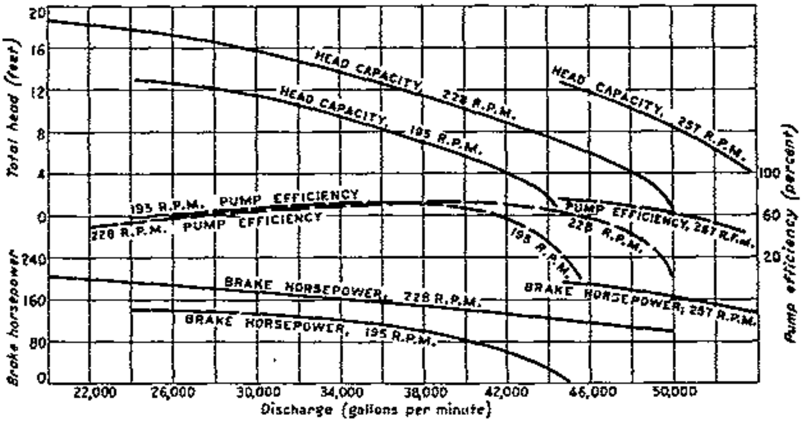


FIGURE 20.—Characteristic curves of typical screw pump.

offered in sizes required by the upper Mississippi Valley drainage plants at prices to compete with centrifugal pumps.

The screw pump operates at high efficiencies against heads less than 10 feet, and against fluctuating heads more efficiently than does the centrifugal pump because it retains nearly maximum efficiency through a greater range of head (fig. 20). One of its disadvantages is that the discharge falls rapidly at heads above 15 feet. For this reason its use is somewhat limited in the upper Mississippi Valley. The screw pump can be built so the flow can be reversed if it is wanted for both irrigation and drainage. This is quite an advantage for some locations. A trash cutter permits the removal of trash from the blades without opening the pump.

MIXED-FLOW PUMPS

The mixed-flow pump also is particularly adapted for drainage pumping (pl. 1, C). It has an open vane, screw-type impeller which combines the screw and centrifugal principles in building up the pressure head. It operates efficiently against somewhat higher heads than the true screw pump. With one change in speed the Hartwell mixed-flow pump operates at 70 to 80 percent efficiency at all heads from 6½ to 26 feet, and the discharge does not decrease excessively at the higher heads (fig. 21). The open-type impeller of the mixed-flow pump facilitates the passage of trash.

VERTICAL TYPES

Submerged single-suction pumps have been used in a few drainage plants. A simplified unit of this type has been developed recently, consisting of a submerged pump connected by a vertical shaft to a built-in vertical motor (fig. 22). Such pumps can be equipped with either the centrifugal or screw-type impellers. Their chief advantage is that a small building will satisfactorily house the motor and switchboard (pl. 2, A). Another advantage is the elimination of priming equipment, which makes them especially suitable for automatic operation. The greatest disadvantage of vertical types is the inaccessibility of the pump for cleaning. When this type is installed,

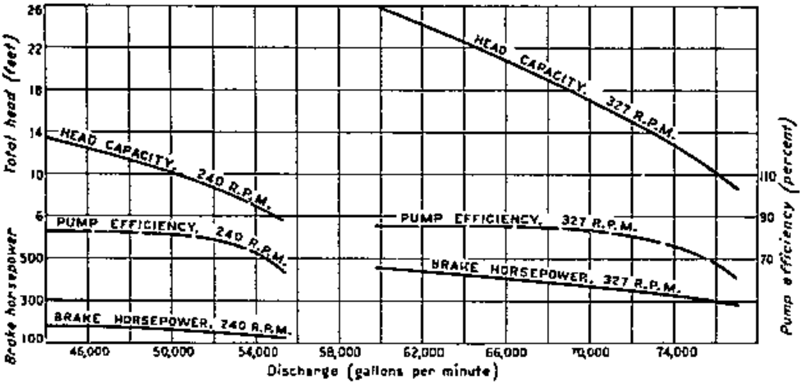


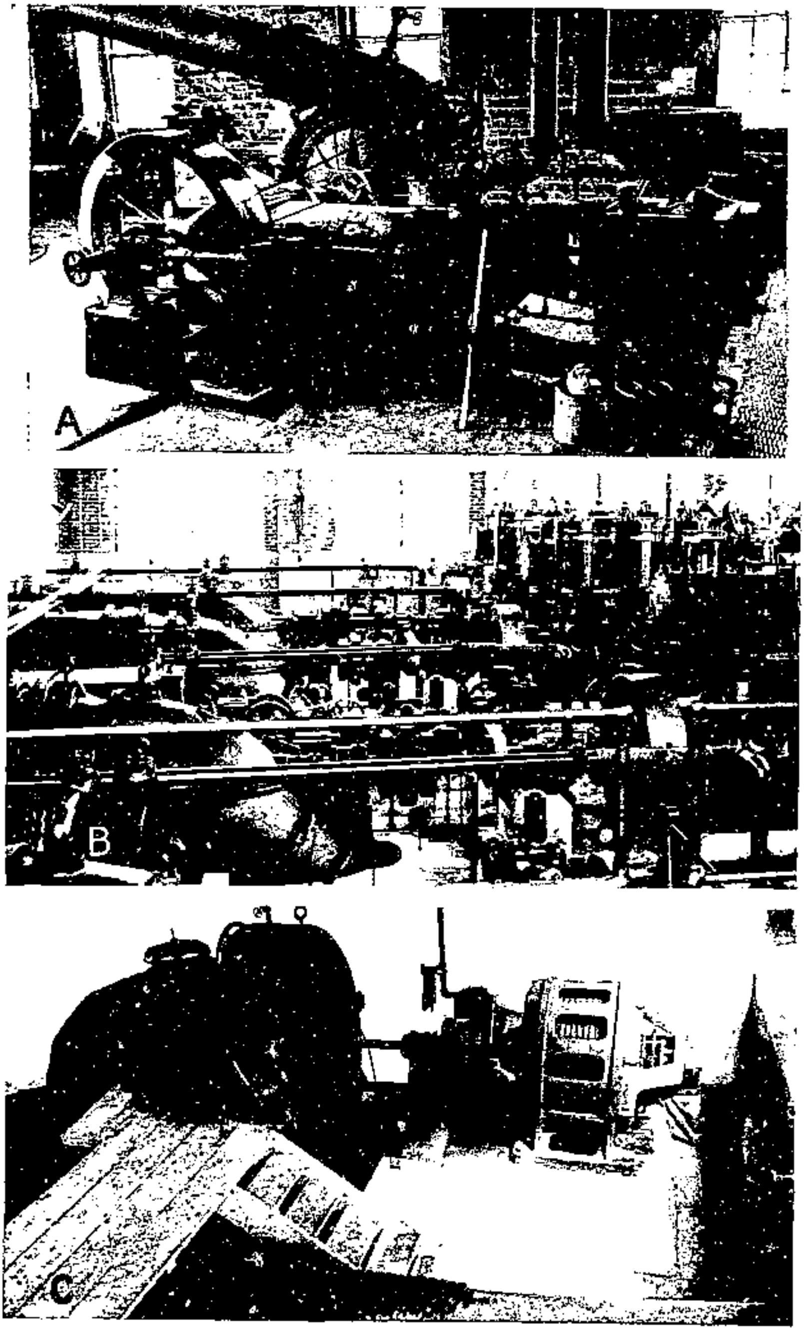
FIGURE 21.—Characteristic curves from tests of 48-inch mixed-flow pump at Hartwell plant.

provision should be made to clean the pumps by (1) closing the suction bay, preferably by gate valves, and draining the pump pit with a small auxiliary pump; (2) hoisting the unit above the water level, which is practicable with small pumps; or (3) in the case of large units, forcing water from the pump as from a pneumatic caisson and putting a man down inside.

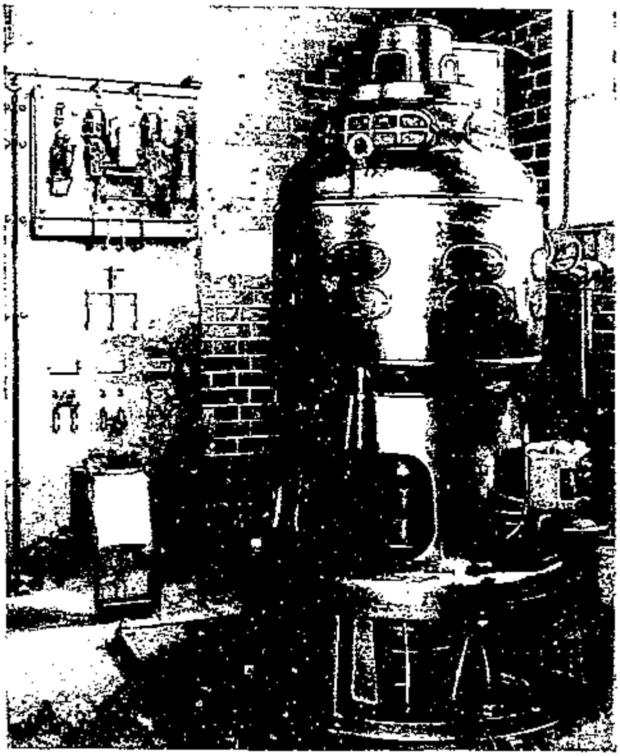
SPEED ADJUSTMENT

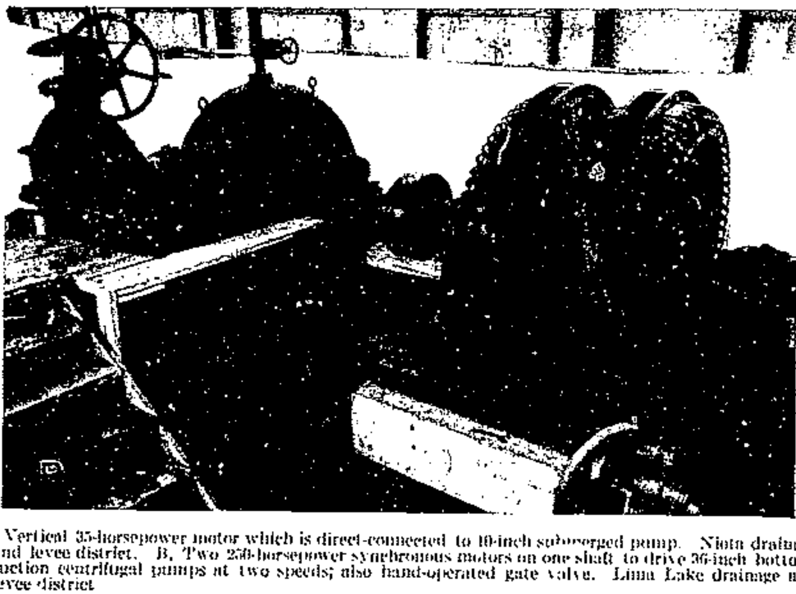
Against high heads, pumps of the types described are most efficient when operated at high speeds. Against lower heads it is necessary to operate at lower speeds to maintain high efficiencies. The amount of speed adjustment provided should be according to the amount of variation between maximum and minimum lifts and the characteristics of the pump to be used. The speed range must sometimes be 25 percent of the maximum speed. Prospective purchasers usually can obtain from the pump manufacturer characteristic curves for use in determining the different speeds of operation.

Figures 19 and 21 show typical examples of the speed adjustment of centrifugal and mixed-flow pumps. Each of those units is direct-connected and obtains 2 speeds by means of 2 synchronous



A, Small overshot single-suction centrifugal pump direct-connected to 75-horsepower steam engine. Des Moines County drainage and levee district no. 8. B, Four 51-inch screw pumps direct-connected to four 240-horsepower Diesel engines. (Fourth unit hardly shows in the foreground.) East Side levee and sanitary district. C, A 36-inch mixed-flow pump direct-connected to 250-horsepower induction motor. Lima Lake drainage and levee district.





A, Vertical 35-horsepower motor which is direct-connected to 10-inch submerged pump. Niota drainage and levee district. B, Two 250-horsepower synchronous motors on one shaft to drive 36-inch bottom-suction centrifugal pumps at two speeds; also hand-operated gate valve. Lima Lake drainage and levee district motors mounted on the same shaft (pl. 2, B), but belt-connected units could obtain the speed changes by changing pulleys. The Hunt pump (fig. 19) operates at 225 and 257 revolutions per minute. At the lower speed its efficiency is below 70 percent of all heads less than approximately 14 feet, and for heads from 17½ to 21 feet either speed gives efficiencies in excess of 70 percent. A 200-revolution-per-minute motor probably would have given 70 percent efficiency at heads from 10 to 17½ feet, and substituted for the 225-revolutions-per-minute motor would have enabled the plant to operate at 70 percent efficiency or better at all heads from 10 feet to 25 feet. The speed adjustment of the Hartwell pump (fig. 22) is better. At 240 revolutions per minute the unit has an efficiency of 70 to 80 percent between 6½- and 9-foot heads, and above 80 percent efficiency between 9- and 14-foot heads. At 327 revolutions per minute its efficiency is 70 to 80 percent at heads between 11 and 15 feet, and above 80 percent at heads between 15 and 26 feet.

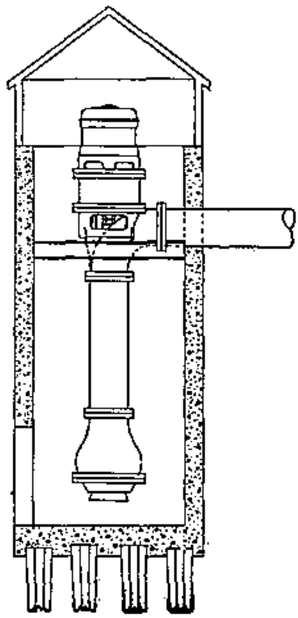


FIGURE 22.—Submerged vertical pump direct-connected to vertical motor.

