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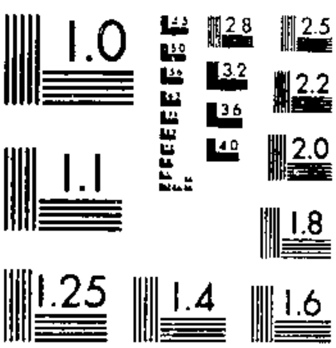

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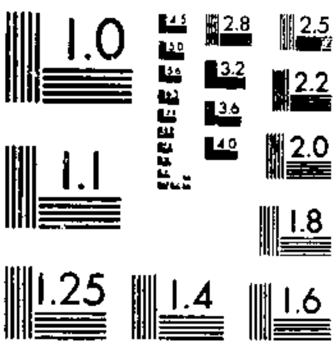

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Technical Bulletin No. 1008—May, 1950

UNITED STATES DEPARTMENT OF AGRICULTURE
WASHINGTON, D. C.

Design and Operation of Drainage Pumping Plants [1, 2]

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CONTENTS

[1] Submitted for publication November 17, 1949.
[2] This bulletin is a revision of and supersedes Technical Bulletin No. 390, Design and Operation of Drainage Pumping Plants in the Upper Mississippi Valley.

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Definition of specific speed and figures 21, 22, and 23.

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SUMMARY

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. The discussion of costs has already been published (*11*).[3] Additional data were collected and analyzed to bring the publication up to date and to widen its application to smaller plants of farm size.

A drainage pumping plant is an integral part of a drainage system. It should be designed along with other drainage works, which nearly always include open drains and levees and frequently tile drains. A drainage pumping plant should be designed to pump the estimated maximum runoff 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. Data that are essential for design may often be secured from the Corps of Engineers, United States Army, the Geological Survey, the Weather Bureau, and the Soil Conservation Service, and other agencies to determine river stages, flow of streams, runoff to be expected from the area, required drainage coefficient, and estimated runoff to be pumped.

The annual runoff to be pumped should be determined in order to estimate the annual plant factor and cost of pumping. The estimated average runoff 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 from 12 to 16 inches. The annual runoff pumped from districts that obtained some gravity drainage and were not affected by backwater from dams was estimated to be from 5 to 15 inches.

The distribution of runoff 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 runoff pumped per year and a coefficient varying with size of drainage area and degree of protection provided for the lands, may be computed (p. 35) as

$$C = .33\ (G + .023\ r)$$

A formula for general application (p. 35) is

$$C = K_1\ [G + (K_2 \times r)]$$

Propeller pumps are especially efficient against heads less than 10 feet and mixed-flow pumps are very efficient against heads ranging somewhat higher, whereas for heads greater than about 14 feet properly designed centrifugal pumps obtain equal efficiencies. Centrifugal

[3] Italic numbers in parentheses refer to Literature Cited, p. 75.

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 propeller or mixed-flow pumps for efficient 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 total 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 provide an excellent 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 runoff with reasonable efficiency should be provided 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.

Welded and riveted sheet steel 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 based on the total cost of pumping, including fixed charges, and on other advantages. Electric plants are cheaper in first cost, whereas oil engines cost less to operate (*11*). 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 semi-Diesel 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 for large plants. 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.

The number of farm pumping plants has increased since the war. The installation of rural electric lines and farm mechanization has encouraged this development. A small pumping plant can be used to drain small areas which cannot otherwise be drained. Attempt has usually been made to decrease the initial cost of such plants and secure automatic operation or to reduce to a minimum the need for attendance while the plant is operating. Light, low-lift, propeller-type submerged pumps have often been used and priming equipment eliminated. Foundations have been lightened, buildings simplified, and the least costly equipment provided. In many cases, annual operating costs have been low and the small amount of pumping required annually would not justify a large additional sum to secure improved plant efficiency. On the other hand it is important for a small as well as a larger plant to operate when necessary to save crops from flooding. The dependability of equipment is thus an important consideration. Essential features such as a screen for trash should be provided, and need for reserve equipment should be considered. Several examples of small farm plants are discussed in the text.

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 avoid rapid wear of the packing on the shaft. Pump speeds should be adjusted according to the lifts to obtain economical operations, 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. They often lead to large savings. Careful records indicate the most efficient units for doing the major part of the pumping and the need for repairing equipment or improving speed regulation.

INTRODUCTION

The national importance of drainage in conserving and utilizing our agricultural lands is receiving increasing recognition. Conservation means using land in accordance with its capability and treating it in accordance with its needs. According to Soil Conservation Service preliminary estimates, farmers are cultivating about 60 million acres that should not be cultivated but should be retired for such uses as grazing and forestry. Most of this land is eroding and is too steep for successful cultivation. As these misuses of land are being corrected large additional areas of land will need to be drained and cultivated as well as developed by clearing and irrigation. As population increases there will be an increasing need for draining good land.

The drainage of lands is particularly important in the conservation program because drained lands are usually flat and not subject to serious erosion. Thus, good conservation means a greater utilization of flat nonerodible lands and shifting of row crops such as corn, cotton, and soybeans to such lands. Steeper lands subject to serious erosion can be used less intensively as other lands are available. This is sometimes done on a farm basis and sometimes it needs to be done for large areas.

The program of soil conservation districts is encouraging proper methods of land drainage. These districts often furnish technical assistance in connection with land drainage by groups and individuals. Such drainage is often accomplished by use of pumps. Technical services, where available, include a determination of the capability of the land, whether it is productive enough to be worth draining, and assistance on design and construction of drains and other drainage facilities such as pumping plants. A plan is worked out with farmers for installation of all practices necessary to conserve the fertility of the soil (*1*).

In connection with the conservation program of soil conservation districts it is likely that the need for drainage pumping will increase. Pumping will be needed to secure proper utilization of the flat lands. Small farm-size pumps are becoming increasingly important in our land-drainage enterprises. They are no more complicated than tractor-drawn farm equipment. Another trend is toward larger pumping plants that have a greater capacity per acre and therefore help to avoid crop losses by securing drainage under conditions of high rainfall. This trend and the need for securing more efficient plants will result in enlarging or replacing many plants.

However, before a new drainage pumping plant is installed the feasibility of the project should be determined. The plant is one part of a complex and somewhat costly development. Such projects usually involve levees, drainage systems, land clearing and development, farm drains, and other farm-conservation structures and practices. Changes in crop prices and adverse economic conditions affect drainage-pumping projects probably more than other kinds of drainage projects because of high operating charges. Pumping and other costs should be compared with income from land drained to determine the justification of the project.

A detailed investigation of the design and operating characteristics of pumping plants in the upper Mississippi Valley was made from 1925 to 1930. The investigations showed 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 stata, and to the extent that drainage ditches cut into those strata. The data collected showed a correlation of plant capacity at maximum lift with average annual amount of pumping. Much operating and cost data gathered in that study were published (*11*).

This bulletin supersedes Technical Bulletin No. 390, Design and Operation of Drainage Pumping Plants in the Upper Mississippi Valley. Recommendations for the design of drainage pumping plants of all sizes and located throughout the United States and the design of farm pumping plants are included in this publication. Agricultural prices have improved materially since the original investigation was made. Consequently, greater returns from the land have encouraged drainage by pumping and by other means. At the present writing, prospects for a high demand for food and good prices appear favorable. Although drainage pumping costs have increased, they have not risen as much as farm income from drained lands. It appears, therefore, that drainage pumping installations that would have been uneconomical when the original publication was issued would now be attractive to farmers and investors.

According to the United States Census of 1940 (*14*), there were 4,367,095 acres in organized drainage enterprises operating pumps in the United States. These pumps had a total capacity of 20,716,025 gallons per minute (g. p. m.), served by power units totaling 102,196 horsepower. In 1940 the rated horsepower capacity of plants included 57.6 percent electric motors, 25.9 percent internal combustion engines, 3.1 percent steam engines, and 13.4 percent plants having two types or other kinds of power. The use of steam decreased from 17,376 horsepower in 1920 to 3,180 horsepower in 1940. The all-electric plants increased from 36,472 horsepower in 1920 to 64,033 horsepower in 1930 and then decreased to 58,917 horsepower in 1940. The capacity of plants having all internal combustion engines increased from 3,916 horsepower in 1920 to 13,118 horsepower in 1930 and 26,464 horsepower in 1940. The capital invested in drainage enterprises operating pumping plants was $125,103,276 to January 1, 1940. Farm pumping plants serving privately owned tracts of less than 500 acres are not included in these figures.

CONDITIONS FOR PUMPING

Individual farmers and those in groups organized for drainage will probably make increasing use of small pumping plants to drain areas that cannot be drained readily by gravity drains. Pumps are used for special conditions, such as the control of water table for muck lands and for lands or crops in humid areas where irrigation during droughts is essential. Pumping for drainage in irrigated areas will no doubt increase because of the special problems encountered in such areas. The better equipment and electric lines now available encourage the installation of small pumping plants to serve individual farms.

Land which is below or nearly level with nearby oceans, lakes, rivers, and streams affords opportunities for successful drainage by pumping. Land may be so flat or irregular in general topography that low areas may be drained more economically by pumping than by the construction of long, deep outlet drains. In irrigated areas, pumping may be needed to lower the ground-water table that has risen to dangerous levels from overirrigation and seepage from canals and laterals. Such pumps may collect water from wells in water-bearing strata or from deep open or closed drains (*9*). Pumping may be the only practical kind of drainage for many areas. For other areas there may be a choice of pumping or of accepting an impaired drainage outlet during part of the year because of high stages of the outlet water levels.

The bottom lands along the Illinois River below Peoria and along the Mississippi River between Rock Island, Ill., and the mouth of the Illinois River originally were 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 60 drainage districts have been organized under State drainage laws. These districts have built levees to keep out the floods, and have installed ditch systems and pumping plants to drain the lands. The extent of pumping for drainage in that region is shown in figure 1, and the major design elements of the pumping plants are given in the appendix (table 9).

These drainage districts usually extend from the riverbank 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 that 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. Most of the lands in pumping districts are so low in relation to river stages that all drainage water must be pumped. Some districts pump only part of the runoff, and during part of the time obtain gravity drainage through sluices.

Low-lift drainage pumping plants are also used for land drainage along the Mississippi River in Arkansas; along the Gulf Coast in Louisiana and Texas; in Florida; in the Pacific Northwest; and in California. Drainage pumping is used in most of the irrigated States. Much of the drainage pumping from irrigated lands is from wells, and requires a greater lift than for the plants considered in this publication. There are a number of smaller farm pumping plants in Florida, Michigan, Ohio, California, and along the Atlantic Coastal plains.

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 planned 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 consider-

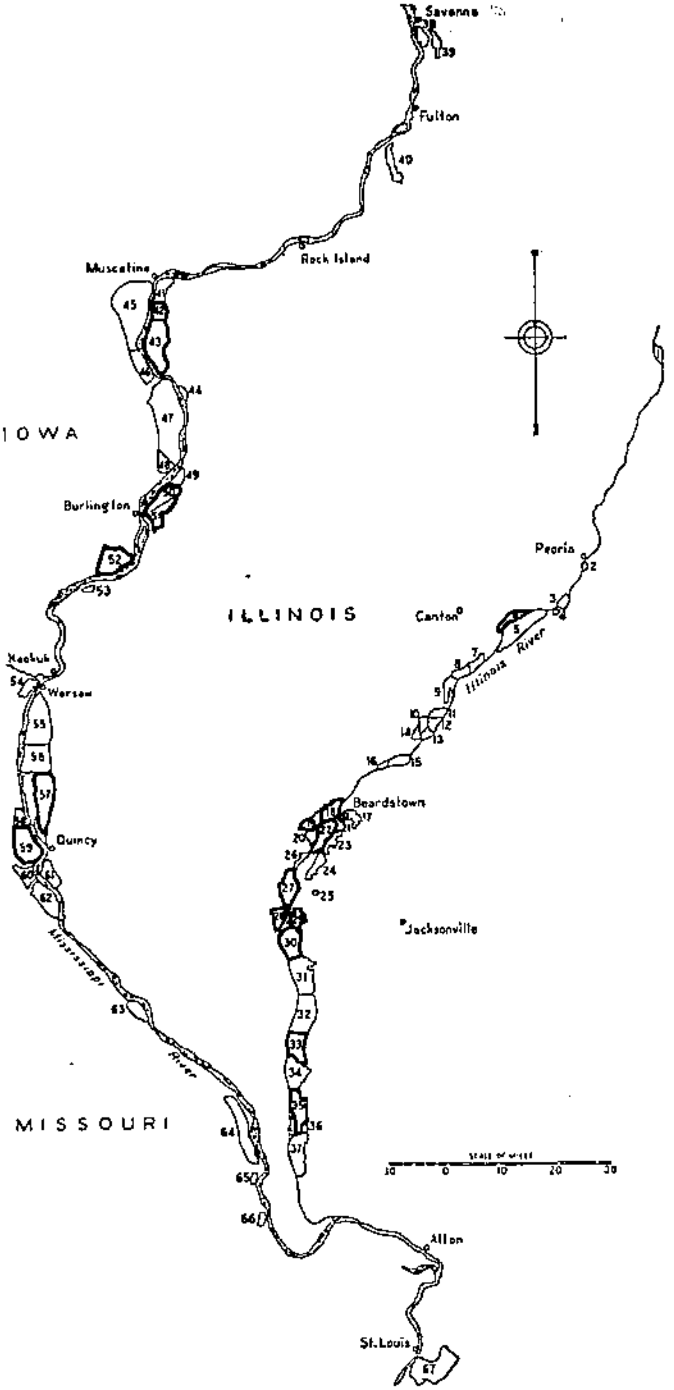

FIGURE 1.—Pumping drainage districts on the Illinois River below Peoria and on The Mississippi River between Savanna, Ill., and St. Louis, Mo., 1930. (Districts identified by key numbers used in table 9, p. 77.)

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able expense in order to reduce the costs of pumping and of maintaining the drainage ditches. Where the lands lie well above low-water stages of the outlet, gravity drainage sometimes can be obtained. Where gravity drainage could be obtained only for occasional short intervals it may be more economical to pump all the drainage than to install a sluiceway or similar structure.