TRANSMISSION EQUIPMENT

Direct-connected pumping units are much preferred to belt- or gear-connected units because power losses in transmission are eliminated, the purchase and maintenance of transmission equipment are saved, and the floor space occupied by each unit is much less. Low-speed induction and synchronous motors that can be direct-connected to pumps are now available, at somewhat higher cost than higher-speed motors which require belts or gears to drive the pumps (pls. 2, B and 3, C).

Leather or chain belts were the earliest transmission equipment with electrically driven drainage pumps (pls. 3 and 4). Rope belts and V-shaped belts are better where the distance between engine and pump is short. Adequate speed adjustment of such units can be obtained usually by three sizes of pulleys for the motor shaft. It has been the general experience that, because of labor involved, pulleys are not changed frequently enough to obtain a satisfactory average pump efficiency. This points out another practical advantage of direct-connected units. At the Hartwell, however, a special chain hoist enables one man to change the pulleys on the three original units in less than an hour. The simplicity of this device, together with competent engineering supervision, has obtained satisfactory speed adjustment.

Reduction gears have been used in a few electric installations (pl. 4, B). Although they save floor space, such gears are expensive, and there is no satisfactory method of changing the speed ratio of large reduction gears.

Vertical oil-engine units should usually be direct-connected to the pump, where both engine and pump operate between 200 and 300 revolutions per minute. Most oil engines can be equipped with a governor that will permit a sufficient range in the speed of the pump. Horizontal oil engines vibrate so much that a belt connection to the pump seems generally desirable to decrease the possibility of burning out engine and pump bearings.

Direct connection of pumps to engines was used in many of the early steam plants. This was possible because those engines and pumps operated at approximately the same speeds, and good speed adjustment could be obtained by changing the governor (pl 1, A).

A clutch connection is of advantage in an oil-engine unit (pl. 4, C) to permit the engine to get warmed up before the load is thrown onto it.

SIZE AND NUMBER OF PUMPS

Drainage pumps in the upper Mississippi Valley range from 18 to 60 inches in diameter of discharge opening, but for reasons of cost it is usually desirable to make selection from the stock sizes of 24, 30, 36, 42, and 48 inches. Centrifugal pumps may differ somewhat in capacity from screw and mixed-flow pumps of the same size. Therefore it may be necessary in estimating costs to compare a 36-inch centrifugal with a 42-inch screw or mixed-flow pump, as those sizes often have comparable capacities at the maximum lift. Of 24 pumps installed in the 3 years 1928 to 1930, 11 were 36-inch pumps. This size is of convenient weight to handle in construction, and large enough to pass most trash through the impeller. Manufacturers build many of that size and for that reason it may be somewhat more economical in first cost.

Large pumps are desirable for drainage pumping because larger pieces of wood and more debris can pass through the pump without clogging the impellers. On the other hand, it is desirable to have at least one unit of capacity small enough that the water in the ditches can be controlled without too frequent starting and stopping of the pumps.

It is an advantage to have two or more units in a plant so that a break-down of one will not stop all pumping. This is especially true for a district that is entirely dependent on the plant for drainage. A district that obtains gravity drainage a large part of the year can place more dependence upon a single unit, as the period of operation is not long and the machinery can be kept in good repair by work during low-water stages.

As a general rule, the most economical size of pump will discharge the water at a velocity of 8 to 10 feet per second against the maximum head. Lower velocities are desirable if the plant operates at such a high annual plant factor [2] that high pump efficiency is of great importance. If the unit operates at such a low plant factor that pump efficiency is less important than first cost, it may be economical to install pumps from which velocities will be as much as 12 to 13 feet per second, even though the efficiency of a pump suffers when such high velocities are used. Many of the pump losses vary as the square of the velocity.

The capacity desirable for the smallest pump in a plant depends upon the minimum rate at which water flows to the plant and the

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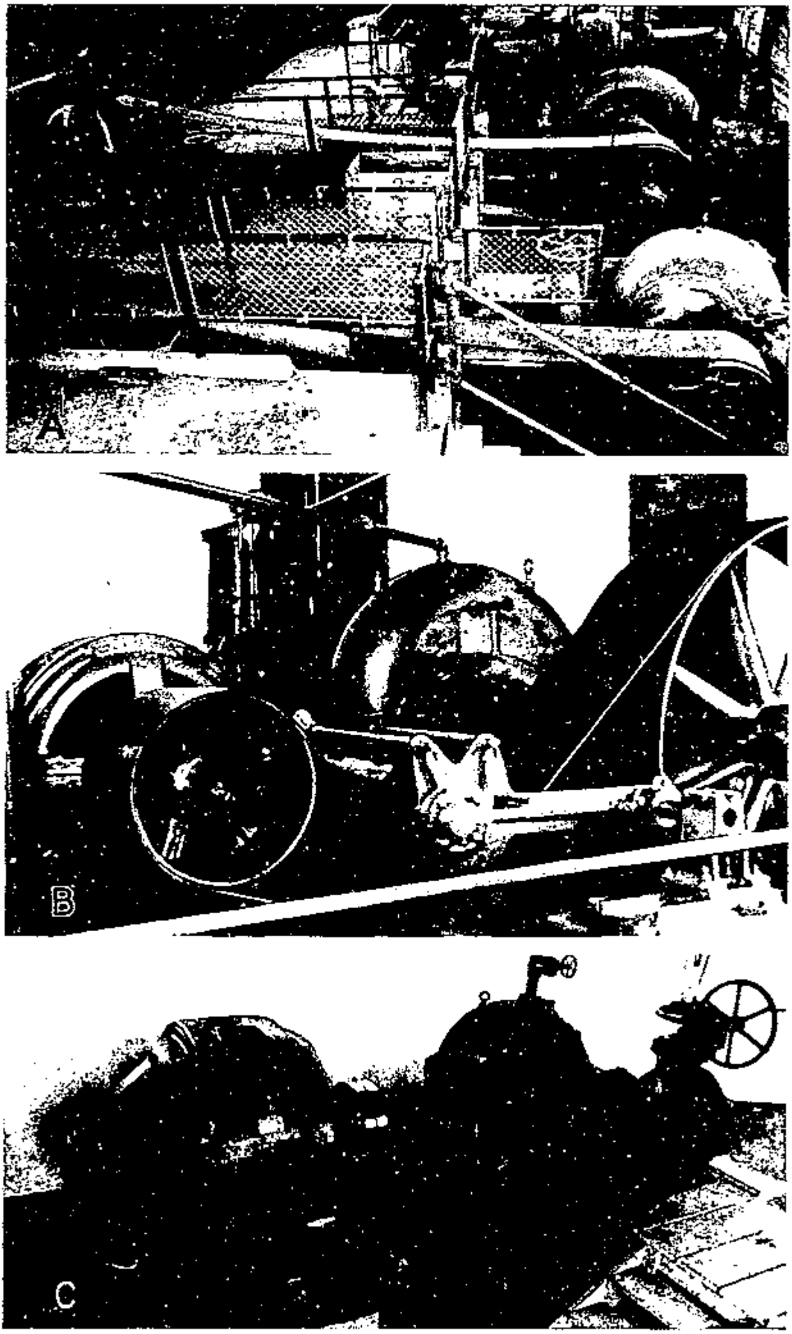
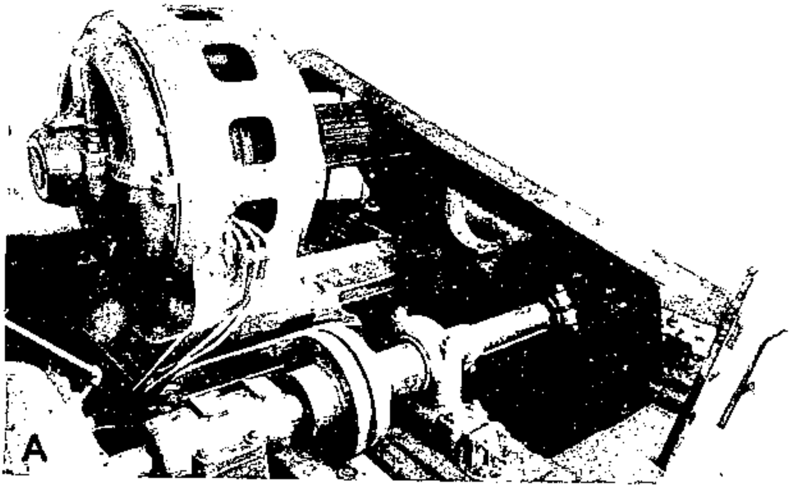
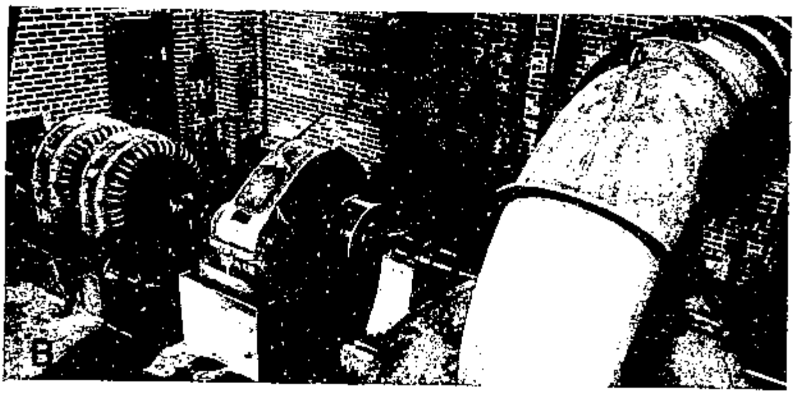
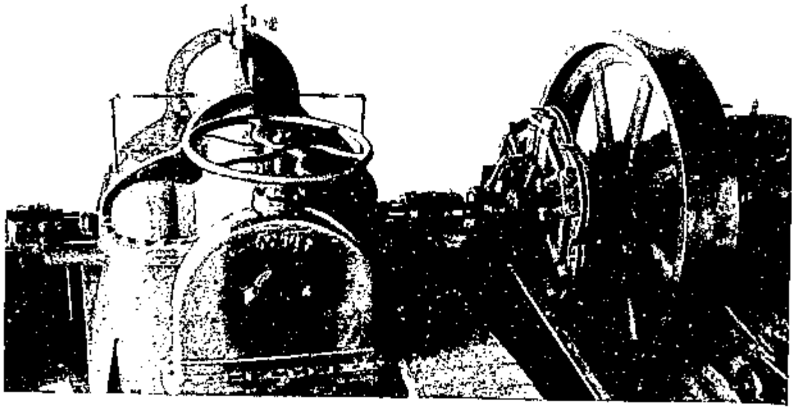
[2] The annual plant factor is ratio of the amount of pumping actually done to the amount that the plant might have done in the year, measured for each unit by the number of hours operated and the rated power of the engine or motor.



A, Typical belt-connected pumping units. Lacey, Langellier, and West Mantanzas drainage and levee district. B, Belt transmission using patented idler pulley where distance between motor and pump shafts is short. Hillview drainage and levee district. C, Low-speed induction motor direct-connected to 36-inch bottom-suction centrifugal pump, for use at high lifts only. Lima Lake drainage and levee district.







A, Chain-belt transmission between 200-horsepower motor and 36-inch centrifugal pump. South Quincy drainage and levee district. B, Large speed-reduction gear connecting two synchronous motors with centrifugal pump. Henderson County drainage and levee district. C, Clutch connection between 150-horsepower semi-Diesel engine and 30-inch centrifugal pump. Union Township drainage and levee district.

amount of this flow that can be stored nearby. If not much storage is available a small pump may be necessary to hold the water low enough by continuous pumping to drain properly low areas possibly 2 or 3 miles distant.

Experience has shown that in the upper Mississippi Valley the larger drainage pumping plants should have one unit that can pump with reasonable efficiency as little as one third the total capacity of the plant, if there is but little water-storage capacity outside the drainage ditches. Three units of equal size could be installed, or two units with one having about half the capacity of the other. If sufficient storage capacity in lakes and sloughs is available, two units of equal size might be used in the plant. Plants draining 5,000 to 10,000 acres should have one pump not exceeding 30-inch size, which would throw about 20,000 gallons per minute, unless there are large lakes or sloughs nearby. This capacity may be increased 5,000 gallons per minute for each acre of storage available at the minimum operating stage of the suction bay. In districts smaller than 5,000 acres the smallest pump may have half the total plant capacity, but not more than 20,000 gallons per minute. Districts larger than 10,000 acres may increase the minimum pump size by 1 gallon per minute for each additional acre drained. It is believed that determination of minimum pump sizes on this basis will provide drainage for lands in outlying portions of the districts with minimum investment in the pumping plants.

SEAL GLANDS

To prevent air from getting into the pump along the shaft, effective seal glands are desirable for all drainage pumps except those submerged. Seal glands operated with water from the discharge pipe are satisfactory only when the discharge-bay elevation is sufficient to keep the discharge pipe under pressure. At low-river stages the discharge pipe of practically every plant is under vacuum and no water is forced through the glands. For water-seal glands the water should be supplied under pressure, from the cooling system in an oil-engine plant or from an auxiliary pump and tank in an electric plant. Seal glands that use oil instead of water, from a reservoir holding a pint to a quart of oil, are now commercially available. A disadvantage of using oil glands is the frequency with which they have to be filled. Furthermore, the packing around the pump shaft must be kept very tight, for which special material should be used.

PRIMING EQUIPMENT

Wet-vacuum and dry-vacuum pumps are most frequently used for priming, although several plants use ejector pumps operated by steam, water, or air. Wet-vacuum pumps are rotary pumps, which are not injured if water gets into them and are easily installed (pl. 5, A). Slightly more than half the priming pumps installed in drainage plants in the upper Mississippi Valley are of this class. A wet-vacuum system is usually cheaper than a dry-vacuum system using an air tank.

Dry-vacuum pumps are usually air compressors arranged to draw air from the drainage pump and discharge it at atmospheric pressure. Ordinarily the clearance between piston and cylinder head of a dry-vacuum pump is so small that the head may be cracked if water is drawn into the pump. To avoid this trouble, the pipe to the priming pump sometimes is looped 34 feet or more above the suction-bay level, but this may not prevent damage if an unskilled operator starts the drainage pump without closing the valve in the priming suction line. A more satisfactory arrangement is to insert in the priming line a tank that will trap out any water that may get into that pipe (pl. 5, B).

Steam ejectors are used for priming the drainage pumps in practically all steam-operated plants because they are convenient and reliable. When the steam pressure is almost high enough to start the engine the ejector can be started and the pumps primed. Water-ejector pumps have few moving parts and none that wear rapidly. However they have not been used as frequently as dry- and wet-vacuum pumps. A simple water-ejector system consists of a small centrifugal pump forcing water through a jet at high pressure.

Priming pumps in electric plants are driven by small electric motors. The recent tendency has been to use direct-connected priming units, instead of the cheaper high-speed motors and belt or chain connection. In oil-engine plants usually a small gasoline or kerosene engine is used, belt-connected to a wet-vacuum pump. Such engines are hard to start in cold weather, and must be replaced each few years. The priming equipment must be very reliable, because all drainage pumps except those submerged must be primed before starting. The priming pump should be large enough to prime the largest unit in the plant in 8 to 12 minutes.

DESIGN OF SUCTION AND DISCHARGE PIPES

The design of suction and discharge pipes is largely governed by empirical rules. Head losses vary greatly and cannot be estimated nearly so accurately as those in long straight pipes. Bends should be avoided as much as possible to minimize the losses. Pipes should be somewhat larger than the pump connections so that friction losses are reduced to an economical point, and the changes in size should be gradual.

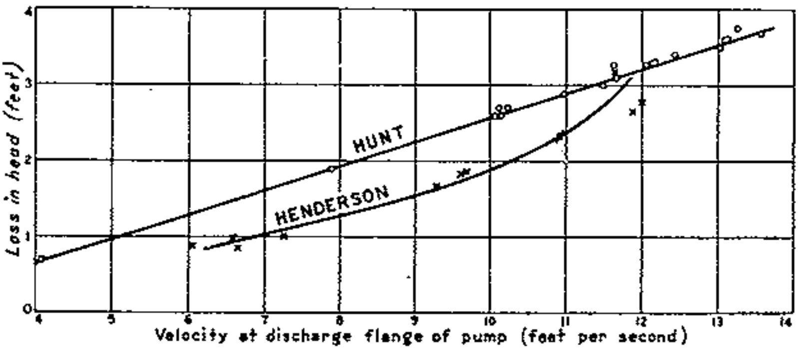


FIGURE 23.—Loss of head in typical well-designed suction and discharge pipes.

A considerable part of the energy used by a drainage pumping plant is required for overcoming entrance, friction, and exit losses in the suction and discharge pipes. Therefore those pipes should be designed so that those losses will be held to the practical minimum. However, because the fixed charges of plant depreciation and interest on investment generally constitute so large a part of the total cost of pumping (*4*) it would not be economical to pay as much for high efficiency in a plant that would operate each year the equivalent of 1 or 2 weeks as in a plant to operate 3 or 4 months.

Entrance losses may be kept low by tapering the suction pipe or progressively expanding the entrance end. Friction losses in the discharge pipe may be reduced by using large-size pipe, connected to the pump flange by a short expanding section. Velocity-head losses at exit may be reduced by enlarging the end of the discharge pipe by a taper or bell section.

HYDRAULIC LOSSES AND PIPE SIZES

Losses of head in typical riveted-steel suction and discharge pipes of several pumping units are given in figures 23 and 24, and the descriptions of those pipes are given in table 6. The head losses were computed by deducting the static lifts from measured total heads on the pumps. They include the losses due to trash and other obstructions in the pipes, to air entering the discharge pipe where it was under vacuum, and to eddies where pipe sizes changed, as well as the entrance, friction, and exit losses. Theoretically most of these hydraulic

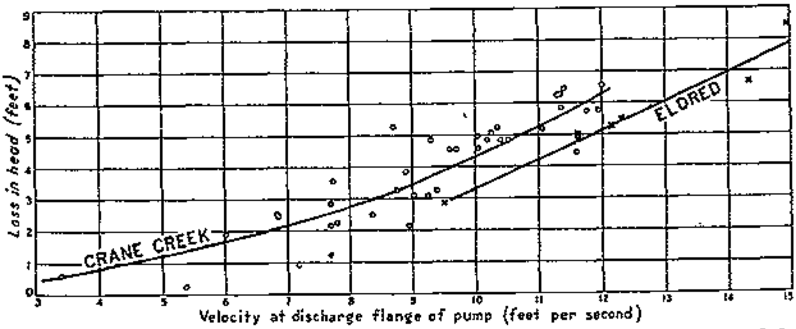


FIGURE 24.—Loss of head in suction and discharge pipes where discharge pipe was not expanded.

losses should increase as the square of the velocity, but the losses measured for the Hunt and Eldred units apparently varied almost as the first power.

TABLE 6.—*Descriptions of riveted-steel suction and discharge pipes for which losses of head are shown in figures 23 and 24*

Item	Plants with well-designed pipes		Plants with discharge pipe not expanded	
	Henderson County	Hunt	Eldred	Crane Creek
Pumping unit ... number	1 and 2	1	1	1
Suction pipes:				
Cross section at entrance ... square feet	40.0	15.9	(1)	(2)
Cross section at pump flange ... do	11.2	7.1	8.5	7.4
Discharge pipes:				
First section length ... feet	24	19	140	94
Diameter ... inches	48–54	39–42	36	36
Second section, length ... feet	128.8	215.0	---	---
Diameter ... inches	54	42	---	---
Cross section at pump flange ... square feet	12.6	7.1	7.1	7.1
Cross section at end ... do	26.3	9.6	7.1	7.1
Valve	None.	(3)	None.	(4)
Head loss in pipes with velocity at pump flange 10 feet per second ... feet	1.0	2.0	3.3	4.3

[1] Indeterminate; see p. 36.
[2] Not determined; see p. 36.
[3] Gate valve at pump; flap gate at end of pipe.
[4] Flap gate at end of pipe.

The Henderson County pipes were expanded more than any of the other units shown, and the loss in head was least (table 6). The suction pipes were expanded in the ratio of 4.4 : 1, between pump flange and entrance end, and the discharge pipes in the ratio of 2.1 : 1. The loss in head was approximately 1.9 feet at a velocity of 10 feet per second. The loss of head in the Hunt pipes was 2.6 feet at the same velocity. The smaller size of the discharge caused some of the difference, and the flap gate on the end of the pipe probably increased the losses somewhat.

The Eldred suction pipe was not expanded but was curved downward and cut off obliquely so the effective entrance area was indeterminate. The pump could pump the water down in the suction bay so low that the bottom edge of the pipe could be seen, which is believed to indicate that the effective entrance area was increased by cutting the pipe obliquely. The fact that the discharge pipe was not larger than the pump connection partly accounts for the high loss of approximately 3.3 feet at a velocity of 10 feet per second. The entrance area of the Crane Creek suction pipe was not determined because the end of the pipe was inaccessible and the construction plans could not be obtained. From the appearance of the pipe at low suction-bay stages it was believed that the expansion ratio did not exceed 1.5 : 1. The discharge pipe of this unit was throughout of the same size as the pump connection. The loss in head was approximately 4.3 feet at a velocity of 10 feet per second, the largest for any of the plants studied.

Loss of head in suction and discharge pipes may be estimated from figure 25. The loss in discharge pipes was determined from the Scobey formula (*3*), with $K_s = 0.51$ applicable to pipe $\frac{3}{16}$ to $\frac{5}{16}$ inch thick having all seams held by rivets with projecting heads, pipes approximately 15 years old conducting nonaggressive waters.

Expanding the suction pipe permits the suction bay being pumped lower without the pump losing its prime. This usually is of greater importance than the reduction in entrance loss, because a high entrance velocity requires considerable depth of water over the end of the pipe to prevent the pump losing its prime. Therefore, expanding the pipe permits use of a more shallow suction bay. Deep foundations are costly, so it is usually economical to expand the suction pipes considerably. By expanding the suction pipe so that the entrance velocity will not exceed 3 feet per second at normal flow, the water ordinarily can be drawn down almost to the edge of the pipe without causing trouble from air getting into the pipe.

Limited observation indicates that a sloping pipe cut off in a plane almost horizontal will draw the water lower than a vertical pipe of the same size that is cut horizontally. It appears that cutting the pipe obliquely increases the effective entrance area somewhat.

The entrance loss to a properly constructed suction pipe is comparatively small. It may be assumed as $0.5 \frac{V^2}{2g}$ if the edges are not belled or rounded. Experiments (*7*) have shown that the loss at entrance to a culvert is negligible for a gradually rounded entrance; hence it appears that belling the end of the suction pipe should reduce the loss to about 0.1 to 0.3 foot, especially when the pipe is greatly expanded as is the usual practice.

The velocity head, which should not be confused with the entrance loss, is $\frac{V^2}{2g}$. Values of the velocity head are shown in figure 25. Part of the velocity head may be recovered by expanding the discharge pipe. The friction loss in a well-designed suction pipe expanded

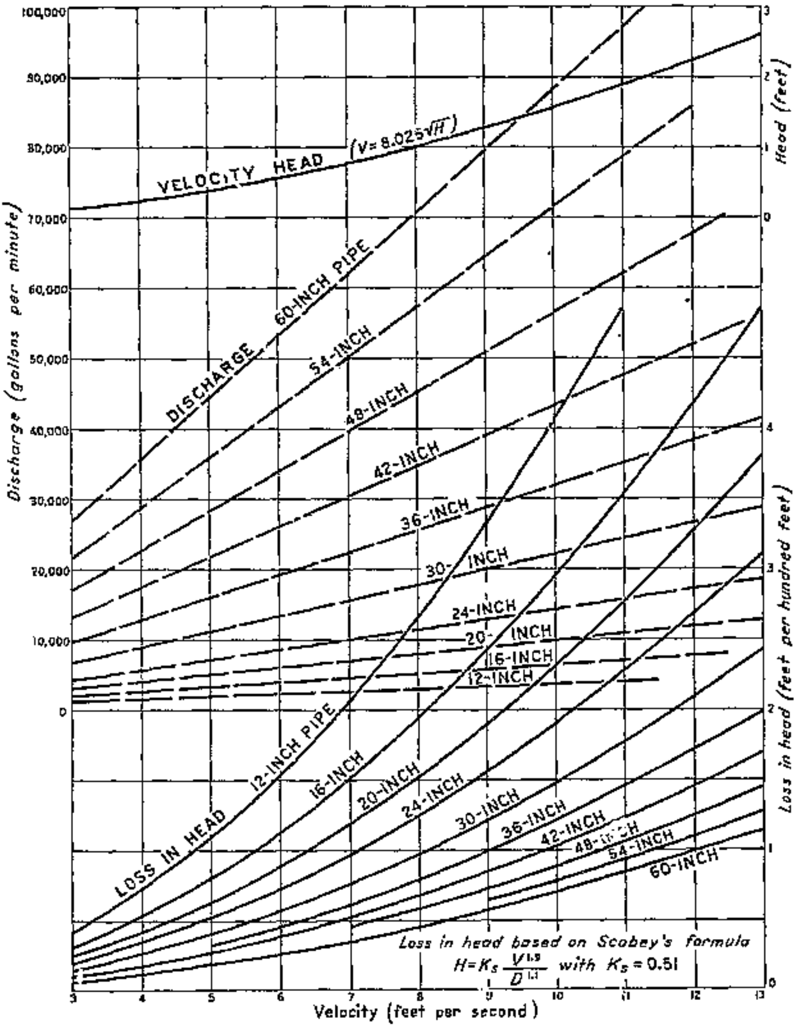


FIGURE 25.—Velocity head, discharge, and friction-head loss in riveted steel pipes.

gradually throughout its length to an entrance area 2 to 4 times that at the pump flange, as indicated in tests of numerous drainage plants, is not likely to exceed 0.2 or 0.3 foot.

The discharge pipes of most drainage pumps are 2 to 6 inches larger than the pump-flange connection. The proper amount of expansion varies, because the saving in annual cost obtained from increased plant efficiency is proportional to the amount of pumping.

The economical amount of expansion can be determined by comparing first costs of different size pipes with the capitalized savings due to increased efficiencies. The economical amount of expansion is greater with electric power than with oil engines because with the former improved efficiencies result in larger savings in cost of operation. The increase in size of pipe should be accomplished by a uniformly expanding section at the pump flange. It is suggested that this section be not less than 10 feet long per 6 inches increase in diameter.

Trouble in keeping the discharge pipes full of water has been experienced at several plants where the pipes were almost level for considerable lengths at the top of the levee. This probably was caused by air accumulating so rapidly that the water could not force it out. At one plant a vacuum pump driven by a 15-horsepower motor was operated 20 to 50 percent of the time to keep the discharge pipes running full. No such difficulty was experienced where the discharge pipes came to a rounded point at the top of the levee. It is believed usually unnecessary to have the level section at the top of the levee more than 4 to 8 feet long.

Ordinarily the bottom of the discharge pipe at its highest point near the top of the levee should be slightly above the expected maximum river stage, so that the levee will not be endangered by seepage along the outside of the pipe. However, the top of the pipe should not be more than 28 feet, the practical limit of suction lift, above the minimum elevation of the discharge bay. The end of the discharge pipe should be submerged at all times, to avoid difficulty in priming the pump. When the end of the pipe is out of water, loss of the siphon effect greatly reduces the operating efficiency of the unit. Around a discharge pipe passing through a levee below maximum flood stage, concrete collars should be constructed to reduce possibility of seepage along the pipe.