Location of Pumping Plants

The location of the pumping plant is determined largely by the topography of the area and the ditch system. It is desirable to install all the pumping equipment at one location if possible to secure lowest construction and maintenance costs. Often, however, the costs and effectiveness of installing two or more pumping plants should be considered in order to accomplish equivalent results. The location of plants should be based on the following considerations: Effect on construction of drains; practicability of small automatic electric plants to drain isolated areas: proximity of roads, railroads, waterways for transportation of fuel and labor; living quarters for labor; proximity of electric power lines; construction problems and foundation conditions.

When an old pumping plant is being replaced it is usually impracticable to change the location of the plant. When other conditions do not control, the best location may be determined by pumping lifts, suction bay storage, and foundation conditions.

Foundation Conditions—Soil Borings

Good foundation soils are not only desirable to minimize construction costs but as a safety factor during flood conditions. Some plants have been destroyed by levee breaks at the pumping plant during floods.

The existence of quicksand or other unstable material at the site may greatly increase construction costs and affect design. Adequate information on foundation conditions should be secured by means of soil borings.

Storage at Suction Bay

It is desirable to provide large storage near the pumping plant. Storage in the suction bay is advantageous for several reasons. Where a substantial amount of storage is available, reduction in size of the pumping plant may be permissible. When the runoff exceeds the plant capacity the water levels in the ditches will not rise so rapidly or so high above optimum stage. The fluctuations of the suction bay are reduced in amount, thereby providing a more constant lift. The water in the bay need not be pumped so low in order to provide convenient intervals between operation of the pumps. Pumping the water low in the suction bay increases the static lift and tends to suck air into the pumps, which decreases their efficiency. When a pumping district has a large amount of storage available a small pump is not as essential for pumping during periods of small runoff. A large storage makes it possible to reduce night operations and consequently may result in decreased labor costs. In cases where advantageous rates for electric power are available through "off peak" pumping a large storage is a distinct advantage. It permits storage of water during day hours and enables a reduction in penalties for peak-hour pumping.

In districts where the topography permits, it is often desirable to leave low sloughs, lakes, or ponds unreclaimed in order to provide storage.

The open-ditch system is nearly always designed with a higher capacity than the pumping plant (p. 35). The excess capacity of the ditch system provides valuable storage during periods of heavy rainfall and enables water to flow more rapidly to the pumping plant.

Design of Open Ditches in Upper Mississippi Valley

The design of open ditches is an essential feature in adequate planning for drainage pumping systems. Ditch systems must conduct the water from the fields to the pumping plant and must also provide for storage of excess water during heavy floods. The runoff capacity provided by open ditches is determined on the basis of experience in the area in which the plants are located. Runoff curves for adequate drainage, often referred to as "drainage coefficient," are given in figure 2 for States in the upper Mississippi Valley. These curves were based on field measurements and observations (*12*). They have been used successfully by the Soil Conservation Service in designing ditches for more than a decade and have provided a basis for adequate drainage systems. Generally the "C" curve has been used for most drainage systems. These curves are applicable in the formula for estimating maximum runoff, given on page 35. However, in areas other than the upper Mississippi Valley appropriately similar curves for the

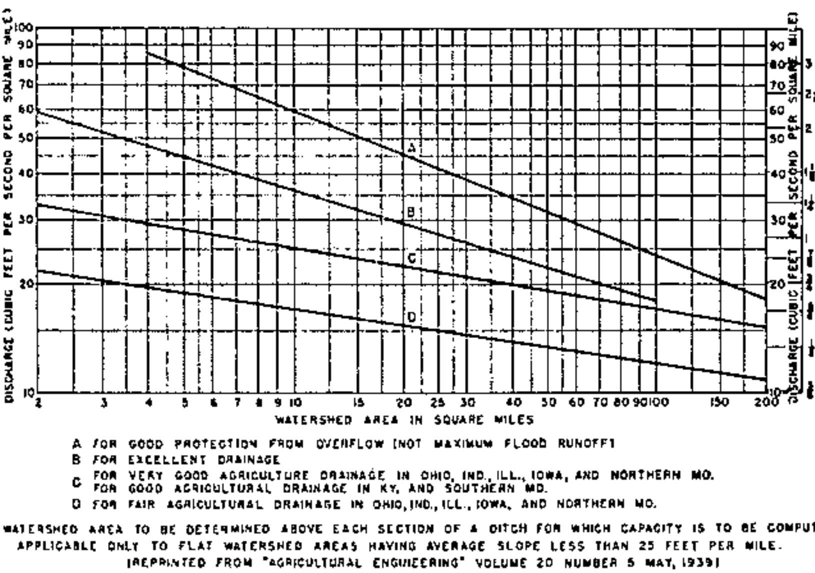

FIGURE 2.—Curves showing "runoff to be allowed" or the "drainage coefficients" to use in upper Mississippi Valley.

locality should be used. Information on curves similar to these can usually be secured through soil conservation districts.

DETERMINATION OF STATIC LIFTS

All available data relating to the stages of the discharge and suction bays should be studied to determine the maximum, minimum, and average static lifts of the pumps. The pump manufacturer needs these data in order to supply equipment that will operate efficiently at all 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 was only 5 feet. When new impellers were installed in the pumps and the speed decreased, the saving in electric power obtained was estimated to average $2,000 per year.

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 at 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. This stage will ordinarily be 4 to 7 feet lower in elevation than the elevation of important areas of land requiring drainage.

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 is likely to be inadequate during floods. The maximum lift may usually be determined from maximum recorded flood stages at nearby gages.

The annual publication of the United States Weather Bureau "Daily River Stages," gives the maximum, minimum, and daily gage heights for the river gages of the principal rivers of the United States. Information relating to maximum and minimum flood heights for stations along the Illinois and upper Mississippi Rivers are given in table 1. The maximum stage of the discharge bay along a river covered by records may readily be determined by plotting a profile of flood stages along the river or by interpolation of flood elevations between gaging stations.

Stream gages are also maintained by the United States Geological Survey, the Corps of Engineers, United States Army, and by other

TABLE 1.—*High-water stages and other data relating to river gaging stations*[1]

Station	Length of record	Elevation of gage zero above mean sea level	Distance of gage above mouth of river	Drainage area above gage	Flood stage	Extreme stages from gage readings			
						Highest	Year	Lowest	Year
Illinois River:	*Years*	*Feet*	*Miles*	*Square miles*	*Feet*	*Feet*		*Feet*	
Morris, Ill.	41	478.50	263.3	7,590	13	21.6	1943	0.0	1900
Peoria, Ill.	74	428.30	162.3	13,672	18	28.0	1943	5.3	1000
Havana, Ill.	52	424.28	119.9	17,691	14	27.3	1943	4.2	1901
Beardstown, Ill.	66	419.80	88.6	23,587	14	29.7	1943	0.7	1901
Mississippi River:									
Dubuque, Iowa	77	584.94	1,543.9	81,978	18	21.7	1880	−2.8	1933
Davenport, Iowa	86	542.00	1,447.0	88,403	15	19.4	1892	−1.8	1033
Muscatine, Iowa	68	530.97	1,419.2	90,501	15	19.5	1922	−1.8	1890
Keokuk, Iowa	78	477.43	1,328.2	118,993	12	20.85	1944	−4.3	1934
Quincy, Ill.	48	458.22	1,280.9	135,132	14	23.0	1944	−3.4	1938
Hannibal, Mo.	74	449.07	1,273.9	137,303	13	22.6	1944	−2.7	1933
Grafton, Ill.	60	403.72	1,182.0	171,492	18	29.0	1943	−1.1	1937
St. Louis, Mo.	88	379.80	1,144.0	694,260	30	39.1	1944	−6.1	1940

[1] From daily river stages, U. S. Weather Bureau, for year 1945.

Federal and State agencies. The stages of the discharge bay for a plant located on a tributary channel may be higher than for one located on the main river. Static lifts for 15 districts studied are shown in table 6, page 23.

Figures 3 and 4 show the hydrographs for selected stations on the Illinois River for 1927 and 1943, and on the Mississippi River for 1929 and 1943. 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 duration of flood. Increasing the maximum design lift unnecessarily increases the power required to drive the pumps and due to pump characteristics tends to decrease the plant efficiency at lower lifts. The maximum lift for plant design sometimes may be decreased to the extent of the rise of the suction bay which necessarily occurs during floods. It is usually impracticable to prevent flooding of some low-lying areas by the worst floods.

Study of operating conditions during flood periods (pp. 30–34) shows the importance of designing a plant to pump its full capacity at the maximum lift. Full-capacity pumping, as the figures show, 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

For design purposes the minimum lift should be assumed to be the difference between minimum stage of discharge bay and optimum stage of suction bay, based on mean monthly figures and not the minimum possible lift. 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.

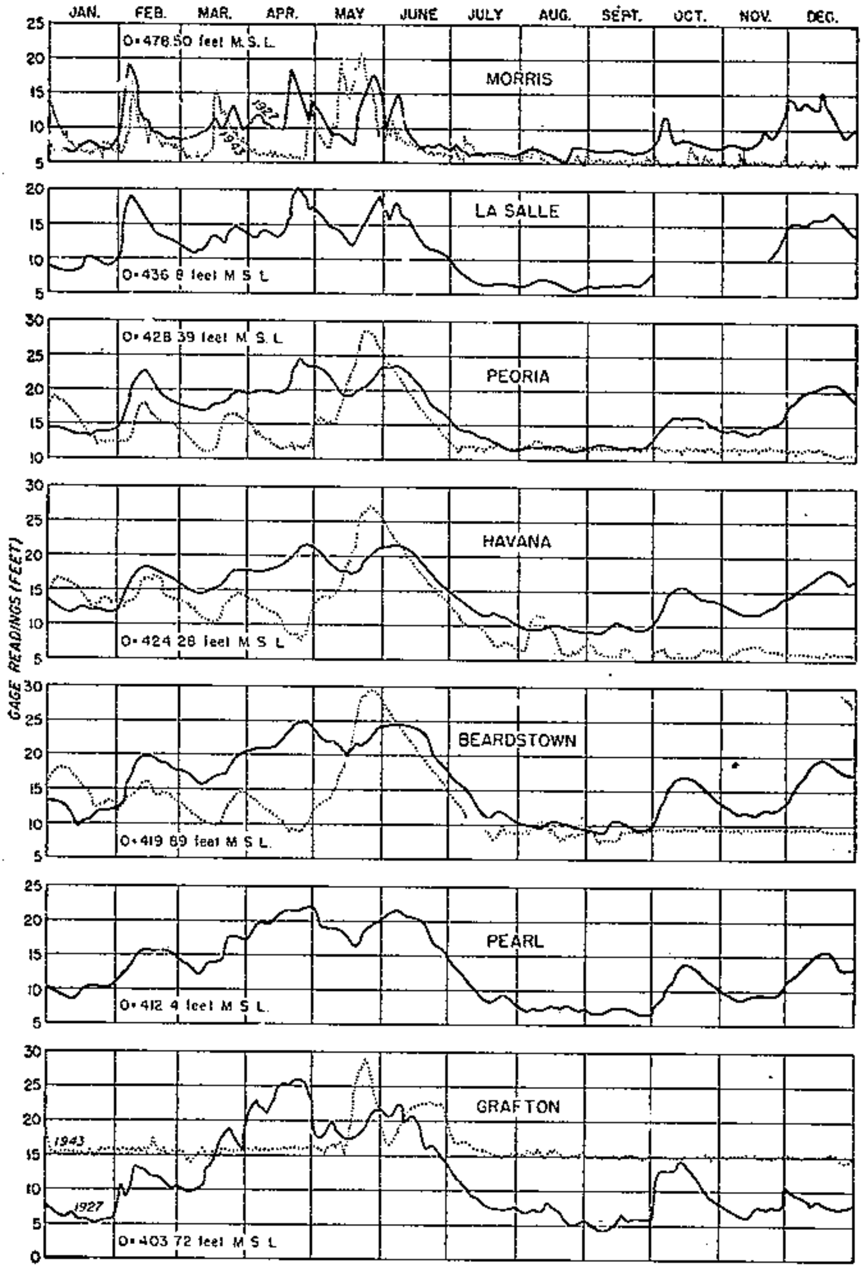

FIGURE 3.—Daily stages of Illinois River, 1927 and 1943.

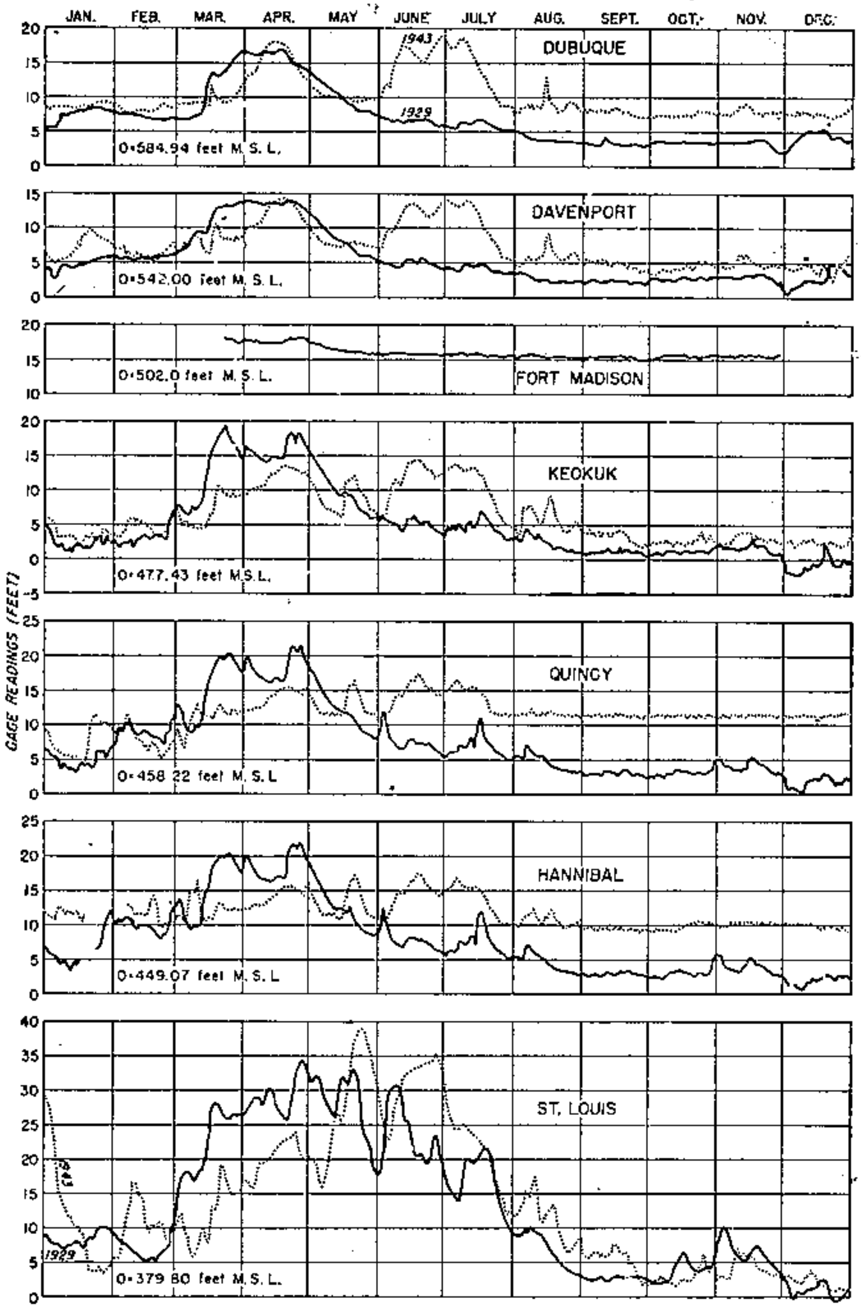

FIGURE 4.—Daily stages of Mississippi River between Dubuque and St. Louis, 1929 and 1943.