The pipe may be laid upon the surface of the levee, and should be supported at intervals by concrete piers. There is no advantage in covering the pipes with earth. Each curve in the pipe should have a radius not less than 3 or 4 times the diameter of the pipe, it appears from the meager data available relating to this subject.

The end of a discharge pipe often is belled to reduce the exit loss by recovering velocity head. A gradual expansion to double the cross-sectional area of the discharge pipe in a distance of 10 to 20 feet effects considerable reduction in head lost. However, a flap gate attached to the end of the pipe causes loss in head because it disturbs the flow and increases the exit velocity, unless it is lifted out of the way. Where cables have been provided to lift up the gate, careless operators have frequently neglected to use them.

MATERIALS FOR SUCTION AND DISCHARGE PIPES

Riveted or welded plate metal from $\frac{1}{4}$ to $\frac{7}{16}$ inch thick usually has been used for both the suction and the discharge pipes because curves and expanding sections could readily be manufactured and it is economical. The pipes should be airtight, because air leaks reduce the pump discharge by increasing the total head on the pump and by occupying space that would otherwise be used by the water. A few plate-metal pipes under vacuum have collapsed because they were not properly braced, and other pipes did not appear safe because of much pulsating distortion during operation of the pumps. Such "breathing" was especially pronounced near the top of the levee in discharge pipes under high vacuum. Such observations lead to the belief that pipes under as much vacuum as 20 inches of mercury should be braced by angle irons riveted or welded around the outside of pipes more than 36 inches in diameter and of plate metal 1/4 inch thick or more than 54 inches in diameter and of plate metal 3/8 inch thick.

Welded pipes cost at the shop in 1930 10 to 15 percent less than riveted steel pipe, but there may be little difference in the cost to the district. Either kind is satisfactory if properly made. Careful inspection should be made after the pipes are in place, to be sure all joints are airtight, and the pipe should be coated both inside and outside with asphalt paint.

Cast iron has been used to a limited extent for discharge pipes running through the levees, but it is less easily handled and fitted than steel pipe.

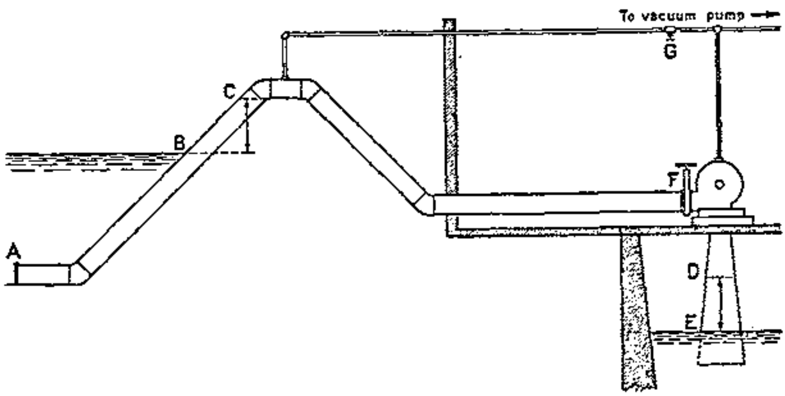


FIGURE 26.—Typical conditions during priming of drainage pump.

Reinforced concrete has been used at some plants for the suction pipes and for sluiceways through the levees. The entrance ends of those pipes usually were cut in vertical planes which necessitates deep suction bays to prevent the pump from taking air before the water is lowered sufficiently in the suction bay. A few of these plants have placed hoods over the entrances to obtain lower water levels. Trash wedging in such a suction bay cannot be removed easily. Therefore concrete suction pipes are not recommended.

VALVES OR GATES IN THE DISCHARGE LINE

Some type of valve or gate must nearly always be installed in the discharge line of a pumping plant to facilitate priming and to prevent flooding the district by back flow when the pump is stopped. A gate valve placed at the pump flange or a flap gate at the end of the pipe is used most frequently.

Typical conditions of priming during high river stages are illustrated in figure 26. If there is no effective gate or valve in the discharge line operation of the vacuum pump will raise the water in the discharge pipe from B to C and at the same time raise the water in the suction pipe an equal distance from E to D. Then water from the discharge bay will commence to flow down through the pump, and often will cause a centrifugal pump to revolve backward. When this occurs it may be difficult or impossible to fully prime the pump before starting, and if a centrifugal pump is not completely primed before starting air is likely to be trapped around the impeller shaft, which reduces the pump discharge. On the other hand, a reasonably tight flap gate at A will keep the outside water from being drawn over the hump at C during priming. If there is an airtight valve at F, by closing it and the valve at G the pump can be primed more rapidly.

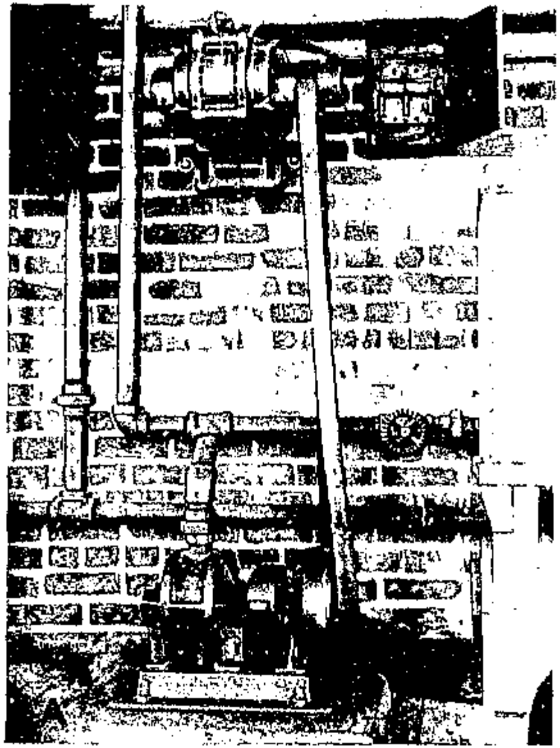
The screw pump and the mixed-flow pump can be started before completely primed without danger of trapping air in the pump. Therefore with these types backward flow of water can be avoided without any gate or valve in the discharge pipe if it runs over the top of the levee. However, a flap gate is always helpful in preventing sudden loads. A gate valve at the discharge flange of the pump is used with some types of oil engines and with synchronous and squirrel-cage motors because it permits the load to be built up gradually. Special butterfly-type check valves were used in a few of the earlier upper Mississippi River plants. This type folds up into a compartment when the pump starts and offers practically no resistance to flow, and closes automatically when the flow of water reverses. It can be placed in the discharge line where most easily accessible for cleaning and repairs.

A flap gate is cheaper than a gate valve, and is satisfactory if the starting torque of the motor or engine is sufficient to pump against the maximum load. The flap gate is automatic, while a hand-operated gate valve is frequently difficult to open and close. A flap gate is easily fouled with logs or trash, and difficult to clean at high river stages when most needed; it is easily broken by boats, and difficult to repair because under water. In 1929 at the Bay Island plant the discharge-bay level rose until water flowed backward through the discharge pipe because the flap gate was out of order when it was necessary to stop the pump. The flow was stopped by a dam of boards, canvas, and sand bags built in the pipe at a manhole. Therefore, where the river stage may get high enough to cause back flow into the district and no gate valve is to be used, it is desirable that a manhole be placed inside the levee near the top point.

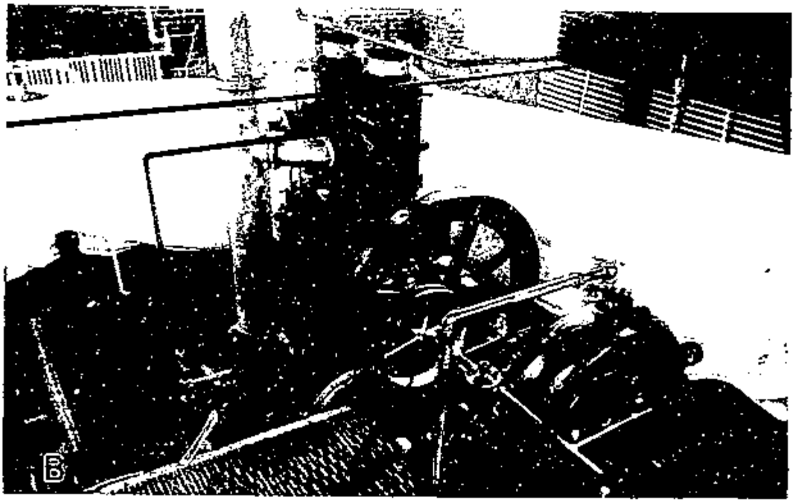
Gate valves smaller than 36-inch may be opened and closed by hand (pl. 2, B), but hand operation of large valves is so difficult and slow that it is considered desirable to equip 36-inch and larger valves with mechanical-operating devices. A number of such devices are commercially available. Electrical devices are most used in electric plants, and pneumatic devices (pl. 6, C) are frequently convenient in oil-engine plants because such plants ordinarily include an air compressor and air tanks for starting the engine. A mechanically operated valve should be equipped for hand operation in emergencies.

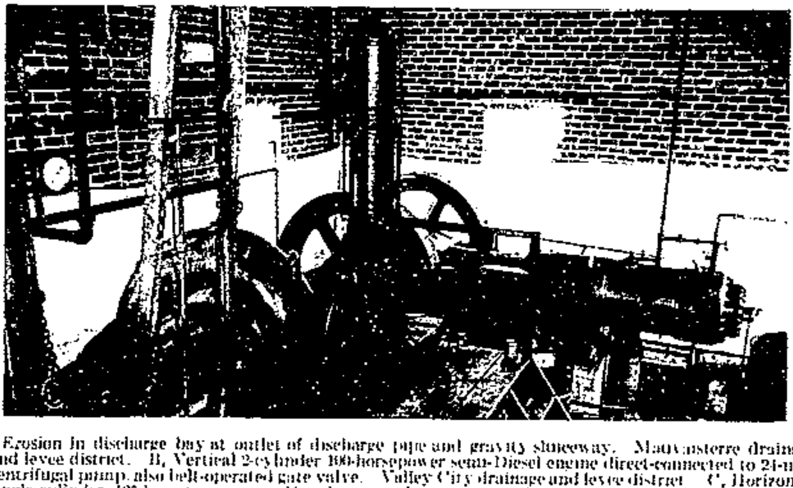
SELECTION OF POWER EQUIPMENT

In the upper Mississippi Valley, drainage pumps are driven by electric motors, internal-combustion oil engines, or steam engines. However, no steam plants have been installed in recent years for draining farm lands, principally because of high investment costs and the



A, Wet-vacuum priming pump driven by a small electric motor. Marion County drainage and levee district. B, Dry-vacuum priming pumps with tank in suction line to trap out water. Consolidated drainage and levee district.





A, *Erosion in discharge bay at outlet of discharge pipe and gravity sluiceway.* Mauvaisterre drainage and levee district. B, Vertical 2-cylinder 100-horsepower semi-Diesel engine direct-connected to 24-inch centrifugal pump, also belt-operated gate valve. Valley City drainage and levee district. C, Horizontal single-cylinder 125-horsepower semi-Diesel engine direct-connected to 36-inch centrifugal pump; also pneumatic gate valve. Des Moines-Mississippi drainage and levee district.

large fuel losses involved in frequent starting and shutting down of the plant. Conditions during and since the period of this investigation have favored the selection of electric or oil-engine equipment for new installations to such a degree that a discussion of steam equipment is not included in this report. Where and when a cheap supply of natural or artificial gas is available, internal-combustion gas engines may be economical, but none such has yet been used for drainage pumping in this section.

The factors affecting choice of power type for drainage pumping in the upper Mississippi Valley have been discussed in another publication (*4, pp. 96–97*). In 1930, pumping with oil engines cost somewhat less than pumping with electric power, except at low plant factors, but electric power was generally more convenient and dependable and plant operation was simpler. A district having 2 or 3 separate plants might have the smaller ones automatic in operation and need to employ but one operator if electric power is used, or have the smaller plants operated by nearby farmers and maintained by the operator of the main plant. An oil-engine plant, however, should not be trusted to an inexperienced operator.

The required capacity of the engine or motor may be determined by the formula

$$P=\frac{0.0002526HQ}{e_pe_t}$$

in which,

P=brake horsepower required,
H=total head on pump, in feet,
Q=discharge in gallons per minute,
e_p=efficiency of pump,
e_t=efficiency of power transmission between engine and pump.

When the power requirement has been determined, some adjustment in the speed and capacity of the pump usually is necessary to fit the load to a power unit of commercial size.

ELECTRIC EQUIPMENT

A company selling electric power nearly always provides the line and transformers for delivering the power to the drainage plant at the voltage needed. The drainage district usually must furnish the accessory equipment including switchboard, starting equipment, motors to drive accessory equipment, and transformers for reducing the voltage for driving the accessory motors and for lighting.

EFFECT OF POWER RATES ON PLANT DESIGN

The terms under which the power will be purchased are often important in determining the economical design of an electric plant. The cost of power often includes both a primary charge, based on the capacity of pumping motors, to compensate the selling company for its capital investment, and a current charge, based upon the amount of energy used, to pay for generating and distributing the power. In some cases no primary charge is made, but the current charge is comparatively high for the first few thousand kilowatt-hours used each month and gradually decreases with increase in quantity used. Many contracts permit motors that are not being used to be sealed off, and the primary charge to be reduced proportionally. It is often required that the units be kept sealed off for a year or longer before the reduced rate becomes effective. This frequently can be done without detriment to the drainage.