TABLE 2.—*Minimum mean monthly stage*[1] *recorded during period 1941–45*

ILLINOIS RIVER

Station	January	February	March	April	May	June	July	August	September	October	November	December
Morris, Ill	5.0	5.3	5.9	6.3	5.9	5.9	5.3	5.1	5.0	5.1	5.1	4.9
Peoria, Ill	10.7	11.0	11.6	11.6	11.4	11.6	11.6	11.5	11.6	11.5	11.7	11.2
Havana, Ill	5.9	5.9	7.8	9.3	8.0	8.4	6.4	5.7	5.8	6.0	6.2	6.0
Beardstown, Ill	8.7	8.9	9.1	9.6	9.4	9.5	9.3	9.1	8.7	9.1	9.4	9.4
MISSISSIPPI RIVER												
Dubuque, Iowa	6.9	6.4	8.1	10.0	10.0	10.3	7.5	6.6	7.6	7.4	7.5	7.4
Davenport, Iowa	3.2	3.4	5.6	7.3	7.2	7.8	4.2	3.1	4.6	4.1	4.2	3.7
Muscatine, Iowa	5.2	6.0	7.1	8.6	8.8	9.3	5.8	4.8	5.9	5.6	5.8	5.7
Keokuk, Iowa	.9	1.2	4.1	6.6	6.0	7.3	3.1	2.2	2.7	2.5	2.6	2.0
Quincy, Ill	3.0	3.8	10.9	11.6	11.1	11.6	11.2	11.2	11.3	11.3	11.3	10.3
Hannibal, Mo	3.9	4.2	10.2	11.5	11.3	11.6	10.2	10.1	9.5	9.7	10.1	9.8
Grafton, Ill	14.7	14.6	15.0	15.4	15.0	15.4	15.0	14.9	15.0	14.9	14.9	14.8
St. Louis, Mo	.5	2.5	4.6	15.6	9.7	15.6	6.8	—.1	5.4	3.0	3.3	1.3

[1] Weather Bureau gage readings. See table 3 for elevations of zeros of gages.

In many instances the minimum elevation of the discharge bay is above the minimum river stage. This is the case when the bay is a considerable distance from the river or where obstructions intervene. The advantages of enlarging and deepening such an outlet channel to secure a lower average lift should be considered. Average low-water stages such as those given in table 2 may be used for estimating minimum lifts. For instance, as 9.1 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 River occurring in December, January, and February should be disregarded because they occur at a season when very little pumping is necessary. Many navigation dams have been constructed along the Mississippi and Illinois Rivers. If the pumping plant discharges into a pool formed by a dam the minimum lift for design purposes would ordinarily be based on pool stages. Similarly, back water from dams may affect stages of tributary streams. The Corps of Engineers of the United States Army is responsible for operation of navigation dams in the upper Mississippi Valley and would be the source of information relating to pool stages.

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 runoff pumped in the respective months.

The average stages, annual and monthly, of the Mississippi River and the Illinois River at the principal cities in the region where drainage pumping is common are given in table 3. 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 watershed area lies farther north. From this table the average elevations of the discharge bay can be estimated along these rivers. Consideration should be given to any other available data relating to water elevations at the particular location of the proposed pumping plant.

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 Runoff," page 25. 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.

TABLE 3.—*Average of mean monthly stages*[1] *recorded during period 1941–45*

Station	January	February	March	April	May	June	July	August	September	October	November	December	Annual
ILLINOIS RIVER													
Morris, Ill	5.9	7.0	7.6	7.9	8.7	6.6	5.7	5.6	5.5	6.0	6.2	5.8	6.5
Peoria, Ill	12.0	13.1	13.3	13.9	15.5	13.4	11.8	[illegible]	11.9	12.3	12.3	11.5	12.7
Havana, Ill	8.2	9.9	10.0	12.7	13.8	12.1	7.9	6.8	9.8	8.1	8.7	8.1	9.5
Beardstown, Ill	10.2	11.8	11.8	13.7	15.1	13.7	10.3	9.5	9.3	10.5	10.9	10.1	11.4
MISSISSIPPI RIVER													
Dubuque, Iowa	7.5	7.2	9.5	12.8	11.3	13.9	10.5	8.1	9.2	8.8	8.6	7.7	9.6
Davenport, Iowa	4.5	4.4	7.0	9.9	8.5	10.7	7.5	5.1	6.0	5.8	5.7	4.8	6.6
Muscatine, Iowa	6.1	6.6	9.3	11.6	10.2	12.5	9.2	6.6	7.4	7.3	7.2	6.2	8.4
Keokuk, Iowa	2.0	2.8	6.6	10.1	9.2	11.4	7.1	4.2	4.4	4.7	4.5	3.1	5.8
Quincy, Ill	8.3	8.4	11.7	13.4	12.9	14.5	12.2	11.4	11.4	11.5	11.6	11.0	11.5
Hannibal, Mo	8.6	9.4	11.7	13.7	13.1	14.7	12.0	10.5	10.5	10.8	10.8	10.2	11.3
Grafton, Ill	15.1	15.1	15.6	17.3	17.8	18.4	15.6	15.1	15.2	15.3	15.4	15.1	15.9
St. Louis, Mo	4.2	7.4	13.6	21.6	22.2	25.3	16.7	7.5	8.4	9.7	8.5	4.7	12.5

[1] Weather Bureau gage readings. See table 1 for elevations of zeros of gages.

The average lift may also be determined by comparison of similar districts. Operation data for plants during the study (1925–30) are shown in table 10 and include the average lift weighted according to the amount of pumping.

DETERMINATION OF RUNOFF TO BE PUMPED

The runoff 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 runoff depends on the amount, intensity, and distribution of rainfall and other precipitation, storage, 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 often an important factor in pumping requirements and its amount depends upon the local conditions. 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 runoff to be pumped can best be estimated from comparisons with similar districts

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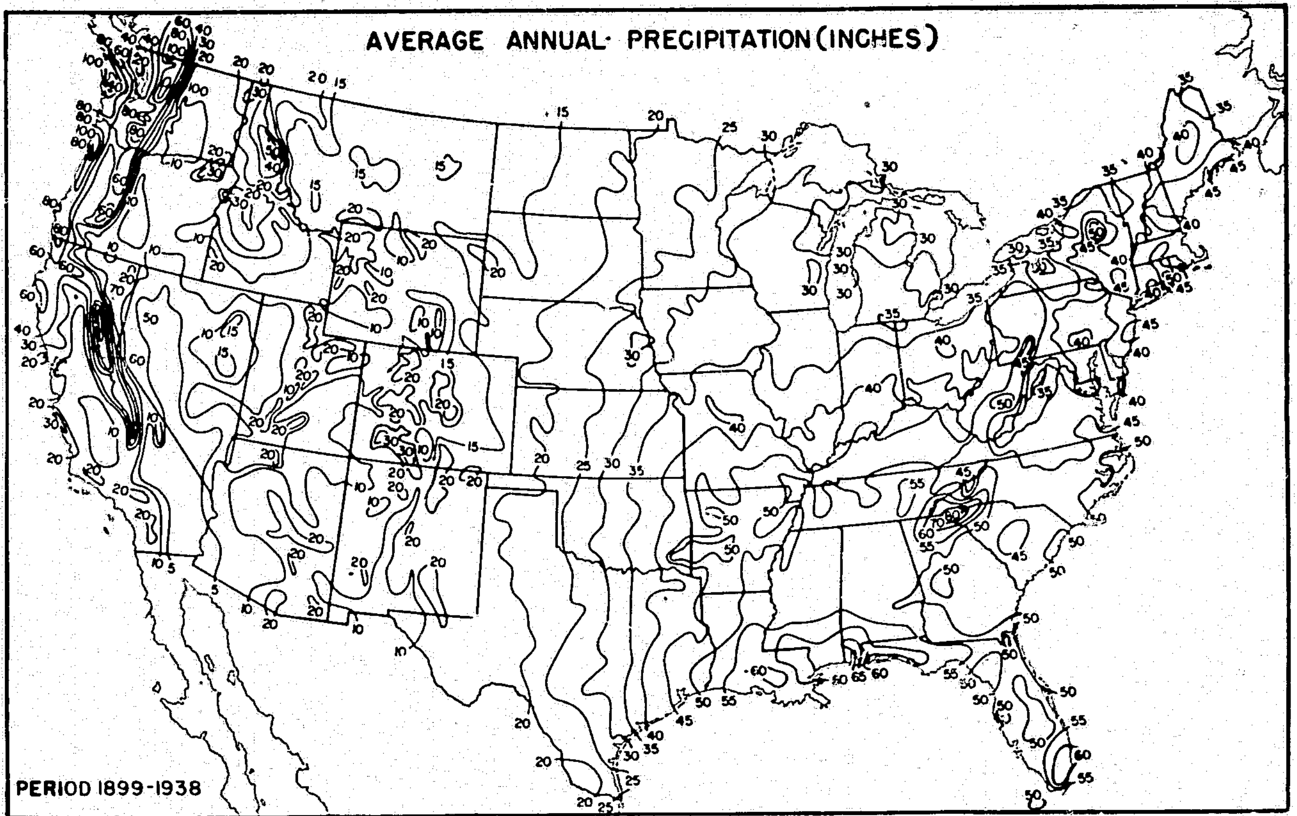

FIGURE 5.—Average annual precipitation (inches) for the period 1899–1938.

from which the runoff has been measured. A large amount of such data is available for the districts in the upper Mississippi Valley (*11*, *15*) and will be discussed in the following pages. Operation data for the plants studied are summarized in table 10. Considerable data are available also for districts in Louisiana (*2*, *8*).

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

Rainfall Charts

Annual and daily rainfall data are valuable in the design of drainage pumping plants. Figure 5 from the 1941 Yearbook of Agriculture (*13*) shows the average annual precipitation. The average precipitation in the upper Mississippi Valley in the approximate location of the drainage pumping plants, is about 35 inches per year. The annual rainfall is one of the factors in estimating annual pumping required.

The 24-hour rainfall to be expected once in 5 years as determined by Yarnell (*16*) is shown in figures 6 and 7. The daily rainfall is one of the factors to be considered in checking the maximum daily pumping requirements. The differences in daily rainfall to be expected is the chief reason that pumping plants along the Gulf Coast require a much larger capacity than those in the upper Mississippi Valley.

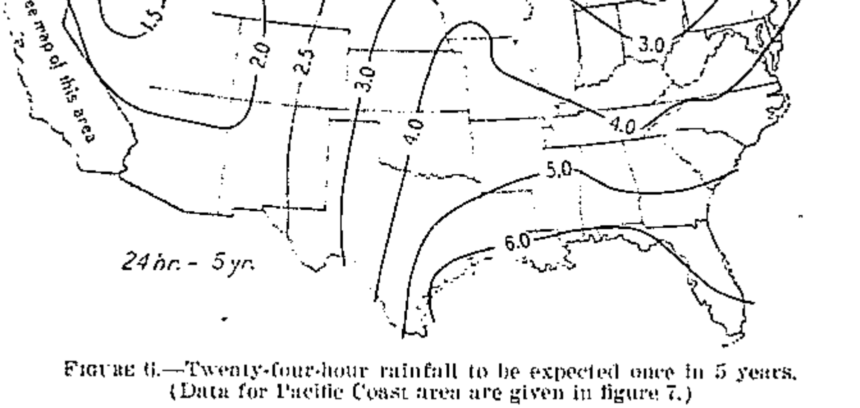

FIGURE 6.—Twenty-four-hour rainfall to be expected once in 5 years. (Data for Pacific Coast area are given in figure 7.)

Evaporation and Transpiration

The evaporation and transpiration often amount to a large proportion of the rainfall. In pumping districts the land is flat or undulating and the water table is often near the surface. In some pumping districts low areas are occasionally flooded. Considerable areas of open water may occur in sloughs, marshes, and borrow pits. Under these conditions in pumped areas the evaporation and transpiration losses are so great that often the pumping is only a small portion of the annual rainfall. Often this is the case even though such an area is enclosed by levees and the seepage and ground-water flow is toward the pumped area rather than away from the area.