Where reduced rates may be obtained by sealing off motors, the plant should contain 3 or 4 units of equal size rather than 1 or 2 larger ones. When 3 or more units are used, 1 can be sealed off for a year much more frequently than when only 2 units of equal size are used. It is further desirable to have all units equally efficient, so that any one can be sealed off without decreasing the average plant efficiency. To avoid deterioration of motors through long idleness, it is usually desirable to seal off the motors alternately rather than have the same one unused for several years. Where sealing off of units is not contemplated, it usually is economical to install a less expensive and consequently less efficient unit for pumping at high heads only, since the maximum plant capacity is needed only at high river stages and for but a small part of the time.

Where the primary charge is the greater part of the power cost, plant efficiency is of less importance than where the only charge is for current at a higher rate per kilowatt-hour. The greater the charge per kilowatt-hour, the greater the investment is justified for increasing the plant efficiency. A rate based only on current used makes gravity drainage especially desirable if it can be obtained.

A large primary charge increases the importance of installing motors no larger than required. Motors sometimes have been overloaded from 10 to 15 percent at the maximum lift, and fans used to cool the motors in order not to increase the primary charge. Such practice should not be contemplated, however, without the manufacturer's advice as to the effect of such overloads on the motor. Primary charges based on actual maximum demand rather than on the rated horsepower of the motors would reduce the temptation for a district to buy inadequate equipment.

INDUCTION MOTORS

The wound-rotor induction motor was used almost exclusively in the electric drainage plants on the upper Mississippi River prior to 1928. The widespread use of this type was due to its dependability, its high-starting torque, and the fact that the starting current does not greatly exceed the operating current. Many power companies insist that this type be used instead of squirrel-cage motors because of the better starting characteristics.

Although there are several methods of changing the speed of an induction motor, the type is inherently a constant-speed machine. In nearly all the earlier electric plants changes of the pump speed were made by changing pulleys on the motor shaft. Often this is the most economical method theoretically, in spite of the fact that belt connection reduces the efficiency of the unit by 3 to 5 percent as compared with direct connection. The great difficulty with belt-connected units has been that operators would not change the pulleys, and by running the pumps at high speeds at low and medium lifts would waste 10 to 25 percent of the current. The motor speed can be regulated by introducing resistance into the rotor circuit, but certain tests showed a cost for driving a 48-inch pump with an induction motor at reduced speed about $8 greater per 12-hour run than for pumping the same quantity of water with a duplicate pump driven at the same speed by a synchronous motor. The induction motor was less efficient principally because of heat losses in the grids when the speed was reduced.

The squirrel-cage motor is the cheapest type of electric motor. Its operating characteristics are similar to those of the wound-rotor motor, but it has a low-starting torque. For this reason power companies do not permit it to be used in many locations, because in starting it requires such a large current at low-power factor that the operation of other motors on the same line may be affected.

SYNCHRONOUS MOTORS

The synchronous motor has several advantages and some disadvantages for drainage pumping in comparison with the wound-rotor induction motor. The synchronous motor is slightly more efficient; it has a larger air gap, so does not require such exact alinement of the shaft; the power factor can be kept at 100 percent or varied at will. A disadvantage of the synchronous motor has been its low starting and pull-in torque, but now 100 percent or greater starting and pull-in torque can be obtained by improvements which, however, add to the cost of the motor. Since 1928 several drainage pumping plants in the upper Mississippi Valley have installed synchronous motors. In some installations of two synchronous motors of different speeds mounted on the same shaft, so the speed of the pump can be changed without changing pulleys, both motors have been of the same rating although nearly always a smaller motor could be used for the lower speed.

Two synchronous units on the same shaft are frequently more economical than a wound-rotor motor with belt drive and the pulleys needed for speed adjustment. In 1930 the cost of two synchronous motors of 225 and 277 revolutions per minute and 200 and 250 horsepower with accessory equipment was about $1,500 more than the cost of a comparable induction motor and belt drive. The efficiency of the synchronous unit probably averages from 5 to 10 percent higher, partly because the pulleys often are not changed when necessary for economical pumping. The saving by even 5 percent increase in operating efficiency and the saving in housing space and in maintenance charges of belts usually would justify the extra cost of the direct-connected twin synchronous unit at as much as 10 percent plant factor.

One important advantage of the synchronous motor is that its power factor is ordinarily unity, but can be varied when some advantage will result to the power-distribution system. However, no price concessions for the use of synchronous motors have been made in power rates in the upper Mississippi Valley, but usually a power factor below 80 percent is penalized.

ACCESSORY EQUIPMENT

One of the most expensive items of accessory equipment in an electric plant is the switchboard with, in addition to the numerous switches, the various watt-hour meters, overload releases, ammeters, and starting and synchronizing mechanism according to the size of the plant and character of equipment installed. Other accessories needed in an electric plant include lighting equipment and small motors to drive priming pumps and water pumps. Water for domestic use of the plant operator and for water-seal glands of the pumps usually is obtained from driven wells.

OIL-ENGINE EQUIPMENT

The ultimate economy of oil engines depends primarily upon getting reliable engines at reasonable cost, which will operate with low fuel costs and will last long without excessive repairs. Similar care should be exercised concerning the accessory equipment. Oil engines should be compared with electric motors on the basis of total cost of pumping, including the fixed charges of depreciation and interest on investment.

To insure that repair parts and expert service will be obtainable when needed, a district should buy the product of a manufacturer with a good financial standing and who has been making oil engines for a period of years. In one exceptional instance a district spent more in overhauling its engine of a type that had not been manufactured for some years than a new engine would have cost.

One advantage of an oil engine is that the speed can nearly always be changed sufficiently by a governor to operate a drainage pump efficiently at both low and high heads. Another advantage is that most oil engines are rated to operate at about the same speeds as drainage pumps, which permits direct connection and the avoidance of belt or chain losses.

DIESEL AND SEMIDIESEL ENGINES

There is little choice between diesel and semidiesel engines for drainage pumping. The diesel engine has slightly lower fuel-oil consumption per brake horsepower-hour than the semidiesel, and is easier to start because it does not require a heated bulb. The recent trend has been toward the diesel type although it is usually slightly higher in first cost.

Oil engines with vertical cylinders (pl. 6, B) vibrate less, and their cylinder walls wear less rapidly, than those with horizontal cylinders (pl. 6, C). Excessive vibration may result in damage or higher depreciation for engine, pump, or buildings, and is especially objectionable for drainage plants because so frequently foundation conditions are poor.

Most of the few multiple-cylinder horizontal oil engines that have been installed in drainage plants have rendered unsatisfactory service. The difficulty of keeping the bearings in alignment on poor foundations with the excessive vibration no doubt has been the principal cause of those failures. The experience of those plants indicates that single-cylinder belt connected horizontal oil engines from 50 to 150 horsepower may be used when more economical than other types, but for engines of more than 150 horsepower it is much safer to use the multiple-cylinder vertical type.

ACCESSORY EQUIPMENT

An adequate supply of cooling water is needed for oil engines. Cooling systems are classified as "open" when the water once run through the engines is wasted, and as "closed" when the water is circulated through the engines time after time and cooled between times in tanks of cold water or radiators. The open system requires a large supply of clear water free from impurities, which is seldom available at a drainage pumping plant. Consequently, a closed system should nearly always be installed. This system requires the constant addition of small quantities of water to replace that which evaporates.

Apparatus to precipitate impurities in the cooling water are not commonly installed, but it is recommended that future installations include equipment for doing this and for filtering the water. In several plants the use of untreated water for cooling has caused deposits of scale in the cylinder head and cooling jacket. Scale causes excessive wall temperatures, which may break down effective lubrication of the pistons and cylinder liners and cause excessive wear, distortion, or cracking of these parts. Scale is usually responsible for the cracking of cylinder heads; overheating of the pistons may cause increased friction and incomplete combustion of fuel and loss of power of as much as 25 percent. Scale accumulating in pipe lines increases the power necessary to circulate the water and decreases the effectiveness of the cooling device. Effective water softening and filtering apparatus can be purchased ready to connect to the cooling lines.

If the pumping plant is close enough to a railroad siding, considerable saving can be effected by piping the fuel oil from tank cars into the storage tanks instead of hauling it in trucks or barges. Because fuel oil is cheaper if bought in carload lots, it is desirable to provide sufficient storage space to permit purchasing an additional carload before the current supply is exhausted. At several plants two tanks, each of about 8,000 gallons capacity, have been used and found economical in first cost and of adequate size for convenient plant operation.

FUEL OIL

All the drainage plants studied burned the lighter fuel oils, which cost very little more than those heavier than approximately 32° Baume. With the heavy oils, use of a lighter oil in starting and stopping the engines generally is advantageous. This obtains more complete combustion, which prevents carbon deposits on piston and cylinder heads and avoids formation of sulphurous-acid gas that may corrode the cylinders and valves. However, if two grades of oil are used, the first cost of the plant is increased by the amount necessary to provide separate storage and piping for the two oils. It is doubtful whether this additional investment is justifiable for drainage pumping because the plant factors are so low.

DESIGN OF BUILDING AND ACCESSORY STRUCTURES

FOUNDATIONS

Good foundation conditions are of primary importance in selecting the location for a pumping plant. The natural location is near the lowest point in the district, which frequently is a soft swamp entirely unsuited for the purpose. Under such conditions it may be necessary to locate the plant on a nearby ridge, and extend the main drainage ditch to it. The plant should be located on the inside of the levee, and as close to it as practicable so that the discharge pipes will not be unnecessarily long, but not so close that the suction bay will weaken the levee. Inadequate foundations have frequently caused pumping units to get out of line and building walls to crack.

Most pumping drainage districts in the upper Mississippi Valley are underlain by quicksand at depths of 3 to 15 feet. The pumping plant should be located where excavation of quicksand in constructing the suction bay will be avoided as far as possible.

Piling should be used under the machinery foundations, the building walls, and the suction bay, and be designed to support the full load that will rest on the piles. If the plant is to contain oil engines, the piling should be sufficient to withstand much vibration.

Considerable difficulty has been experienced with the pump or engine of direct-connected units settling out of line. This has resulted in burned-out bearings, loss of efficiency, expensive repairs,

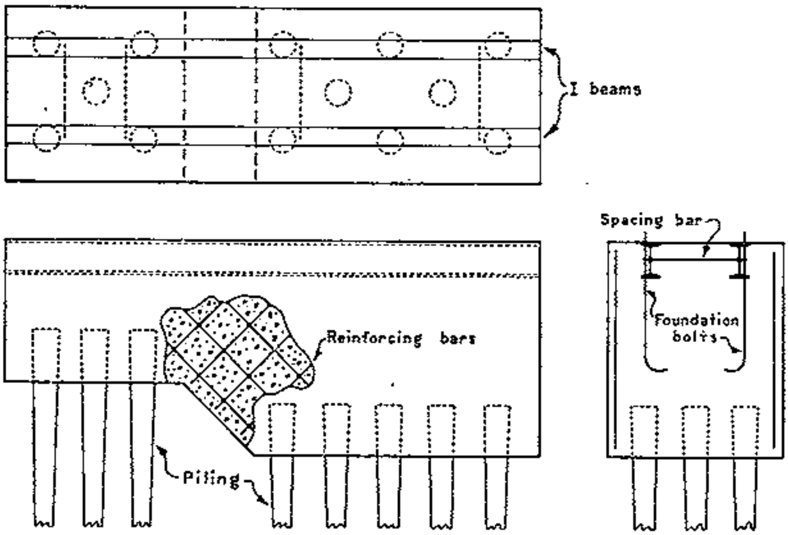


FIGURE 27.—Suggested reinforced concrete foundation for direct-connected pumping unit.

and in some instances loss of use of the pumping unit for long periods. A continuous concrete foundation beneath both pump and engine or motor, reinforced with light steel I-beams and diagonal steel rods as shown in figure 27, would avoid such difficulties. It is believed that had this construction been followed, much of the trouble caused by units settling out of line would have been avoided. Although this construction is somewhat more costly than plain concrete, the extra expense is believed justified.

BUILDING SUPERSTRUCTURE

The pumping-plant building should be strong enough to withstand storms, adequate in size to house the equipment, resistant to fire, warm enough to prevent freezing, permanent, and pleasing in appearance. The arrangement of engines, pumps, and principal accessories in a well-planned oil-engine plant somewhat more completely equipped than the average is shown in figure 28. An arrangement economical of

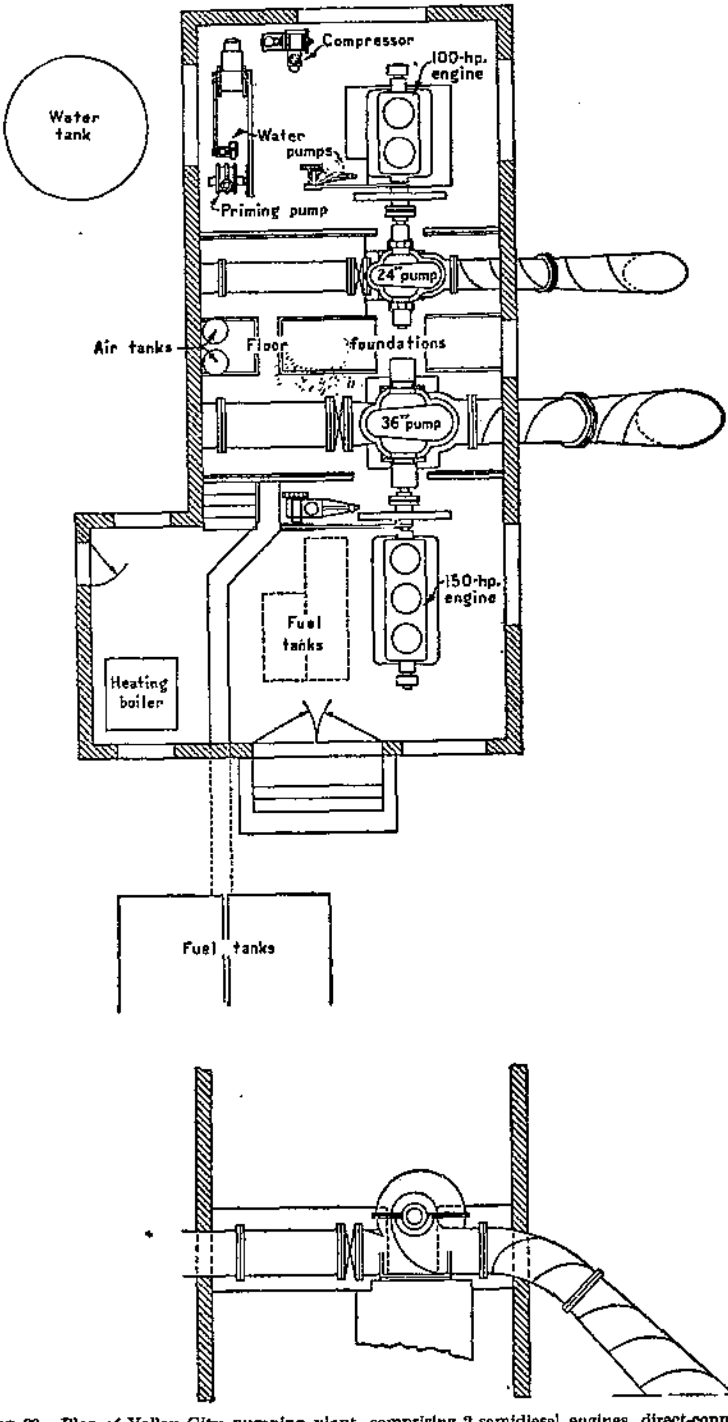


FIGURE 28.—Plan of Valley City pumping plant, comprising 2 semidiesel engines direct-connected to centrifugal pumps and accessory equipment.

floor space is illustrated in figure 29. The belt connection facilitates this arrangement. The relatively sharp bends in the discharge pipe are less objectionable in an oil-engine unit than in an electric unit because fuel oil costs less than electricity per brake horsepower-hour.