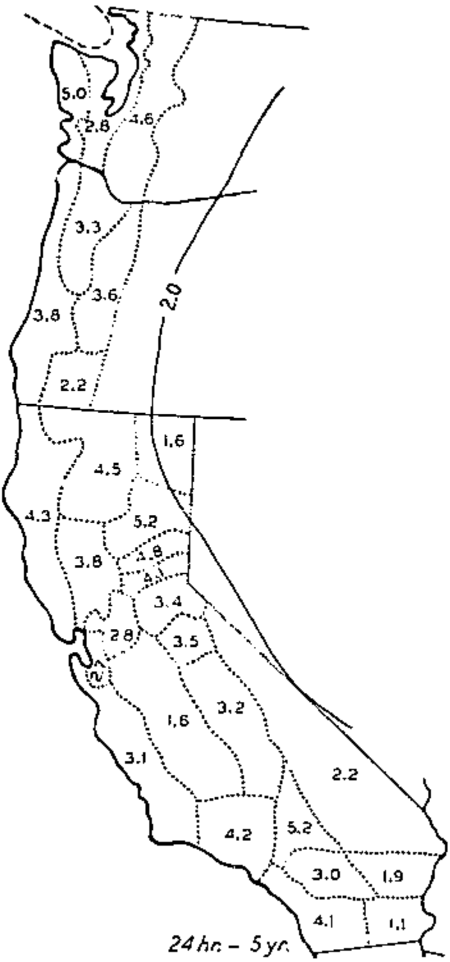

FIGURE 7.—Twenty-four-hour rainfalls in the Pacific Coast district to be expected once in 5 years.

Not much can be done in a practical way to affect the evaporation and transpiration from pumping districts. However, a knowledge of the amounts involved aids in understanding the amounts of pumping required for various periods of time. Table 4 shows the average monthly evaporation from open-water surface measured from evaporation pans operated by the United States Weather Bureau for several stations.

Evaporation and transpiration depend on the crops grown and surface treatment. The Florida Everglades experiment station made a study of amounts of water evaporated and transpired by certain plants growing in tanks of peat soil. Table 5, quoted from the report of L. A. Jones and others (*4*), relating to the Florida Everglades, shows the results of these studies. The amounts of transpiration and evaporation with living plants varied but were a large proportion of the evaporation from a free-water surface. The evaporation and transpiration from the tank on which a mulch of cane trash was maintained was substantially less than from other tanks.

TABLE 4.—*Average monthly evaporation from open-water surface for period 1943–47* [1]

Station	January	February	March	April	May	June	July	August	September	October	November	December	Total for period
Belle Glade, Fla	3.40	4.23	5.82	6.79	6.91	6.07	6.19	6.27	5.48	4.70	3.63	3.08	62.50
Moore Haven, Fla	2.93	3.74	5.16	6.03	6.20	5.13	4.68	4.58	4.50	4.28	3.50	2.89	53.62
Ames, Iowa	-----	-----	-----	4.54	5.40	6.89	8.13	7.03	5.07	3.63	-----	-----	49.70
Iowa City, Iowa	-----	-----	-----	4.03	5.15	6.32	7.72	6.32	4.94	3.13	-----	-----	38.27
Hays, Kans	-----	-----	-----	7.28	8.52	12.14	14.10	13.87	10.88	6.65	-----	-----	73.52
St. Louis, Mo	-----	-----	[2]2.98	3.94	4.22	5.87	6.70	5.62	4.07	2.70	1.40	[2]1.14	-----
Columbia, Mo	-----	-----	-----	[3]4.50	[3]5.36	[3]6.60	[3]8.42	[3]7.01	[3]5.74	[3]3.52	-----	-----	-----
Lincoln, Nebr	-----	-----	-----	4.67	5.59	7.67	8.69	8.05	6.25	3.71	-----	-----	44.53
Ithaca, N. Y	-----	-----	-----	-----	4.07	4.01	5.82	4.94	4.02	2.36	-----	-----	29.12
Dickinson, N. Dak	-----	-----	-----	4.13	4.92	5.11	7.24	7.14	4.69	-----	-----	-----	33.23
Ohio State University, Ohio	-----	-----	-----	[3]3.47	4.18	4.92	5.41	4.78	3.41	2.33	-----	-----	-----
Medford, Oreg	.47	1.20	2.26	3.81	5.79	6.29	8.51	7.06	4.96	1.99	.68	.48	43.42
Beaumont, Tex	2.26	2.42	3.31	[4]4.32	4.95	5.64	5.71	5.28	4.47	3.80	2.64	2.18	-----

[1] From Weather Bureau Stations. See Climatological data.
[2] For 1 year
[3] For 3 years.
[4] For 4 years.

TABLE 5.—*Average evaporation and transpiration from tanks of soil and vegetation and from open pan of water at Everglades Experiment Station, Belle Glade, Fla., 1934–43*

Month	Sugarcane (10 years)	Sawgrass (7 years)	Bare soil (4 years)	Cane trash (6 years)	Standard open pan (20 years)
	Inches	*Inches*	*Inches*	*Inches*	*Inches*
January	1.24	3.09	1.56	0.46	3.53
February	1.28	3.40	2.28	.57	4.13
March	1.74	4.69	3.04	.69	5.74
April	2.91	6.19	4.06	.67	6.59
May	3.89	8.16	3.94	.67	7.39
June	7.18	7.56	4.30	1.75	6.30
July	6.98	8.18	4.71	1.55	6.71
August	6.78	7.41	4.71	1.37	6.29
September	5.47	6.22	4.35	1.27	5.48
October	6.08	5.95	3.08	.02	5.29
November	4.39	3.76	2.00	.58	3.90
December	3.08	3.09	1.70	.72	3.20
Year	49.02	68.32	40.02	11.22	64.43

RUNOFF FROM WATERSHEDS

The Soil Conservation Service has published data relating to runoff from smaller watersheds. Data applicable to the section of the country in which the plant is to be installed may often be obtained from this Service on request. Such data are helpful in estimating annual amounts of pumping. Allowances need to be made for differences in soils, topography, and seepage.

The Geological Survey of the Department of the Interior was maintaining 5,810 gaging stations in July 1946, including those in Hawaii. Measurements of the flow of streams and contents of the lakes and reservoirs had been made by the Geological Survey and cooperating organizations at about 10,000 gaging stations as of that date. Data for many of the streams cover runoff from major river basins and would not be applicable to runoff from pumping districts. However, surface-water-supply papers are published annually by the Geological Survey. They form a valuable source of information for determining runoff where comparable watersheds can be selected.

Annual Runoff Pumped

The best way of determining the average annual runoff pumped from districts where no records are available is by comparison with districts that have similar conditions of topography, soil, drainage, and static lift. The estimated average annual runoff for certain districts in the upper Mississippi Valley, are shown in table 6, together with the static lifts and the amounts that backwaters from power and navigation dams have increased the minimum lifts.

These data are estimates based on daily records of operation before 1931. Since then construction of navigation dams has changed operating conditions of some pumping districts materially. Where navigation dams raise the average stage of the discharge bay the static lift and the amount of pumping will be larger because of changed conditions. The extent to which 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 in table 6 according to the amount that backwater from dams increased the runoff to be pumped. Districts having an average annual runoff at 20 inches or greater, obviously, were materially affected by backwater. This represents an increase of at least 6 to 8 inches in annual runoff due to seepage. The Crane Creek district has been included in the same group because its runoff has also been increased greatly owing to the dam. The elevation of 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 land in 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 runoff to be pumped by more than 2 or 3 inches a year.

The estimated average annual runoff of districts of group 1, the districts that pump a considerable amount of seepage from the Illinois River, ranged from 16 to 35 inches in 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 runoff pumped is 20 inches per year. The lifts are not affected by backwater because the pumping plant is below the dam.

The average runoff pumped from the Illinois River districts of group 2 (table 6) varied only from 14 to 16 inches. Apparently there was little seepage into these districts and the runoff 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 River prior to the time that the dams were put in, were, in some cases, below the normal stages

TABLE 6.—*Estimated annual runoff to be pumped, static lifts, and backwater effects for typical districts*

Key No.[1]	District	Average runoff 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	0	6
6	Banner Special	16	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
51	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	6	13	1	0
20	Mauvaisterre	5	4	12	1	6

[1] For identification in figure 1 and table 9.
[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 (table 10).
[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.

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 runoff from these districts varies from 12 to 15 inches, which is considered typical for districts in this territory that have to pump all runoff but into which there is little seepage.

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

The data indicate that the runoff from districts not affected greatly by backwater from dams, but which pumped all runoff, 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 largly the result of differences in porosity of subsoils and the extent to which the drainage ditches cut into the pervious strata.

The ratio of pumped runoff 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 runoff to rainfall. Figure 8 shows how this ratio of annual pumped runoff to annual 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 runoff to rainfall although its average lift ranged only from 5 to 8 feet. It is 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 equally 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

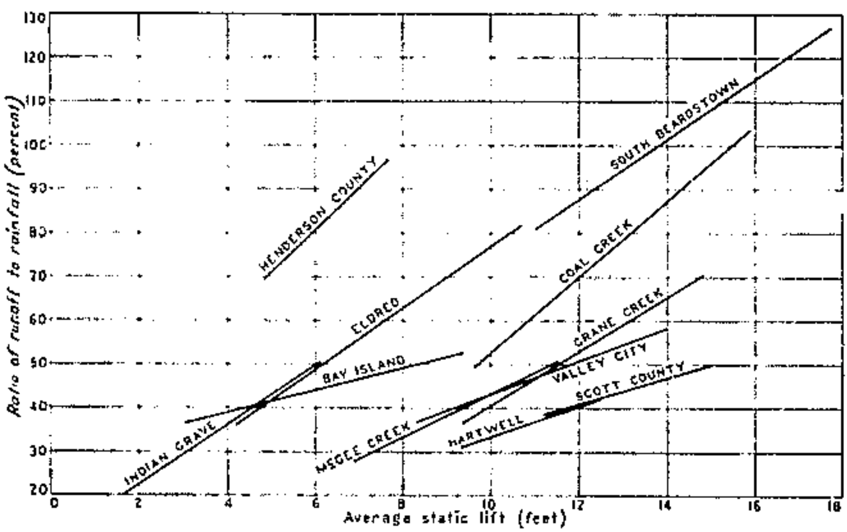

FIGURE 8.—Observed relations between the ratio of pumped runoff to rainfall and the static lift for certain drainage districts.

Creek district may be classed as seeped, but the remainder has tight subsoil. The other five districts represented in figure 8 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 figure 8, 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 the use of districts that drain low lake beds. The average runoff 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 runoff pumped by the South Beardstown plant in the 6 years, 1925–30, was 7 percent greater than the rainfall, and the average lift was 15 feet. The estimated annual runoff is 35 inches, at an average lift of 14 feet (table 6). The amount of gravity drainage from the Eldred district is so small (*11*) that it does not materially affect the accuracy of the curve shown in figure 8.

The runoff 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 runoff during months when gravity drainage is expected.

Distribution of Runoff Pumped

The principal factors that cause seasonal variations in amount of runoff 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 runoff pumped by a number of typical districts and the average monthly rainfall and static lift is shown in figures 9, 10, and 11. The runoff 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 runoff 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 runoff was comparatively small from the other districts shown in figures 9, 10, and 11.

Runoff distribution for a proposed plant will be similar to that of nearby plants on the same river; but 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 runoff 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 by using the pump manufacturer's rating curves and making suitable allowance for reduction in pump efficiency on account of age and condition of equipment. Power consumption in indicated horsepower-hours per acre-foot by a number of plants is given in table 10. From the power consumption and average lift

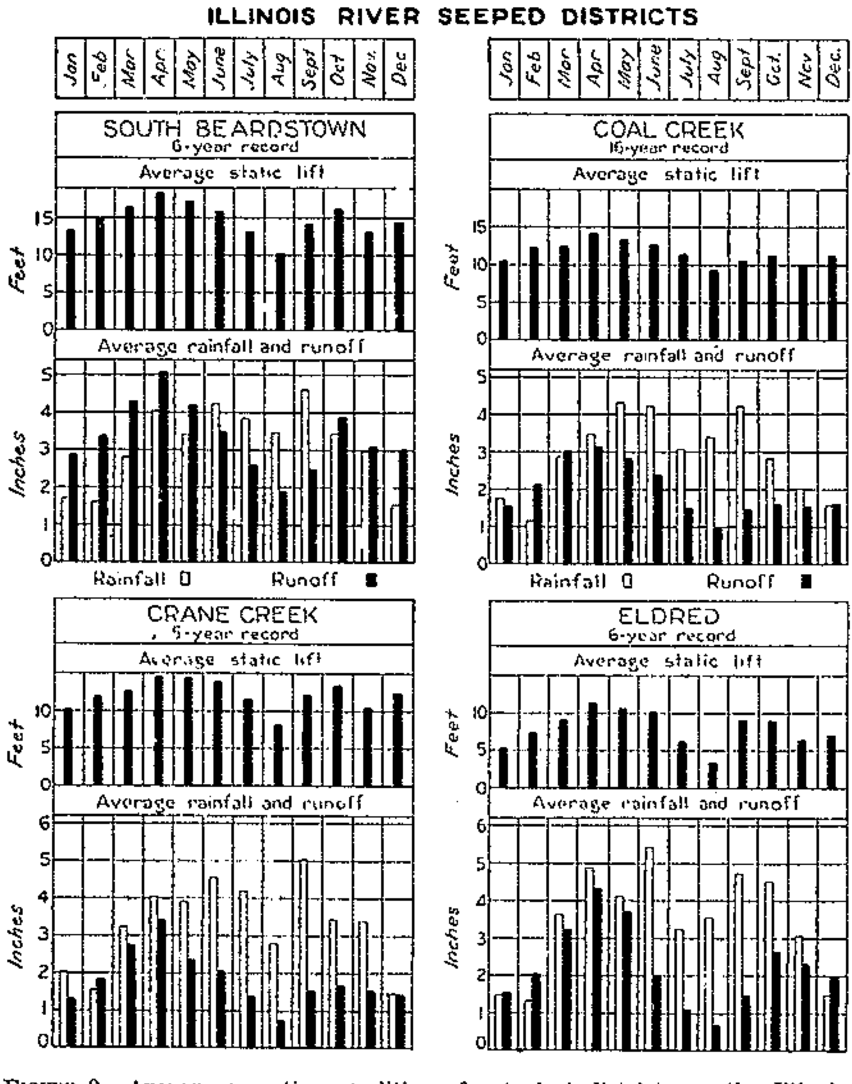

FIGURE 9.—Average operating conditions for typical districts on the Illinois River pumping much seepage.

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 Runoff Pumped

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-runoff depth per 24 hours. At least half

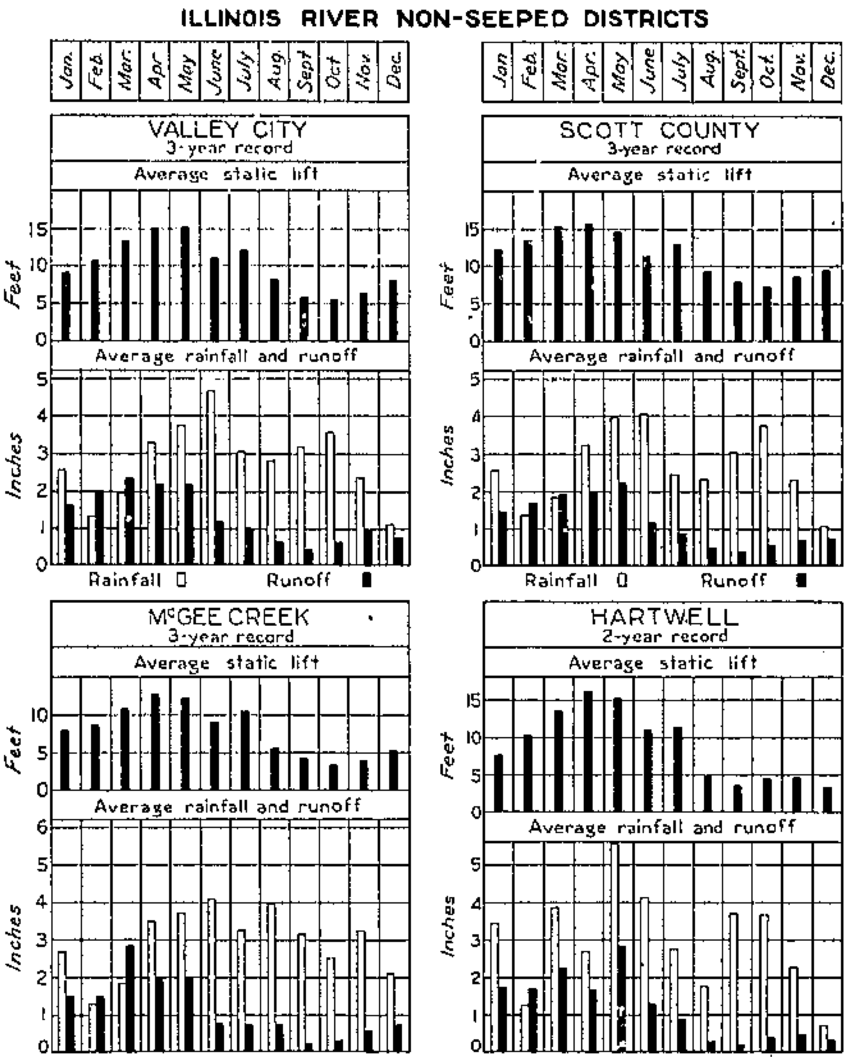

Figure 10.—Average operating conditions for typical districts on the Illinois River pumping little seepage.

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 additional capacity costs more than the damage to be prevented. Proper determination of the maximum rate of runoff to be pumped is the most important problem in designing a plant.

A plant must be large enough to provide adequate drainage, yet unnecessary capacity may be uneconomical because the first cost of a plant depends on its size. Also the cost of electric power usually depends on the rated capacity of electric motors installed.

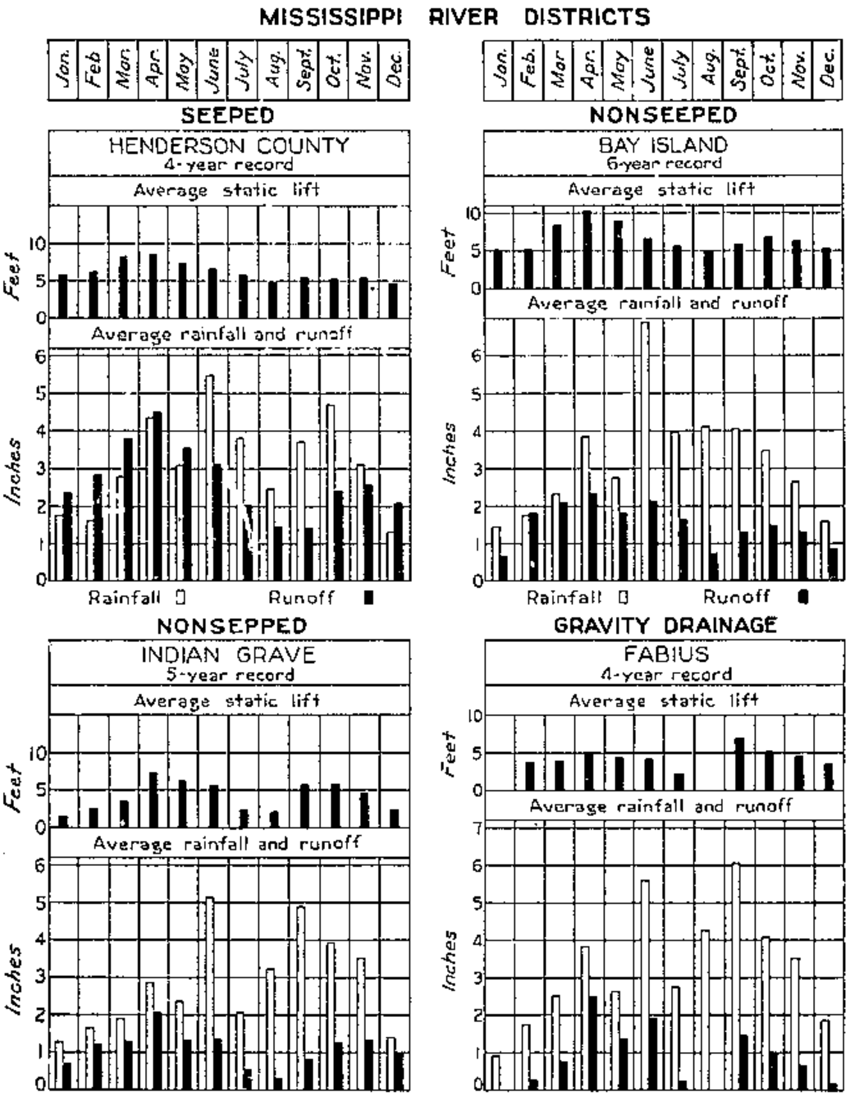

FIGURE 11.—Average operating conditions for typical pumping districts on the Mississippi River.

Table 7 gives the design factors originally used in designing the pumping plants covered in this investigation, the design factors recommended for those plants as a result of this study, and the actual plant capacities at the recommended maximum lifts, as determined from rating curves prepared from tests of the plants.

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 7. Various methods of comparing these data for calculating economical plant capacities were 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

TABLE 7.—*Design factors for typical pumping drainage districts*

District		Original design factors		Recommended design factors			Actual 24-hour capacity at maximum lift recommended
Key No.[1]	Name	Maximum 24-hour runoff	Total head on pumps	Average annual runoff [2]	Maximum 24-hour runoff [3]	Maximum static lift	
		Inches	*Feet*	*Inches*	*Inches*	*Feet*	*Inches*
22	South Beardstown	0.38	27	35	0.56	24	0.44
52	Green Bay	.36	26	32	.51	13	.48
51	Henderson County	.20	15	27	.46	13	.26
18	Coal Creek	.26	18	24	.48	22	.23
35	Eldred	.52	14	20	.44	18	.21
19	Crane Creek	.26	24	16	.43	21	.22
28	Valley City	.26	21	16	.42	20	.26
43	Bay Island	.23	15	15	.33	17	.12
6	Banner Special	.37	20	15	.42	23	.31
30	Scott County	.35	22	14	.38	22	.33
27	McGee Creek	.40	16	14	.37	20	.34
33	Hartwell	.55	21	14	.38	18	.44
57	Indian Grave	.27	17	12	.34	10	.21
50	Fabius	.34	16	7	.32	13	.27
29	Mauvaisterre	.15	17	5	.34	12	.16

[1] For identification in figures 1 and 12 and table 9.
[2] See table 6.
[3] Computed as $C=.33\ (G+.023\ r)$, see page 35.

was greatly simplified because the drainage areas were similar in topography, crops, weather, river fluctuations, and generally in size.

Based on the experiences of the drainage districts, the frequency of river floods, and the damage to crops resulting from lack of drainage it was concluded that a drainage pumping plant located in the upper Mississippi Valley should be able to prevent flooding of large areas for periods longer than 2 or 3 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 to the runoff to be pumped annually. Both the required capacity and annual runoff 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 12 the actual plant capacity at maximum lift has been platted against the average annual runoff to be pumped, r, for each

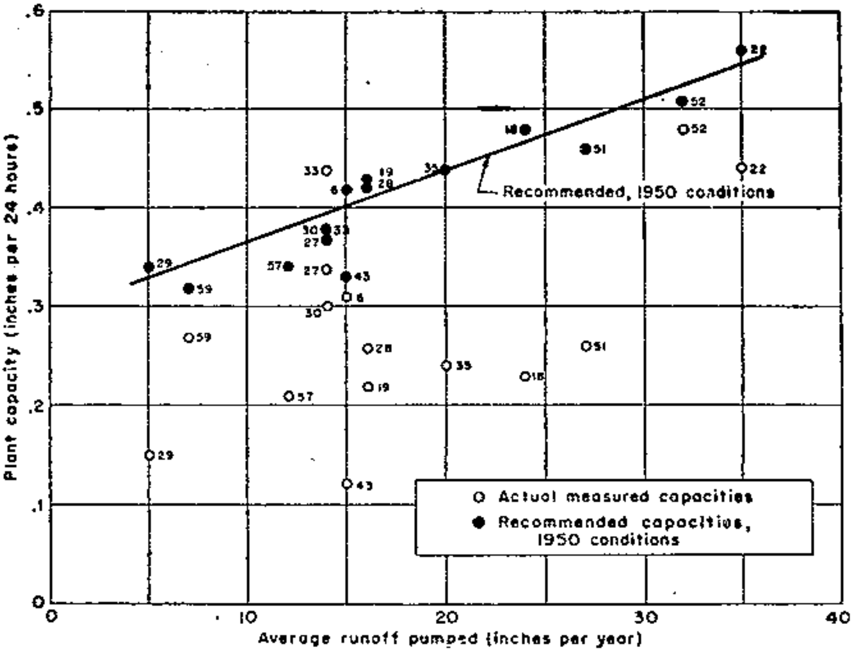

FIGURE 12.—Actual and recommended plant capacities at maximum lifts for drainage pumping plants in upper Mississippi Valley. (Numbers identify data in tables 7 and 9.)

plant. Those plant capacities were determined from rating curves prepared from tests of the pumps, and the amounts of runoff are those shown in table 7. In figures 13 to 20 are shown the daily records of rainfall, runoff pumped, and stages of the suction and discharge

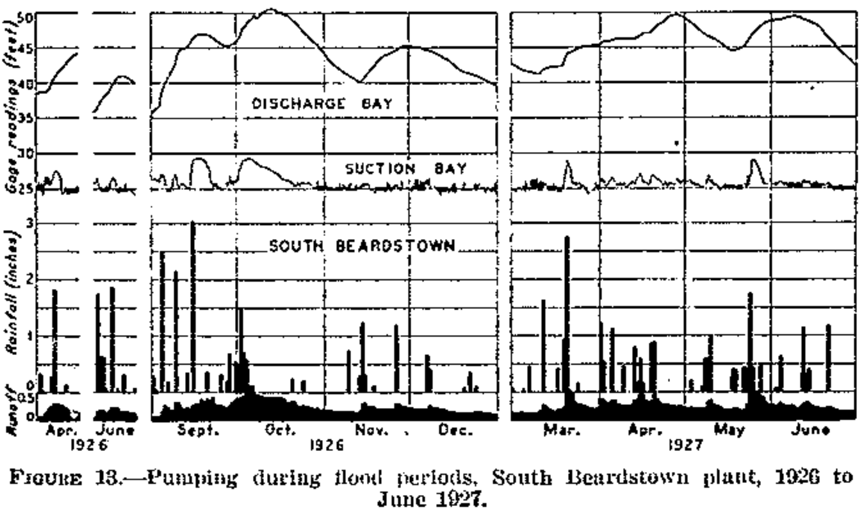

FIGURE 13.—Pumping during flood periods, South Beardstown plant, 1926 to June 1927.

bays during flood periods of 14 of the plants. Those records and the records of the Fabius plant indicate that, at maximum lift, increased capacity would generally be necessary for maintaining proper drainage in the respective districts. For 1950 conditions increased capacity is considered desirable (p. 35).

The plants represented by the statistics given in figures 13 to 15 lowered the suction bays nearly to the optimum stages within several days even during extreme floods. With seriously inadequate plants the suction bay stages remained 4 to 6 feet above the optimum for

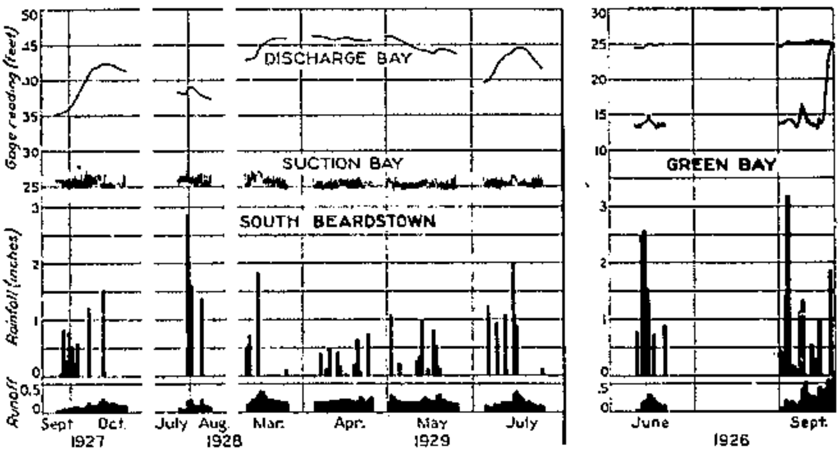

FIGURE 14.—Pumping during flood periods, South Beardstown plant, September 1927 to 1929, and Green Bay plant.