Where the building does not extend over the suction bay, brick is often an economical material (pl. 7, A). Where a wall is to be supported on girders over the suction bay, a lighter type of construction is desirable. For such plants a steel-frame building with stucco or sheet-metal walls is well suited (pl. 7, B). For utmost economy in first cost, walls of concrete block or hollow tile can be used (pl. 7, C). Frame construction or galvanized steel on wood frames is not recommended because of the fire hazard. If a pumping plant burns, usually no pumping is possible for several weeks or months, until the equipment is overhauled and repaired. During that time lack of drainage may cause crop losses many times greater than the saving from use of a wooden building.

In a few instances pumping plants have been able to continue operation when breaks in creek-diversion levees have let water stand against the pump-house walls above the floor level, because the building was watertight. Such operation reduced the amount of flooding and crop damage in the district, and prevented damage to the pumping machinery and accessory equipment that would have resulted from flooding of the plant. Watertight construction can be obtained by extending concrete foundation walls above the floor level and making provision to seal the openings in the walls with boards and canvas or sandbags (pl. 7, B). The tendency of the building to float when water stands high outside must be considered in designing the plant for such a condition.

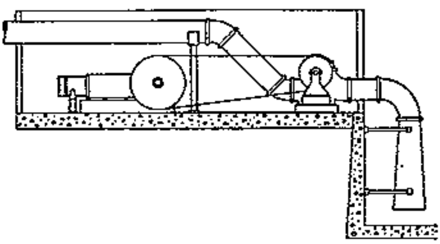


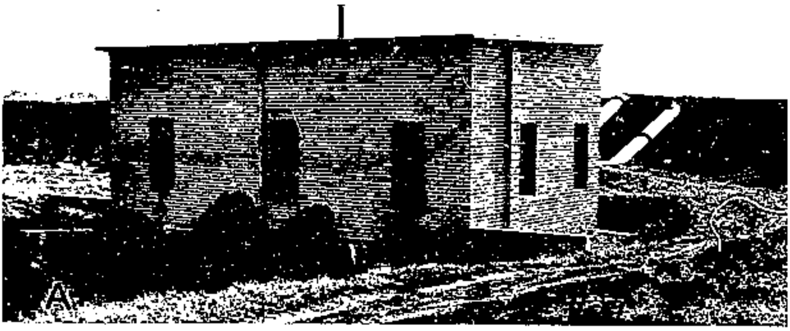
FIGURE 29.—Discharge pipe from centrifugal pump belt-driven by horizontal oil engine in McGee Creek plant is carried over the engine to reduce floor area required.

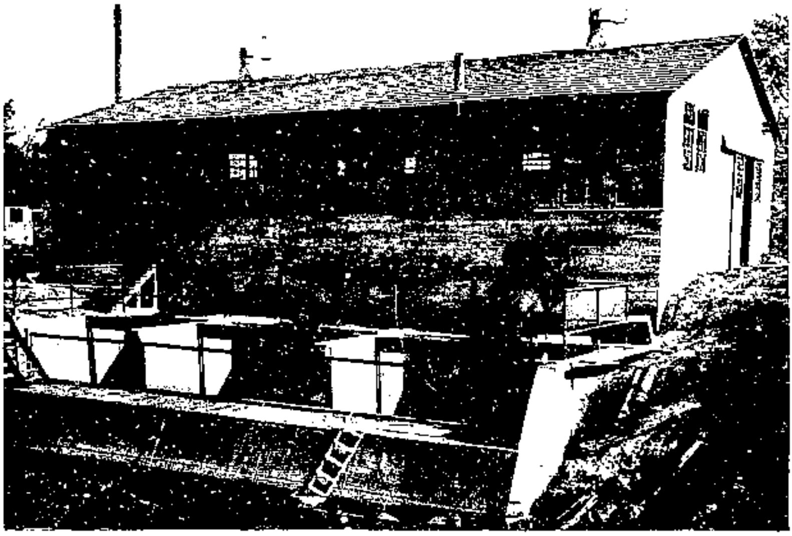
SUCTION BAY

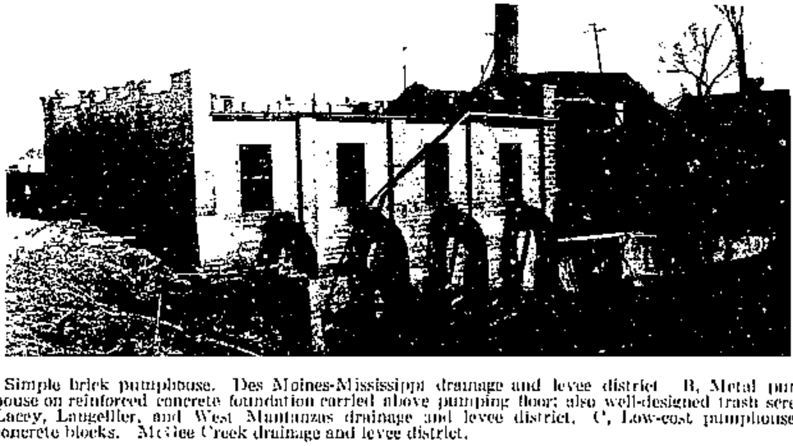
The usual type of suction bay consists of a concrete retaining wall on the suction side of the building cast integrally with wing walls and a heavy concrete floor (fig. 30). The wing walls are usually 1 to 2 feet thick and the floor usually about 2 feet thick. The side walls are usually supported on piling. This type of construction, although sometimes expensive, has been found to be effective in preventing undermining of the building, and it is recommended for average conditions in the upper Mississippi Valley where the suction bay extends down into quicksand.

The suction pipes slope down through the back retaining wall of the suction bay, or pass horizontally through the building wall and turn down vertically into the bay. At the McGee Creek plant the clearance allowed between the end pipes and the wing walls was too small, which resulted in vortices forming in the suction bay even when the water was at a high stage.

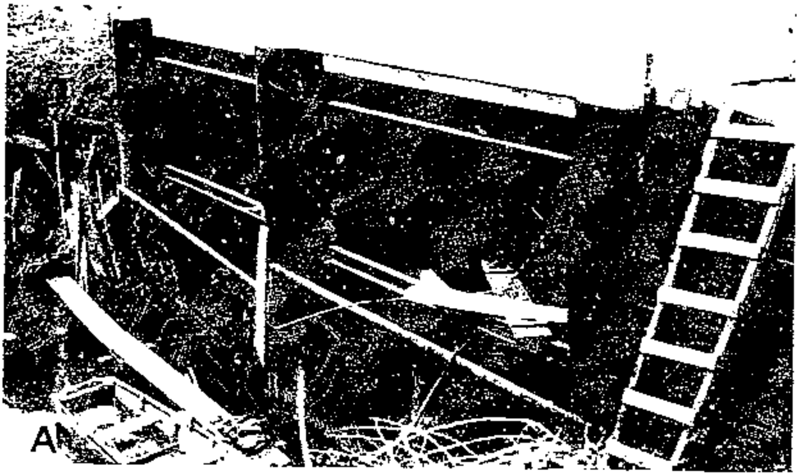
Between the suction pipe and the bay walls and also under the end of the suction pipe there should be sufficient clearance for the water

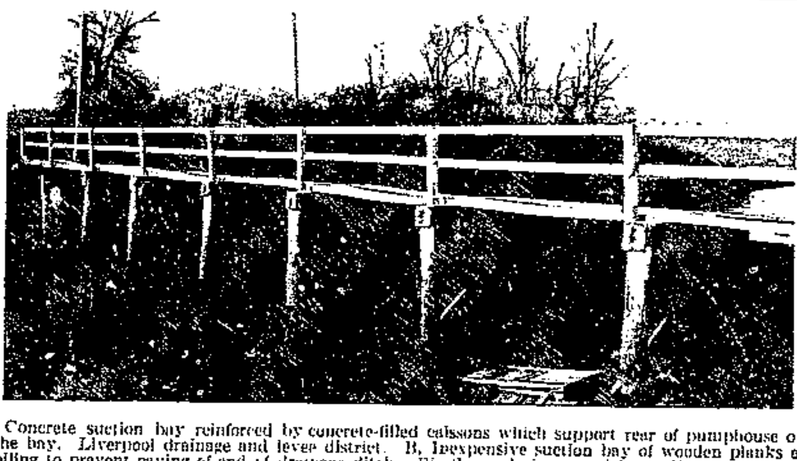






A, Simple brick pumphouse. Des Moines-Mississippi drainage and levee district. B, Metal pumphouse on reinforced concrete foundation carried above pumping floor; also well-designed trash screen. Lacey, Langellier, and West Matanzas drainage and levee district. C, Low-cost pumphouse of concrete blocks. McGee Creek drainage and levee district.





A, Concrete suction bay reinforced by concrete-filled caissons which support rear of pumphouse over the bay. Liverpool drainage and levee district. B, Inexpensive suction bay of wooden planks and piling to prevent caving of end of drainage ditch. Big Swan drainage and levee district. C, Trash screen upstream from suction bay supported by substantial wooden trestles. Hunt drainage and levee district.

to flow into the pipe uniformly from all sides. The clearance between the suction pipe and the floor should be at least one half, and preferably two thirds, the diameter of the entrance opening of the suction pipe. A minimum clearance of 2 feet should be allowed under the pipe in order that trash, boards, and logs that become wedged under the pipe can be removed readily. Between the bottom edge of the suction pipe and the side walls of the bay the clearance should at least equal the entrance diameter of the pipe. Between the back wall and the suction pipes a clearance of one half to three fourths the entrance diameter seems adequate. The spacing of suction pipes at the entrance ends should be, on centers, not less than 4 or 5 times the diameter of the pipes at the pump flanges. In designing suction bays it is well to keep the velocity of water down at all points to less than 3 feet per second during operation at full capacity and optimum stage of the suction bay.

A safe type of construction used at a number of plants is shown in plate 8, A. Open steel caissons were first sunk to below bay-floor level where the corners of the bay would be, and between as deemed desirable. Round piles were driven in the bottoms of the caissons,

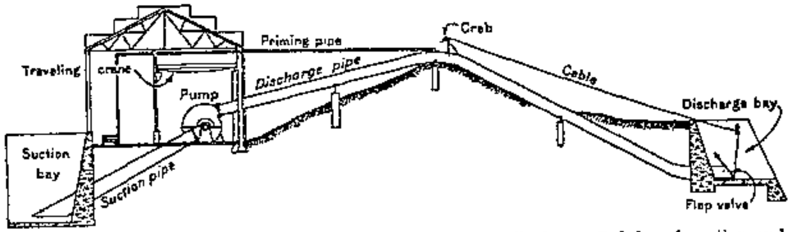


FIGURE 30.—Elevation of well-designed pumping unit in Hartwell plant. Reinforced suction and discharge bays; horizontal entrance opening to inclined suction pipe; discharge pipe passing over top of levee, with long-radius bends; crab for lifting flap gate out of flow from pipe.

and the steel shells filled with concrete. Sheet piling around the site prevented caving during excavation between the piers and during construction of the walls which are supported on round piles. Steel braces were placed between the piers as excavation proceeded. This construction permits setting the pumps on steel girders directly over the bay, but costs more than some other types.

At small plants where there is no danger of undermining the building, the concrete floor of the suction bay may be omitted. At some plants the suction bay is the end of the drainage ditch protected by planks and wooden piling (pl. 8, B). Steel sheet piling is more permanent. Such a bay should be placed a little distance from the building, to lessen the possibility of the building being undermined. Where such construction is safe, it effects a considerable saving in the cost of the plant.