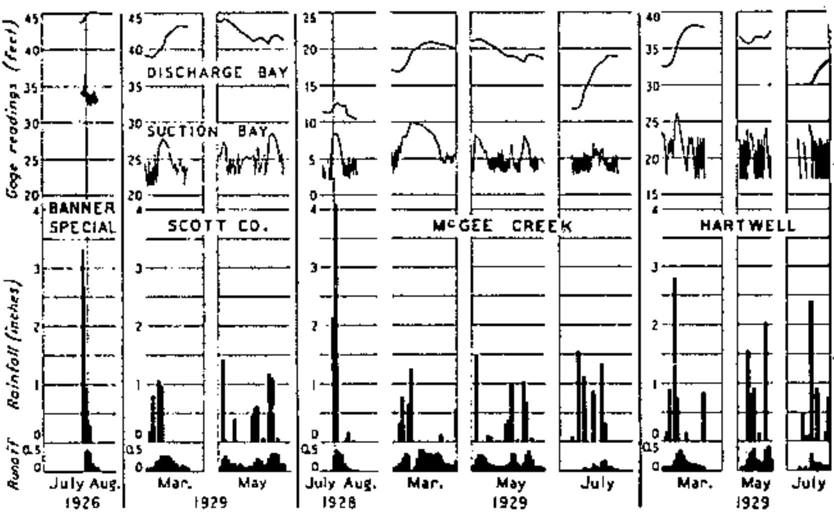

FIGURE 15.—Pumping during flood periods by Banner Special, Scott County, McGee Creek, and Hartwell plants.

periods of 2 weeks or longer, although pumping at full capacity was continuous. Flooding of Green Bay district in September 1926 (fig. 14) was due to a break in the levee, and flooding of McGee Creek district in March 1929 (fig. 15) was caused by lack of fuel for 3 days. The height and rapidity of the rises in the suction bay of Valley City plant (fig. 18) when flood conditions were not particularly severe, indicate that the capacity of that plant was inadequate.

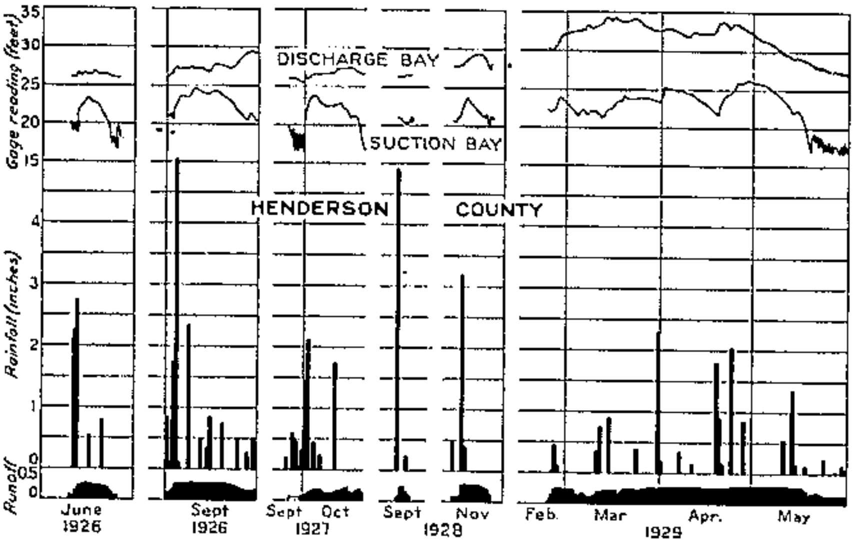

FIGURE 16.—Pumping during flood periods by plant having inadequate capacity. Henderson County plant.

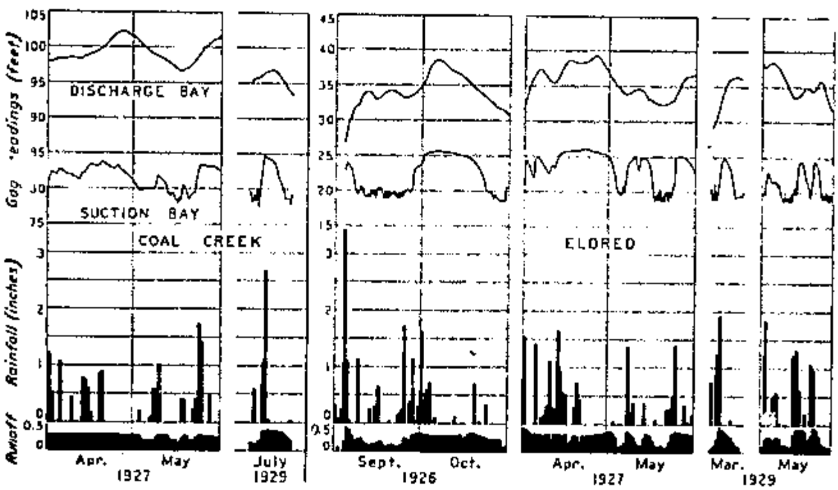

FIGURE 17.—Pumping during flood periods by plant having inadequate capacity. Coal Creek and Eldred plants.

Estimates were made of the required capacities at the maximum lifts and these have been plotted in figure 12.

FORMULA FOR MAXIMUM PLANT CAPACITY

A formula for computing the required capacity of drainage pumping plants in the upper Mississippi Valley was developed. The recommended plant capacities are considerably less than the maximum runoff to be expected. When the runoff from rainfall exceeds the

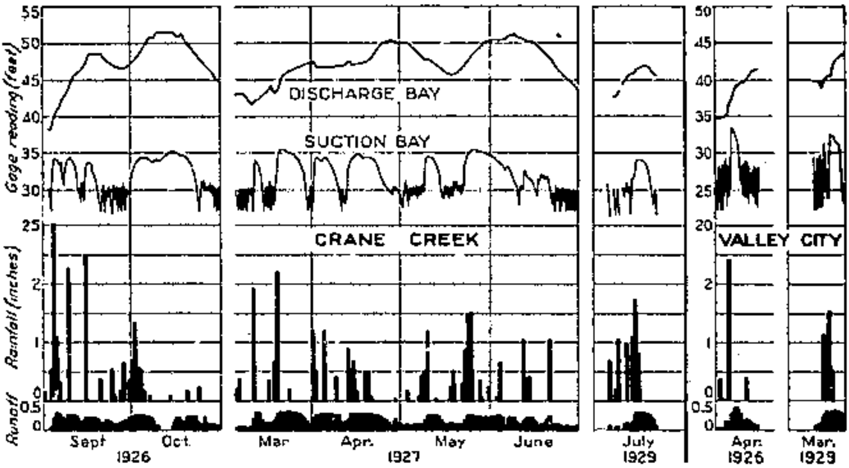

FIGURE 18.—Pumping during flood periods by plant having inadequate capacity. Crane Creek and Valley City plants.

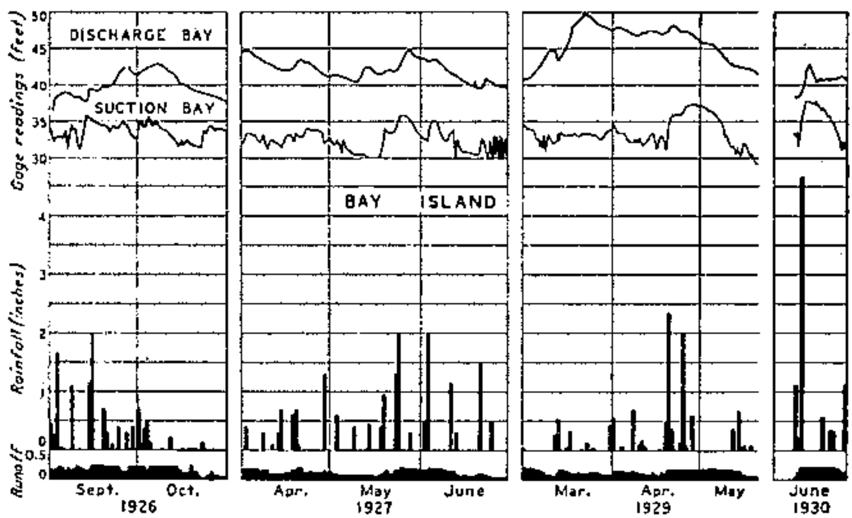

FIGURE 19.—Pumping during flood periods by plant having inadequate capacity, Bay Island plant.

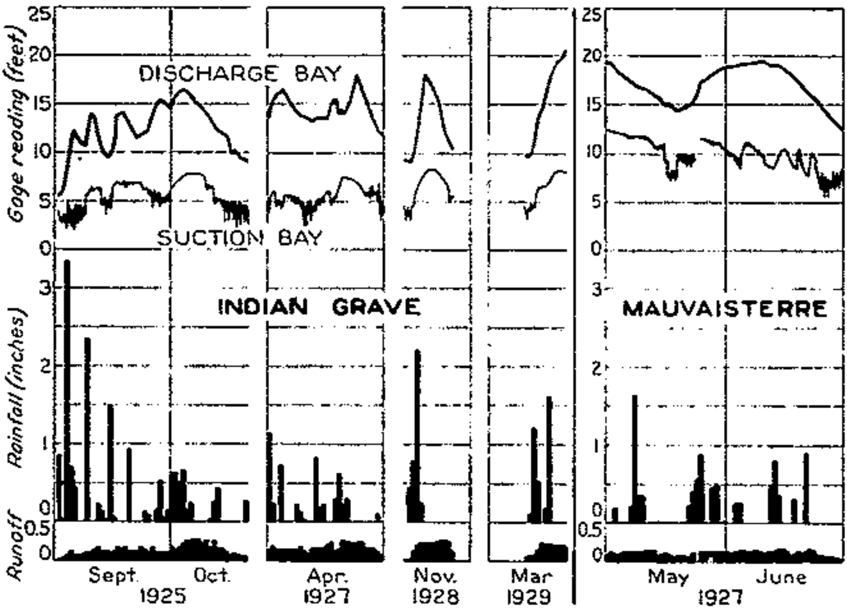

FIGURE 20.—Pumping during flood periods by plant having inadequate capacity, Indian Grave and Mauvaisterre plants.

capacity of the pumping plant, the excess runoff goes into storage. This storage includes ground-water storage caused by a rise in the ground-water table, ditch storage, and surface storage in sloughs and low areas. Storage may occur at locations away from the pumping plant and not be reflected in a large or rapid rise of the suction bay. Evaporation and transpiration losses of water help to dry up a drainage district, especially during the growing season and are then important factors to consider (p. 20).

Crop losses result when the pumping plant capacity is inadequate due to flooding of low areas or to frequent rises in the ground-water table. It is economical to provide a larger pumping capacity to protect land or crops of high value. Since the investigations were completed, land and crop prices have increased. Under present conditions it is believed that the capacities of pumping plants should be increased about 25 percent above the capacities generally considered adequate in 1930. This is due to the need for better protection from frequent damage to crops.

The pumping capacity includes runoff from gravity plus seepage which occurs in pumping districts. However, due to storage which exists in a district, the pumping plant pumps only a fraction of the maximum daily runoff.

These relationships may be expressed as follows:

$$\text{Maximum plant capacity} = \text{coefficient} \left(\begin{matrix} \text{gravity} \\ \text{drainage} \\ \text{runoff} \end{matrix} + \begin{matrix} \text{seepage} \\ \text{runoff} \end{matrix} \right)$$

The formula for maximum plant capacity may thus be modified in general terms as follows:

$$C = K_1[G + (K_2 \times r)] \quad (1)$$

in which

C = plant capacity at maximum lift, in inches runoff per 24 hours.

K_1 = pumping-district coefficient, a ratio between drainage coefficient from land in a pumping district and drainage coefficient of similar land receiving gravity drainage. Where land and crop values are high this coefficient may approach unity if it is desired to give all land the same degree of protection as it would receive for gravity drains. For small areas, such as a few hundred acres, it would approach unity except for storage. However, due to the storage factor, as discussed above, this ratio may ordinarily be reduced. For the Mississippi valley plants studied during 1925 to 1931, the coefficient K_1 was found to be about 0.26. However, due to increased land and crop values it is recommended that it be increased to 0.33 for 1950 conditions.

G = drainage coefficient or drainage modulus for land having gravity drainage expressed in inches per 24 hours. This would be the capacity for which open drains would be designed. For upper Mississippi conditions this would usually be from three-quarters to 1 inch in 24 hours (fig. 2, C curve).

$K_2 \times r$ is a term which gives the seepage flow to be pumped for maximum runoff conditions expressed in inches per 24 hours. Maximum seepage was found to be related to average annual runoff to be pumped.

K_2 is a coefficient (ratio) to convert annual runoff to maximum seepage in inches per 24 hours. This coefficient would be about 0.023 for conditions in the upper Mississippi Valley.

r is average annual runoff to be pumped. For the investigation this was found to vary as follows: 5 to 12 inches per year for districts having considerable gravity drainage; 13 to 16 inches per year for nonseeped districts pumping all runoff; and 16 to 35 inches per year for heavily seeped districts.

The original investigations as modified by later experience would indicate the formula for maximum plant capacity for upper Mississippi Valley pumping plants to be as follows:

$$C = .33\,(G + .023\,r) \quad (2)$$

Where C = maximum plant capacity in inches per 24 hours; G = drainage coefficient for similar gravity drainage systems (fig. 2, C curve); r = annual runoff to be pumped in inches.

This formula was arrived at by empirical methods and is believed to represent a relationship in which observed data seem to be reasonably well related. It is a rational approach to breaking the runoff into two parts, gravity flow and seepage. Coefficients may be arrived at for other areas if sufficient data are available.

Table 7 gives the results of applying this formula to districts studied, and these results are plotted in figure 12.

For other locations formula (1) may be used to advantage. A study of the operating data available in this bulletin and in reference cited should help determine various factors and coefficients. A study of the rainfall data, p. 19), runoff from watersheds data, evaporation and transpiration data (p. 20), data on topography and storage available and on soils and crops should be made to determine pumping requirements of the pumping plant. Recommendation for the factor *G* (drainage coefficient or runoff for drained lands) may be secured from the Soil Conservation Service on request.

PLANT CAPACITY FOR LOUISIANA AND TEXAS

A considerable area of coastal marsh in Louisiana and Texas has been developed by drainage pumping. Early investigations of the drainage pumping plants in southern Louisiana were made by Charles W. Okey, drainage engineer, United States Department of Agriculture (*8*). Examinations were made of 30 drainage pumping districts in southern Louisiana during the period 1911–15. Table 11 shows the operating conditions during this period for 9 pumping districts.