DISCHARGE BAY

Concrete discharge bays have been built at some plants to protect the outside of the levee from erosion, but usually no serious erosion has occurred where no bay was constructed. A concrete structure protects the pipes and flap gate from damage by boats, and anchors the flap gate. The flap gate usually is light enough, however, to swing free on the end of the pipe if that is braced with angle iron and supported on a few piles. At the Mauvaisterre plant, erosion at the discharge was not great after the plant had been operating for 4 years (pl. 6, A), though the velocity of discharge had been 10 to 12 feet per second and the levee was very sandy. That erosion could have been largely prevented by a few tons of riprap at the end of the pipe, which would have cost less than extending the pipe 5 to 10 feet farther from the levee. The undermining of the gravity sluice outlet structure should have been prevented by a cut-off wall, sheet piling, or riprap.

GRAVITY SLUICEWAYS

If sufficient gravity drainage can be obtained so that construction of a sluiceway through the levee will be economical, the structure should be located a few hundred feet from the pumping plant. There is always some danger that pressure of the river water will open a channel along the sluiceway, and the vibration of pumping is likely to loosen the earth about any pipe or structure through the levee near the plant. In some districts the sluiceway has extended under the pump house. The Gregory plant near Keokuk, Iowa, was constructed in this manner, and was destroyed by a levee break in 1922. Part of the Indian Grave plant was similarly destroyed about 1920. Seepage along the sluiceway probably caused both of the levee failures. If the sluiceways had been some distance away, the plants probably would not have been destroyed by the rush of water. Serious leaks developed along a similar sluiceway at the Bay Island plant and the sluiceway was closed off.

A run-off capacity of 0.20 inch per 24 hours over the watershed area is believed adequate for a gravity sluiceway built to supplement a pumping plant. When the run-off is greater, the river would probably be high enough to necessitate pumping. An average velocity of 6 to 8 feet through the sluiceway is suggested for design. The difference in water level between entrance and discharge bays necessary to produce such velocities would be between 1 and 2 feet. The top invert of the outer end of the culvert should preferably be that much below the maximum allowable stage in the suction bay to insure that the maximum capacity is available. The bottom of the sluiceway should be not lower than the bottom of the ditches near the sluiceway, to avoid excessive silting in the sluiceway. These conditions may require that the culvert be a shallow and wide box, or be made of several small pipes rather than one of larger diameter. The entrance end of the sluiceway should be rounded to increase the capacity (7).

An iron or steel gate should be constructed on the discharge side of the sluiceway, and be operated by a screw or gear from a handle set above high water. To prevent seepage along the sluiceway concrete collars should be constructed about 10 feet apart and about 12 by 12 inches in size around the sluiceway box or pipe. Heavy concrete sluiceways should be set on piling if the foundation is poor. Substantial head walls should be provided at both suction and discharge sides to prevent erosion of the levee. To prevent undermining of a concrete outlet structure, such as occurred at the Mauvaisterre plant (pl. 6, A) although the average velocity through the culvert probably did not exceed 4 to 5 feet per second, a cut-off wall, sheet piling, or riprap should be used.

TRASH SCREENS

One or more good screens should be provided for each pumping plant to prevent trash from getting into the pumps. These may be of small iron rods three eighths to one half inch in diameter or of rectangular bars usually about ¼ by 1½ inches. The spacing between bars ranges from about 1½ to 4 inches, although 1½- to 2-inch spacing is most common. The area of the screen opening at average operating stages should be from 2 to 3 times the combined area of the suction-pipe openings. If it is too small, the water will head up unnecessarily, the static lift will be increased, and the pumps will lose their priming at a higher stage of the suction bay. The screens are conveniently made in sections 2 feet wide and supported by small channels or I-beams.

The most convenient location for the trash screen is across the front of the suction bay (pl. 7). If the bay does not have a concrete floor, the screen should be set on piling. At a few plants two screens have been used, one across the entrance of the suction bay and the other some distance up the main ditch. The screens need substantial supports (pl. 8, C). Sloping the screen facilitates cleaning it.

CONSTRUCTION OF THE PUMPING PLANT

Competitive bids for construction of pumping plants and purchase and installation of equipment is desirable in order to assure reasonable costs. Care must be taken that State drainage laws are complied with in advertising for bids and letting contracts.

Usually a district can secure better prices by dividing the pumping-plant construction into at least two contracts, one including such work as foundations, suction and discharge bays, and building, and the other including purchase and installation of the machinery and the suction and discharge pipes. The first can usually be done cheapest by some local experienced contractor familiar with the foundation conditions along the river and having his construction equipment close at hand. The second contract should attract competition from manufacturers and dealers in machinery who will not take contracts involving foundation and similar work.

A cofferdam of wood or steel piling is practically always required in the construction of a concrete suction bay. If quicksand is encountered the cofferdam may fill in as quickly as dewatered. One method of avoiding this difficulty has been to excavate the sand to about 1 foot deeper than the bottom of the floor without dewatering the cofferdam, and then lay under water a false concrete floor about 1 foot thick. The concrete should be laid on canvas so that the cement will not be washed out so readily. If the under floor is nearly watertight, the cofferdam can be emptied without difficulty and the suction-bay floor constructed. The under floor should not be considered a part of the suction-bay floor because a thin concrete slab laid under water may have little strength. The bracing for a cofferdam should be carefully planned, as inadequate bracing frequently causes caving that necessitates expensive reconstruction. After the suction bay has been completed, the sheet piling at the front of the bay should, if possible, be driven down flush with the floor to help prevent undermining of the bay and the building.

After the suction bay has been completed, construction of the other foundations can be done with comparative ease because they do not go nearly so deep. Construction of the superstructure and installation of the equipment are not particularly difficult.

TESTS OF PUMPING PLANTS

Acceptance tests should be made of all drainage pumping equipment to determine if it meets the guarantees of capacity and efficiency. Pumps and engines often can be tested most economically in the factory, in the presence of a representative of the district, and accepted before they are shipped to the plant. However, field tests generally are necessary for pumps larger than 36-inch, because few manufacturers are equipped to test them. Because the equipment required to test oil engines is heavy and expensive to ship, it is usually economical to have the district's engineer test the oil-engine equipment for 24 to 48 hours at the factory, to determine the oil consumption per brake horsepower-hour and whether the engine will operate satisfactorily at full load. New electric motors usually can be accepted without special tests, because their efficiencies ordinarily do not vary more than 2 percent from those guaranteed.

The acceptance tests should determine also whether the priming equipment will prime the pumps in the time specified, which should not be more than 15 minutes. The joints of the suction and discharge pipes should be inspected to be sure that they are airtight; considerable reworking of welded field joints may be necessary. Before acceptance there should be a thorough and detailed inspection of all the machinery to detect flaws, and all accessory equipment should be operated to be sure that it performs satisfactorily.

After the plant has been accepted and put into operation, occasional tests are desirable to determine whether the plant efficiency can be improved by changing operating conditions, such as by adjusting speeds and by repairing or giving better care to the equipment. The methods and equipment recommended for testing pumping plants have been described elsewhere (*4*).

OPERATION OF THE PUMPING PLANT

Great care should be used to select competent and trustworthy operators for drainage pumping plants. The rapid deterioration of several plants can be traced to improper operation resulting from the employment of inefficient instead of efficient operators. Keeping the plant clean reduces the fire hazard as well as improves the appearance.

Even though there is a well-maintained trash screen, sticks and trash often lodge in the pumps or pipes which decrease the plant efficiency and may endanger the equipment. Although sticks often can be heard immediately after they get into a pump, it is not always possible to determine whether the pump is clean except by opening it. The district officials should see that the operator inspects the pumps at frequent intervals and keeps them and the pipes clean.

The trash screen should be kept free from trash to prevent water heading up unnecessarily. The packing on the pumps should be renewed about once a year, and be kept sufficiently tight that leakage of air into the pumps will not be large, yet not so tight as to cause excessive wear of the pump shaft. Air in the pipes reduces the vacuum and increases the head against which the pump operates.

Centrifugal pumps should not be operated empty if any of the parts depend upon water for lubrication. In priming them, they should be completely filled with water, for any air in the casing may be pocketed around the shaft and reduce the discharge. Screw pumps and mixed-flow pumps can be primed satisfactorily after they are started.

The seal glands of pumps frequently become clogged with silt or grease and permit much air to get into the pump, which causes a decrease in discharge and efficiency and rapid wearing of the packing and shaft. Priming is more difficult when the seals are neglected. The operator should understand the purpose of the seal glands and be industrious enough to keep them working properly.

It frequently happens that wear, accident, or other cause reduces the efficiency of a pump very greatly. Occasional tests will show when the efficiency is abnormally low, and then inspection can be made to determine the character of repairs needed, which may be renewal of the impellers. The most efficient unit or units in a plant should be used for as much of the pumping as possible.

In plants that have considerable variation in static lift the speed of pump should be carefully regulated to obtain a high plant efficiency. The importance of this has been shown on page 30. The management of a district should determine the most efficient speeds of operation, by testing the plant or from the characteristic curves, and see that the pumps are run at the most economical speeds. Many operators who are paid by the month want to get their work done in the least time possible, and consequently overspeed the pumps at low and average lifts. When there is danger of flooding crops, the pumps should be speeded up to obtain quick removal of the water. Also, for an electric plant operating under an off-peak agreement it may be advisable to speed up the pumps to finish the pumping during the off-peak hours. If operating at slow speed increases the cost of labor, the extra cost should be considered in determining the economical speed. In some instances continuous slow pumping to keep the water low in the ditches may improve the drainage of some parts of the district.

Operation records are valuable court evidence to determine the need for or the amount of pumping, when special assessments are being levied or during law suits, as well as in supervising the operation of the plant. Daily records should include at least the following items: (1) Discharge-bay gage readings, twice daily or when pumps are started or stopped; (2) suction-bay gage readings at the same time; (3) time of starting and stopping pumps; (4) speed of pumps; (5) watt-hour meter readings, in electric plants; and (6) rainfall. The amount of fuel on hand should be reported monthly or semimonthly for oil-engine and steam plants. Repairs to plant and equipment and supplies received and expenses incurred also should be noted on the operator's daily sheets unless a separate procedure is preferable for checking expenditures.

Adequate operation records, accurate cost records, and occasional tests are absolutely necessary for proper supervision and economical operation of the pumping plant. Such data often enable large savings to be made in operating costs.

SUMMARY AND CONCLUSIONS

In a study of 17 representative drainage pumping plants in the upper Mississippi Valley, extending over a period of 6 years, daily records of operation were obtained to determine the amounts of pumping, static lifts, and costs of pumping, and tests were made to determine the pump discharges and plant efficiencies. In the fall of 1930 a survey was made of all the pumping plants being operated by organized drainage districts in that section. The discussion of costs has already been published (*4*).

A drainage pumping plant should be designed to pump the estimated maximum run-off rate at the maximum lift. In the upper Mississippi Valley, minimum lifts range from 0 to 9 feet, and maximum lifts range from 12 to 25 feet. The average lift during the low-water season for a period of years should be the basis for designing a low-lift pumping unit in a multiple-unit plant.

The annual run-off to be pumped should be determined in order to estimate the annual plant factor and cost of pumping. The estimated average run-off pumped from districts that were affected by backwater from dams ranged from 16 to 35 inches per year, and from districts not affected greatly by backwater ranged from 12 to 16 inches. The annual run-off from districts that obtained some gravity drainage and were not affected by backwater from dams was estimated as from 5 to 15 inches.

The distribution of run-off according to static lift should be determined in order that the plant can be designed for maximum efficiency at the lift at which the greatest amount of pumping must be done, and that one or more units can operate efficiently at any lift.

The required capacity for a drainage pumping plant in the upper Mississippi Valley, at maximum lift, in terms of average total run-off pumped per year and a coefficient varying with size of drainage area and degree of protection provided for the lands, may be computed (p. 25) as

$$C=K_1(0.22+0.006r)$$

A formula for general application (p. 25) is

$$C=K_1[K_2+K_3(r-a)]$$

Screw pumps are especially efficient against heads less than 10 feet and mixed-flow pumps are very efficient against heads ranging slightly higher, whereas for heads greater than about 14 feet properly designed centrifugal pumps obtain equal or better efficiencies. Centrifugal pumps have a greater capacity near the maximum lift than the other types of the same rated sizes. Some plants operating against considerable ranges in lifts have installed both screw or mixed-flow pumps for efficiency pumping at low and medium lifts and centrifugal pumps for efficiency at high lifts. Well-designed drainage pumps have efficiencies slightly above 80 percent near the maximum lift, and operating at two speeds can maintain an efficiency of 70 percent or greater against heads ranging from 7 to 25 feet.

The speed of all three types of pumps should be varied according to the lifts or heads pumped against, to obtain best average efficiencies. At low and medium lifts the speed should be reduced, in some cases by as much as 25 percent, from the most efficient speed for high-lift pumping. Oil engines usually can be varied in speed by adjustment of the governors sufficiently to permit proper pump-speed adjustment. In electric plants, two synchronous motors of different speeds mounted on the same shaft appear to provide the best method of speed adjustment. Where pumps are belted or chain-connected to induction motors, the most practical means of varying the pump speed is by changing pulleys on the motor shaft.

A unit having capacity small enough to pump at the minimum rate of run-off with reasonable efficiency should be provided, unless there is large water-storage capacity in the vicinity of the pumping plant, in order to hold the water in the suction bay at optimum stage for drainage of the lands in the district without too-frequent starting and stopping of the pumps.

Riveted and welded plate metal is usually the most convenient and economical material for the construction of suction and discharge pipes. The friction head lost in those pipes in the plants studied, as determined from tests, ranged from 1.9 feet to 4.3 feet at 10 feet per second velocity. To reduce entrance losses and permit the suction bay to be pumped low without the pump losing its prime, the end of the suction pipe should be expanded to have from 2 to 4 times the area at the pump flange. Discharge pipes usually should be expanded 2 to 6 inches larger than the pump flange, the transition being made by a short expanding section. The saving in cost of pumping as a result of reduced friction head is considerably greater in electric than in oil-engine plants because the expenditures for electric energy exceed those for fuel and lubricating oils. Discharge pipes should be reinforced, if necessary, to prevent collapse when under a high vacuum.

The discharge pipe should go over the levee or as near the levee top as possible without exceeding the practical suction limit of 28 feet above the lowest stage of water in the discharge bay. The end of the discharge pipe should be below the absolute minimum stage of the discharge bay, so that the pump can be readily primed and so that the discharge pipe will act as a siphon to reduce the head on the pumps. The end of the pipe usually should be belled to reduce the head lost at exit. The horizontal section of pipe at the top of the levee should be short to reduce the amount of air that will be trapped.

Choice of kind of power for a pumping plant should be made according to the total cost of pumping, including fixed charges. Electric plants are cheaper in first cost, whereas oil engines cost less to operate (*4*). The economy of an oil engine depends upon its successful operation for a long period of years, therefore experimental types should be avoided. Multiple-cylinder horizontal oil engines are subject to more vibration than those with vertical cylinders, but single-cylinder horizontal semidiesel engines up to 150 horsepower have been satisfactory in operating drainage pumps.

Foundations should rest on piling designed to support the full load. Reinforced concrete foundations are recommended for direct-connected units, so that pump and engine will not settle out of line. A substantial fire-resistant building of brick or other permanent construction is desirable. A concrete suction bay is usually desirable for upper Mississippi Valley pumping plants, to prevent undermining of the building. An effective screen should be provided to keep trash out of the pumps and pipes.

Construction contracts for a pumping plant ordinarily should be let in two parts, one contract for the buildings, foundations, and accessory structures, and the other contract for the pumping and accessory equipment.

Competent and trustworthy operators should be employed for pumping plants, to obtain efficient operation and avoid rapid deterioration of equipment. Pumps and pipes should be kept free of trash. The screens should be cleaned regularly. The seal glands should be cleaned frequently, to prevent air from getting into the pump and rapid wear of the packing on the shaft. Pump speeds should be adjusted according to the lifts, to obtain economical operation, although after heavy rains it may be desirable to pump the water out of the district as rapidly as possible. Records of operation and expenditures and occasional tests of the equipment are essential for proper operation of the pumping plant, and often lead to large savings. They indicate the most efficient units for doing the major part of the pumping, and the need for repairing equipment or improving speed regulation.

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APPENDIX

The pumping equipment available for land drainage on Illinois River and on the Mississippi between Dubuque and St. Louis, in 1930, is shown in table 7. The locations have been shown in figure 1.

The capacities and types of pumps and power equipment of many of the plants are considerably different from practices recommended at the present time, even where the machinery was the most economical obtainable under conditions existing when it was installed. The data presented in the table will be helpful to any engineer desiring to make a detailed study of those pumping plants that are similar to one that he may have to design.

With but three exceptions, all the pumping units of each district are located in one plant. In the Hillview district, the 5,000-gallon unit and the 22,000-gallon unit are each in a separate plant to drain isolated areas of low land. In the Savanna-York district, the 1,500-gallon unit is located separately to serve a small area not economically drained to the main plant. In the Muscatine-Louisa district no. 13, the 16,000- and 36,000-gallon units installed in 1920 are in one plant draining the lower part while the other three units are located in another plant more economically to drain the upper part of the district.

TABLE 7.—*Drainage pumping plants in the upper Mississippi Valley, 1930*

Key no.[1]	Drainage district	Water-shed area	Engine or motor			Pump			Run-off depth per 24 hours[3]
			Type	Began oper-ating	Rating	Began oper-ating	Size and type[2]	Capac-ity (esti-mated)	
	ILLINOIS RIVER DISTRICTS	*Acres*		*Year*	*Horse-power*	*Year*	*Inches*	*Gal. per min.*	*Inches*
1	Hennepin	4,100	Diesel	1924	150	1910	24	15,000	0.272
			Electric	1924	50	1915	18	6,000	
2	East Peoria	1,550	do	1910	75	1910	18	9,000	.513
				1910	50	1910	15	9,000	
3	Pekin-LaMarsh	3,600	do	1910	100	1910	26	13,500	.340
				1925	75	1925	20	10,000	
4	Rocky Ford	1,600	do	1914	100	1914	18	10,000	.895
				1919	100	1919	26	17,000	
5	Spring Lake	22,500	do	1915	300	1909	48	52,000	.276
			do	1915	300	1909	48	52,000	
			Diesel	1926	120	1909	24	13,000	
6	Banner Special	9,760	Electric	1916	250	1916	36	36,250	.366
				1916	75	1916	20	10,500	
7	East Liverpool	3,350	do	1923	150	1923	30	18,000	.471
				1923	75	1923	20	11,750	
8	Liverpool	3,700	Semidiesel	1922	135	1922	20	9,000	.258
				1922	135	1922	20	9,000	
9	Thompson Lake	6,350	do	1927	220	1922	30	22,000	.367
			Electric	1930	200	1922	30	22,000	
10	Kerton Valley	1,740	do	1920	75	1920	20	10,000	.305
11	Lacey	[4]8,640	do	1930	200	1930	30 M	29,600	.409
12	Langellier			1930	125	1930	30	20,000	
13	West Mantanzas			1930	100	1930	24	17,000	
14	Sea Horn	2,005	do	1927	60	1909	20	10,000	.264
15	Big Lake	4,300	do	1914	100	1914	24	12,000	.296
				1929	100	1914	24	12,000	
16	Kelley Lake	1,200	do	1918	75	1918	20	10,000	.972
				1918	100	1918	24	12,000	
17	Lost Creek	2,260	do	1921	100	1921	30	19,000	.446
18	Coal Creek	7,525	do	1914	100	1911	20	12,500	.264
				1914	100	1914	20	12,500	
				1914	100	1914	20	12,500	
19	Crane Creek	6,230	Steam	1912	250	1912	36	30,000	.255
20	Big Prairie	2,200	Electric	1929	100	1914	30	18,000	.434
21	Beardstown Drainage and Sanitary.	800	do	1929	125	1929	24 M	15,000	1.524
				1929	75	1929	20 M	8,000	
22	South Beardstown	8,350	do	1918	100	1918	20	12,000	.382
				1918	150	1918	30	16,000	
				1918	150	1918	30	16,000	
				1918	150	1918	30	16,000	
23	Valley	3,200	do	1916	100	1916	24	12,000	.109
24	Meredosia Lake	5,000	do	1929	200	1929	36	32,000	.488
				1929	100	1911	24	14,000	
25	Coon Run	1,987	Electric	1930	50	1930	20 M	10,000	.267
26	Little Creek	2,000	Steam	1911	90	1911	24	15,000	.716
			Semidiesel	1923	80	1923	24	12,000	
27	McGee Creek	15,700	Steam	1915	225	1915	36	31,000	.401
			Semidiesel	1926	150	1926	30	22,000	
			do	1926	150	1926	30	22,000	
			do	1926	150	1926	30	22,000	
			do	1926	150	1926	30	22,000	
28	Valley City	7,160	do	1922	150	1922	36	25,000	.265
				1922	100	1922	24	10,750	
29	Mauvaisterre	7,150	Diesel	1926	120	1926	30	20,600	.152
30	Scott County	12,500	Steam	1913	420	1913	45	45,000	.352
			do	1913	155	1913	24	16,000	
			Semidiesel	1928	150	1928	30	22,000	
31	Big Swan	15,700	Electric	1910	400	1912	45	45,000	.267
				1917	150	1917	30	21,000	
				1927	150	1912	24	13,000	
32	Hillview	18,500	do	1914	125	1914	30	20,000	.590
			do	1914	125	1914	30	20,000	
			do	1920	260	1929	36 M	41,000	
			Steam	1920	810	1920	60	100,000	
			Electric	1923	50	1923	15	5,000	
			do	1923	150	1923	30	22,000	

[1] For identification of districts shown in fig. 1.
[2] All are centrifugal pumps except 6 indicated screw pumps (S) and 8 indicated mixed-flow pumps (M).
[3] Plant capacity calculated from estimated capacities of pumps.
[4] 3 districts drained by 1 pumping plant.

TABLE 7.—*Drainage pumping plants in the upper Mississippi Valley, 1930*—Con.

Key no.	Drainage district	Watershed area	Engine or motor: Type	Engine or motor: Began operating	Engine or motor: Rating	Pump: Began operating	Pump: Size and type	Pump: Capacity (estimated)	Run-off depth per 24 hours
	ILLINOIS RIVER DISTRICTS—continued	*Acres*		*Year*	*Horsepower*	*Year*	*Inches*	*Gal. per min.*	*Inches*
33	Hartwell	12,900	Electric	1915	150	1915	30	22,500	.554
				1915	150	1915	30	22,500	
				1915	150	1915	30	22,500	
				1929	458	1929	48 M	67,000	
34	Keach (Fairbanks)	18,300	Steam	1910	350	1910	48	55,000	.543
				1927	225	1910	30	30,000	
35	Eldred	9,300	Diesel	1921	250	1921	36	37,500	.522
			Steam	1911	190	1911	36	37,000	
			Diesel	1929	120	1911	24	17,000	
36	Spankey	1,125	Semidiesel	1923	50	1923	18	8,000	.377
37	Nutwood	17,500	Steam	1910	500	1910	48	52,000	.354
			Semidiesel	1925	300	1910	36	30,000	
						1925	36	35,000	
	MISSISSIPPI RIVER DISTRICTS								
38	Carroll County no. 1	3,500	do	1923	135	1923	30	20,000	.470
			Diesel	1928	80	1928	20	11,000	
39	Savannah-York	3,600	Electric	1918	75	1918	30	18,000	.604
				1918	75	1918	30	18,000	
				1927	15	1927	12	3,500	
				1927	15	1927	8	1,500	
40	Meredosia	8,500	Semidiesel	1924	100	1924	24	15,000	.306
				1924	200	1924	36	34,000	
41	Drury	7,000	Steam	1908	250	1908	50	70,000	.530
42	Union no. 1	[1]52,000	do	1909	500	1909	60	115,000	.234
43	Bay Island			1909	500	1909	60	115,000	
44	Keithsburg	1,800	Gasoline	1930	30	1915	12	3,500	.604
			Semidiesel	1915	50	1915	18	8,000	
			do	1920	55	1920	20	9,000	
45	Muscatine-Louisa no. 13	55,000	Steam	1916	400	1916	54	85,000	.249
			do	1916	400	1916	54	85,000	
			do	1916	150	1916	36	36,000	
			Electric	1920	200	1920	36	36,000	
			do	1920	100	1920	24	16,000	
46	Louisa-Des Moines no. 4	16,000	Steam	1909	315	1909	50	63,500	.421
				1909	315	1909	50	63,500	
47	Des Moines County no. 7	30,700	do	1911	350	1911	54	80,000	.415
			do	1911	350	1911	54	80,000	
			Electric	1928	400	1911	54	80,000	
48	Des Moines County no. 8	6,000	Steam	1911	125	1911	24	15,000	.207
				1911	75	1911	18	8,400	
49	Henderson County no. 3	2,200	do	1916	75	1916	20	11,000	.747
				1916	75	1916	20	11,000	
				1917	75	1917	18	9,000	
50	Henderson County no. 1	[1]22,400	Electric	1928	350	1913	48	55,000	.260
51	Henderson County no. 2			1928	350	1913	48	55,000	
52	Green Bay	14,000	do	1918	300	1918	42	47,500	.360
				1918	300	1918	42	47,500	
53	Niota	1,303	do	1917	35	1917	10	3,200	.259
				1917	35	1917	10	3,200	
54	Des Moines-Mississippi	5,500	Semidiesel	1929	100	1929	30	20,000	.482
				1929	125	1929	36	30,000	
55	Hunt	15,840	Electric	1928	250	1928	36	35,000	.256
				1928	250	1928	36 M	41,500	
56	Lima Lake	16,040	do	1928	250	1928	36	35,000	.355
				1928	250	1928	36 M	41,000	
				1928	250	1928	36	35,000	
57	Indian Grave	21,000	do	1918	200	1918	36	36,000	.273
				1918	400	1918	36	36,000	
						1918	36	36,000	
58	Union Township	3,700	Semidiesel	1923	150	1923	30	22,000	.315
59	Fabius	14,900	Diesel	1917	250	1917	42	47,500	.338
				1917	250	1917	42	47,500	
60	Marion County	6,470	Electric	1915	100	1915	30	20,000	.328
				1915	100	1915	30	20,000	
61	South Quincy	10,405	do	1917	200	1917	36	34,000	.347
				1917	200	1917	36	34,000	

[1] 2 districts drained by 1 pumping plant.

TABLE 7.—*Drainage pumping plants in the upper Mississippi Valley, 1930*—Con.

Key no.	Drainage district	Water-shed area	Engine or motor			Pump			Run-off depth per 24 hours
			Type	Began operating	Rating	Began operating	Size and type	Capacity (estimated)	
	MISSISSIPPI RIVER DISTRICTS—continued	*Acres*		*Year*	*Horse-power*	*Year*	*Inches*	*Gal. per min.*	*Inches*
62	South River	11,200	Steam	1911	250	1911	36	32,000	.303
				1911	250	1911	38	32,000	
63	Riverland	6,400	Semidiesel	1921	125	1921	30	20,000	.331
			Diesel	1930	120	1921	30	20,000	
64	Elisberry	25,900	Steam	1915	300	1915	48	56,000	.238
				1915	300	1915	48	56,000	
65	Sandy Creek	1,125	Semidiesel	1923	38	1923	18	8,000	.377
66	Brevator	1,800	do	1920	50	1920	24	10,000	.295
67	East Side Levee and Sanitary:								
	Cahokia plant	2,688	Diesel	1930	90	1930	36 S	20,000	.780
				1930	90	1930	36 S	20,000	
	South plant	33,280	do	1930	240	1930	54 S	50,000	.319
				1930	240	1930	54 S	50,000	
				1930	240	1930	54 S	50,000	
				1930	240	1900	54 S	50,000	

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