The pumping and reservoir capacity was determined for 25 drainage districts and these capacities were summarized in a bulletin by Okey (*8*). The pumping capacity of the districts then ranged from 0.93 inches per 24 hours to 2.55 inches per 24 hours. Reservoir capacity ranged from 0.28 inch per 24 hours to 2.20 inches per 24 hours. The combined capacity of the pumping plant and reservoir ranged from 1.29 to 3.45 inches per 24 hours. An analysis of the combined 24-hour pumping and reservoir capacities of the 25 districts covered showed that: 2 districts had from 1.00 to 1.49 inches; 5 districts had from 1.50 to 1.99 inches; 11 districts from 2.00 to 2.49 inches; 5 districts from 2.50 to 2.99 inches; and 2 districts from 3.00 to 3.49 inches per 24 hours. It should be noted that the data given in figure 6 indicate that rains in excess of 6 inches in 24 hours may be expected once in 5 years.

The investigations of Anderson and Moore,[4] Western Gulf Region, Soil Conservation Service, revealed that many of the original pumping plants having a capacity of about 1.5 inches per 24 hours or less were found to be inadequate. As a result of the examination by Anderson and Moore a runoff capacity of 3 inches per 24 hours, including pumping capacity and reservoir capacity, is now recommended. Storage is computed as that available in areas below the elevation of the lowest cultivated land. It includes storage available on marsh lands, in borrow pits, and slough. Ditch storage is often so small it can be neglected. For example, if a reservoir storage in a pumping district is 1 inch per 24 hours the pumping plant should be designed for not less than 2 inches per 24 hours. The combined capacity figure of 3 inches per 24 hours applies to land used principally for growing sugarcane. A greater capacity may be advisable for special or truck crops or where local property requires better protection. The recommended combined pumping and reservoir capacities may be reduced to 2 inches per 24 hours if the principal crop is rice or pasture. These rates apply to the area along the Gulf Coast of Louisiana and Texas south of a line from Natchez, Miss., to Natchitoches, La., then roughly parallel to the Gulf Coast to Victoria, Tex. North of this line and in Arkansas the total pumping capacity and storage capacities may be reduced by 20 percent to 2.4 inches per 24 hours for land used principally for cotton and to 1.6 inches per 24 hours for rice or pasture land. These recommendations are based on drainage of flat areas less than 3,000 acres.

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[4] ANDERSON, T. C., and MOORE, R. B. Unpublished Report, 1947.

In the examinations of Anderson and Moore it was found that many of the original plants failed from lack of pumping capacity. Many of the successful districts originally started out with smaller capacity but have increased them. Seven typical drainage pumping installations for 1947 conditions are described in the following paragraphs:

1. Pumping plant consisting of two pumps rated at 38,000 gallons per minute each, will remove 2.5 inches per 24 hours from a 1,600-acre watershed. In the district there are 578 acres of woodland and marsh of which 380 acres may be considered storage area. Storage in overflow area is slightly less than 3.5 inches and total capacity approximately 6.0 inches.

2. Plant contains two pumps rated at 38,000 g. p. m. each, which drain a 2,300-acre area. Pumping capacity is 1.7 inches per 24 hours. Of this area 1,400 acres are in woodland and marsh, of which 820 acres may be considered storage area. Storage amounts to 4.3 inches per 24 hours, total capacity approximately 6 inches per 24 hours.

3. One pump rated at 40,000 g. p. m. drains 1,000 acres. The pumping capacity is 2.1 inches per 24 hours. The storage is not sufficient to give adequate drainage. It is planned to increase the area to 1,200 acres and add another pump with 40,000 g. p. m. capacity. This would give a rate of removal of 3.55 inches per 24 hours.

4. A plant containing two pumps rated at 40,000 g. p. m. each and one at 72,000 g. p. m. gives a rate of removal of 3.4 per 24 hours from a 2,500-acre watershed. Storage capacity is believed to amount to 1.5 inches. The original plant for this district has a much smaller capacity but was enlarged in accordance with pumping experience.

5. A plant that has two pumps rated at 50,000 g. p. m. each can remove 1.2 inches per 24 hours from 4,400-acre watershed. Most of the land is in pasture and hay crops. A considerable acreage is still in woods and swamp which act as a storage area. Truck crops are grown only on the higher land in the area.

6. A pumping plant of two pumps having a combined capacity of 50,000 g. p. m. can remove 3.3 inches from a watershed area of 812 acres. When this plant was examined by Okey (*8*), it had a capacity of 2.15 inches per 24 hours in 1917. The reservoir capacity was then 0.40 inch per 24 hours. This capacity was found to be inadequate and the pumping plant was subsequently increased to the present figure of 3.3 inches per 24 hours.

7. A pumping plant of two pumps having a combined capacity of 65,000 g. p. m. can remove 3.4 inches from a watershed area of 1,015 acres. This district was also examined by Okey in 1917 when the original plant had a capacity of 1.91 inches per 24 hours. The reservoir capacity was then rated at 0.67 inch per 24 hours. This pumping capacity was found to be inadequate and later increased to 3.4 inches per 24 hours.

Plant Capacity for Florida

For open-ditch drainage of the Everglades the engineering board of review (*6*) developed and recommended the following formula:

$$Q = \frac{69.1}{M} + 9.6$$

in which Q=runoff in cubic feet per second per square mile of drainage area, and M=drainage area in square miles.

This formula was used by L. A. Jones (4) in preparing plans for water control of organic soils of the region. The formula provides 1-inch runoff depth per 24 hours from 16 square miles, three-fourths-inch from 43 square miles, and one-half-inch from 322 square miles. As in other areas this formula is based on the assumption that the land will be overflowed for short periods of time following excessive rainfall. It is not intended to provide complete flood control.

B. S. Clayton, drainage engineer, Soil Conservation Service, in conducting long-time investigations of conditions in the Florida Everglades found that many of the northern Everglades pumping districts serve from 5 to 13 sections of land. Most of these pumping plants were designed to remove 1-inch runoff per 24 hours. Much of this land is used to grow sugarcane. Experience over a 20-year period indicates that the 1-inch rate is generally ample for growing sugarcane on the organic soil of the area.

It was found, however, that truck crops suffered losses where the capacity was only 1-inch per 24 hours. As a result of Clayton's investigations the following rates are recommended for land used for growing truck crops in organic soils: 3.0 inches for 1 section of land or less, 2.0 inches for 2 to 3 sections of land, 1.4 inches for 4 to 9 sections of land, and 1.0 inch for 10 to 16 sections of land.

These rates approximate those given by the runoff formula quoted above. A long pumping record at the Everglades experiment station, Belle Glade, Fla., indicated that a runoff of 3 inches per 24 hours was required to protect crops on areas of 1 square mile or less.

In recent years a considerable number of pumping plants have been installed to serve land used for growing pasture grass for cattle. These pumping plants usually drain from 2 to 4 sections of land. A runoff of from 1 to 2 inches per 24 hours was commonly provided for these grazing areas and appeared to be adequate.

Minimum Runoff Pumped

The minimum rate of runoff 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, mixed flow, and axial flow or propeller pumps are commonly used for drainage pumping. In double-suction centrifugal pumps (fig. 21) the liquid enters both sides of the impeller at its hub and flows radially to the periphery. In single-suction centrifugal pumps the liquid enters only one side of the impeller. In centrifugal pumps the discharge pressure is developed principally by the centrifugal forces. In mixed-flow pumps the discharge pressure is developed partly by centrifugal force and partly by the lift of the vanes. This type of pump has a single-suction impeller, the discharge is in axial and radial directions and the casing is usually of the volute type. In the propeller type or axial-flow pump the pressure is developed by the lift of the blades. The discharge is in an axial direction. This type is often called a screw pump.

Centrifugal pumps may be designed for efficient pumping against total heads exceeding approximately 12 feet. Mixed-flow pumps can operate efficiently at heads from 6 to 26 feet. The axial-flow or propeller-type pump is ordinarily limited to pumping against heads less than 10 feet.

The advantages of using more than one type of pump should be considered in proposed installations, such as a centrifugal pump for

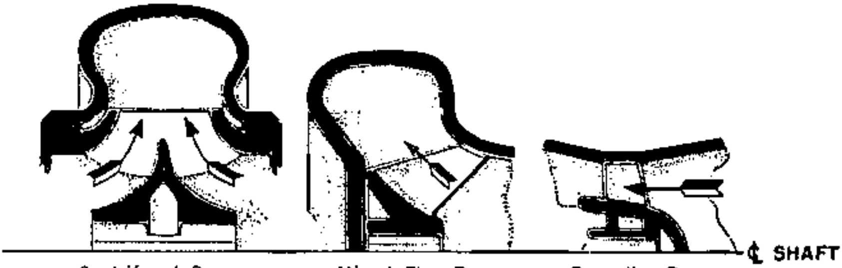

FIGURE 21.—Sketch of pumps used for drainage. [Courtesy of Hydraulic Institute, New York City.]

use at higher heads, and a propeller 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 propeller 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 less than low-speed motors and a high-speed pump can be directly connected to a cheaper motor.

Pump Efficiencies

Usually, drainage pumps may be furnished with the maximum efficiency something over 80 percent. A well-designed pump should have an efficiency above 70 percent over a wide range of operating lifts.

Important pump installations are usually supplied by a manufacturer based on specifications prepared by the purchaser. The purchaser specifies the requirements at maximum and other operating lifts. Generally, the manufacturer supplies a set of characteristic curves to the purchaser similar to those shown on pages 43 and 44. Such curves show pump capacity related to head, speed, efficiency, and brake-horsepower requirements. Most manufacturers base such curves on factory tests of the pump furnished or on the measurements of a geometrically similar pump.

Pump efficiency is computed by the following formula:

$$e = \frac{g.p.m. \times H_t}{BHP \times 3960}$$

where

e = pump efficiency
$g.p.m.$ = gallons per minute
H_t = total head on pump
BHP = brake horsepower input into pump shaft

TOTAL HEAD

The total head on the pump is equal to the total energy in the water at the discharge flange minus the total energy at the suction flange of the pump. It is expressed by the formula

$$H_t = \left(H_d + \frac{V_d^2}{2g} + d_1\right) - \left(H_s + \frac{V_s^2}{2g} + d_2\right)$$

in which H_t is the net total head, in feet of water.

H_d is the discharge pressure head, in feet of water, measured near the discharge flange of the pump. It is positive if the pipe is under pressure, and negative if under vacuum, at the point of measurement.

V_d is the average velocity, in feet per second, in the pipe at the point where H_d is measured;

d_1 is the elevation of the gage measuring H_d in feet above some reference plane. It is positive or negative depending upon whether the gage is above or below the reference plane.

H_s is the suction pressure head, measured near the suction flange of the pump. It is nearly always negative, since the suction pipe is usually under vacuum.

V_s is the average velocity in the pipe at the point where H_s is measured.

d_2 is the elevation of the gage measuring H_s above the same reference plane from which d_1 is measured.

g is the acceleration due to gravity, used herein as 32.16 feet per second per second.

The expressions $\frac{V_d^2}{2g}$ and $\frac{V_s^2}{2g}$ are the velocity heads in the discharge and suction pipes, respectively.

The total head is equal to the static lift plus all losses in suction and discharge pipes.

SPECIFIC SPEED AND MAXIMUM SUCTION LIFT

The specific speed [5] of an impeller is a valuable index of the type of pump and is important in determining the maximum suction lift. The specific speed of an impeller is the revolution per minute to which a geometrically similar impeller would run if it were of such size as to discharge 1 gallon per minute against 1-foot head. Centrifugal pumps

[5] See note of acknowledgment, p. 2.

are generally designed for specific speeds ranging from 1,500 to 6,000 for double-suction pumps. Mixed-flow and axial-flow pumps are generally designed for specific speeds from 4,000 to 20,000.

The formula for specific speed is as follows:

$$N_s = \frac{\sqrt{g.p.m.} \times r.p.m.}{H_t^{3/4}}$$

Where N_s=specific speed, g. p. m. is gallons per minute, r. p. m. is revolutions per minute, H_t is total head for a single stage pump which is used in drainage plants.

Excessive speed with too high a suction lift often results in serious

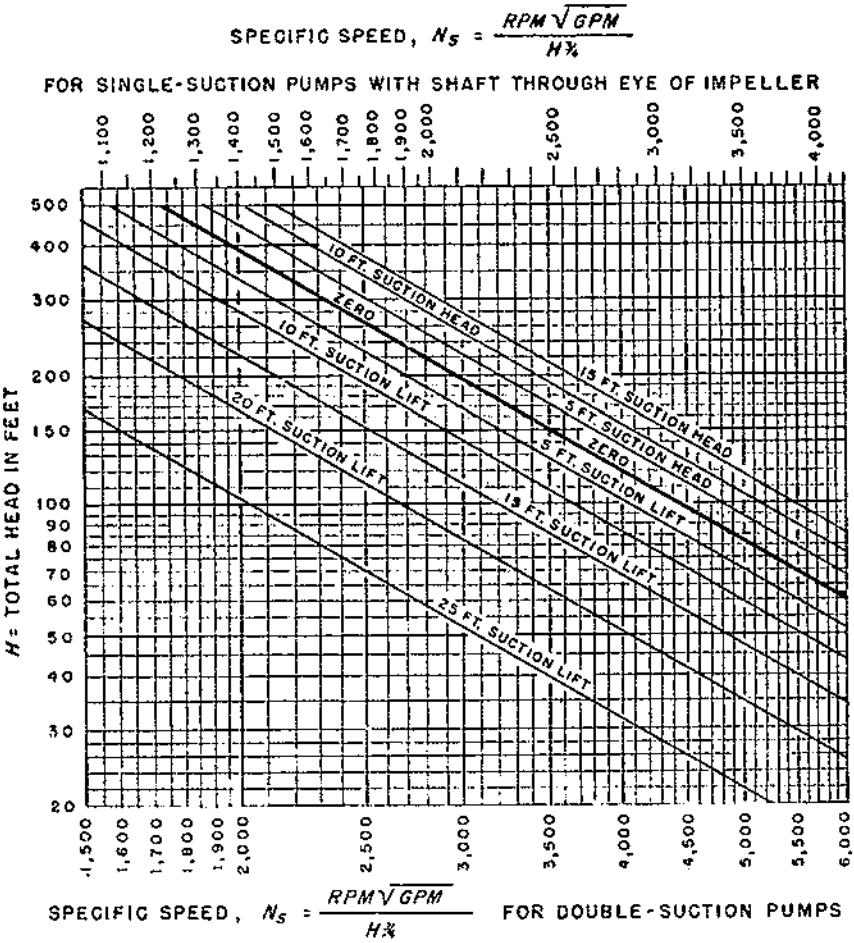

FIGURE 22.—Hydraulic Institute upper limits of specific speeds for single-stage pumps pumping clear water at sea level at 85° F. for centrifugal pumps. [Courtesy of Hydraulic Institute, New York City.]

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trouble from vibration noise, pitting, and cavitation. A pump with a low specific speed will operate safely with a greater suction lift than one of a higher speed. If the suction lift is fairly high (over 15 feet) it is especially necessary to give particular consideration to the pump design. Usually this requires a slower speed and a larger and more expensive pump. If the suction lift is low or the specific speed may be increased, a smaller and cheaper pump may be used.

The Hydraulic Institute, of which leading pump manufacturers are members, has adopted standards (*3*) which cover upper limits of specific speed with respect to capacity, speed, head, and suction lift. These curves are shown in figures 22 and 23. In many drainage pumping plants the suction lift is high when pumping at low stages. Some impellers have been severely pitted by cavitation. This was no doubt influenced by the high specific speed for the suction lift. It is important for plant designers to keep the suction lift as low as possible and in all cases avoid exceeding limits shown in figures 22 and 23.

Centrifugal Pumps

The double-suction volute centrifugal pump was used almost exclusively in drainage plants in the upper Mississippi Valley for about 20 years prior to 1928. Single-suction centrifugal pumps were used in many earlier plants, but few have been used in recent years.

Considerable development of centrifugal pumps has taken place since 1915. An increase of almost 20 percent in efficiency has been ob-

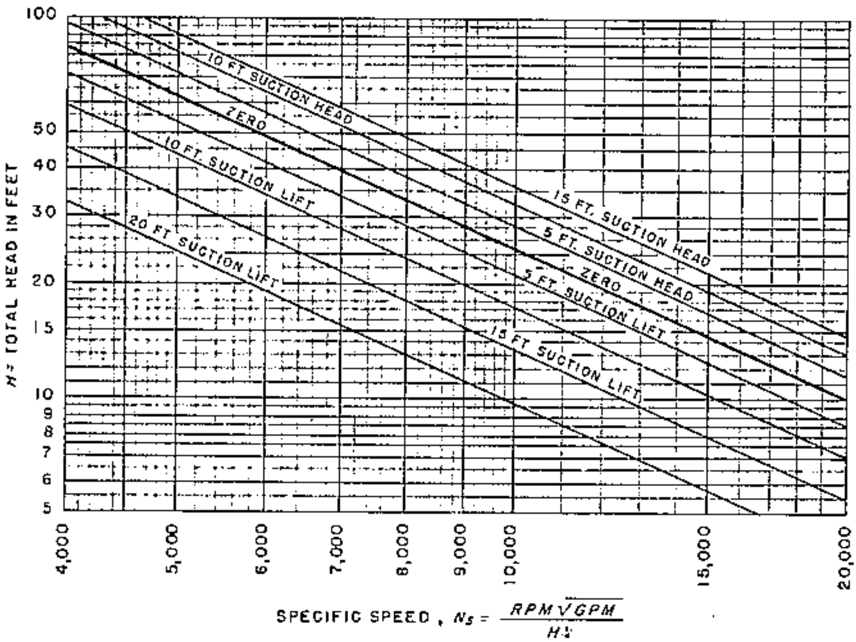


FIGURE 23.—Hydraulic Institute upper limits of specific speeds for single-stage pumps pumping clear water at sea level at 85° F. for single-suction mixed-flow and axial-flow pumps. [Courtesy of Hydraulic Institute, New York City.]

tained by improved design. 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. Typical characteristic

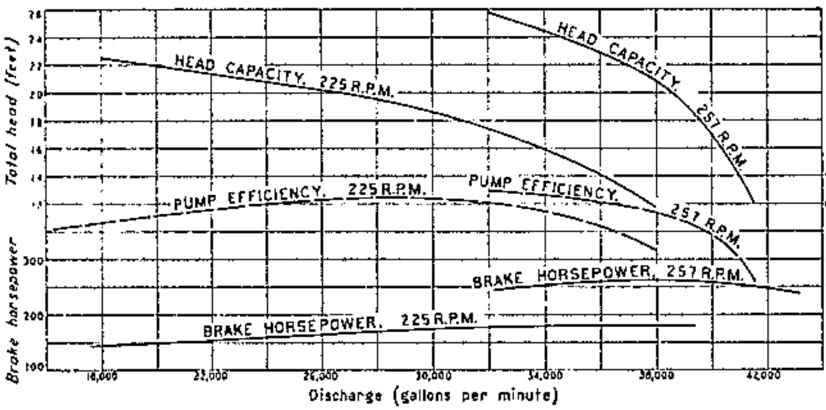


FIGURE 24.—Characteristic curves of typical modern centrifugal pump (taken from field tests of 36-inch pump, Hunt plant).

curves are shown in figure 24. The Francis-type impeller, with 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.

Axial Flow or Propeller Pump

The axial flow or propeller pump is especially adapted for low-head pumping (pl. 1, B). This type is also called a screw pump. The impeller has several 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 use extensively in Louisiana and Florida for many years but has been used less in the upper Mississippi Valley which is due no doubt to the high maximum heads in the latter area.

The propeller 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. 25). 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.

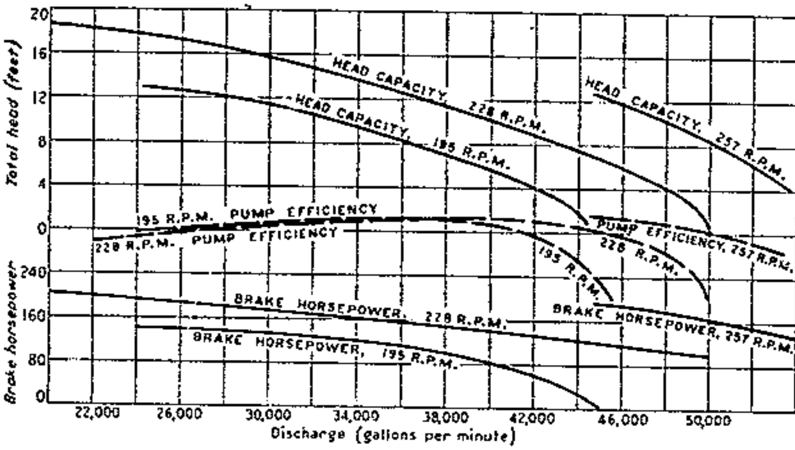


FIGURE 25.—Characteristic curves of typical propeller pump.

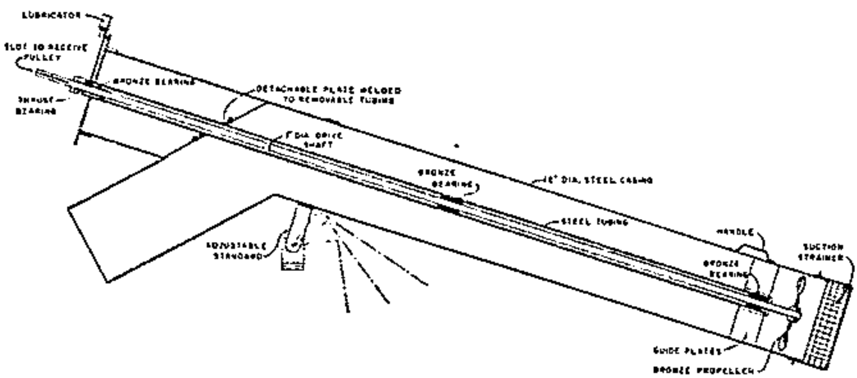


FIGURE 26.—Propeller pump for light service.

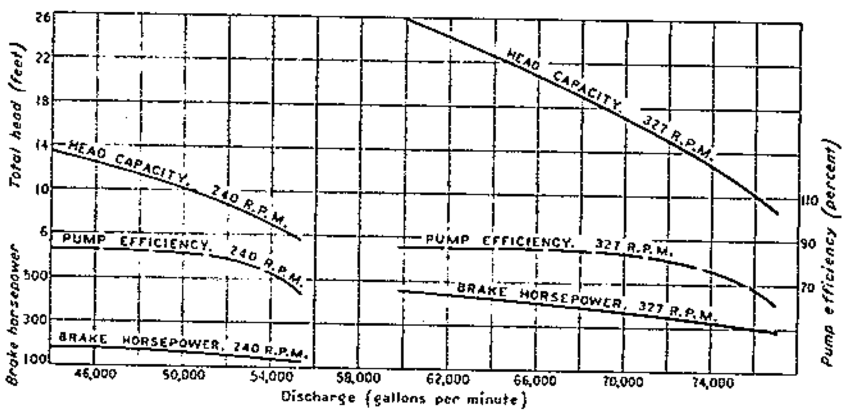


FIGURE 27.—Characteristic curves from tests of 48-inch mixed-flow pump at Hartwell plant.

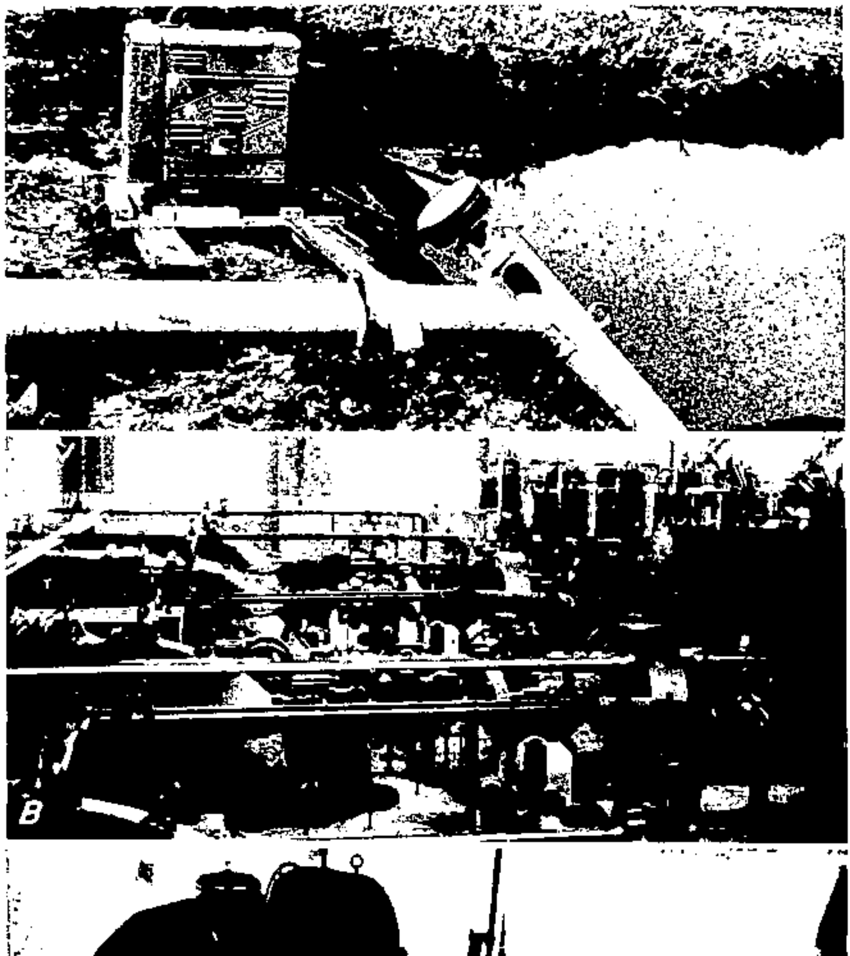


PLATE 1.—A, Light propeller pump for drainage pumping operated by gasoline engine, Florida Everglades. B, 54-inch screw pumps direct connected to 240-horsepower Diesel engines. (Fourth unit hardly shows in the foreground.) East Side levee and sanitary district. C, A 30-inch mixed flow pump direct-connected to 250-horsepower induction motor. Lima Lake drainage and levee district.

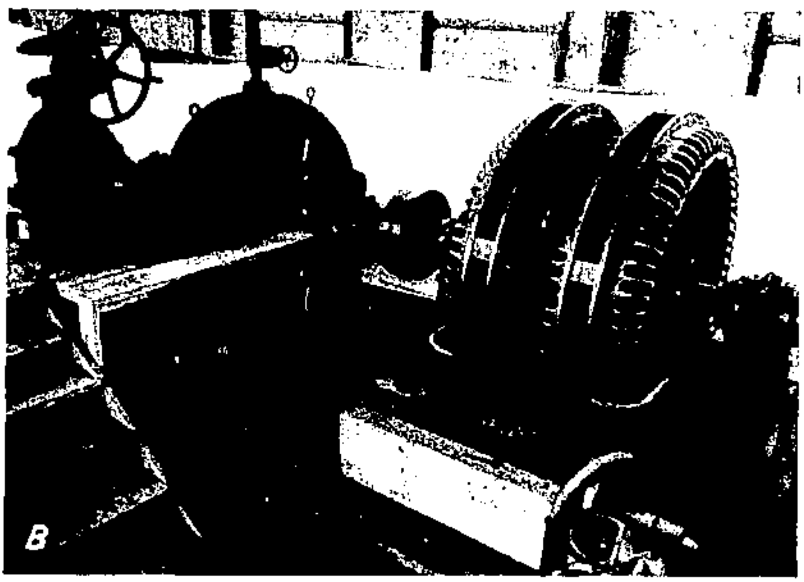


PLATE 2.—*A*, Vertical 35-horsepower motor which is direct-connected to 10-inch submerged pump. Ninta drainage and levee district. *B*, Two 250-horsepower synchronous motors on one shaft to drive 36-inch bottom-suction centrifugal pumps at two speeds; also hand-operated gate valve. Lima Lake drainage and levee district.

This type 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, such as Florida, where supplemental irrigation is desirable and in organic soils. One manufacturer provides a trash cutter which permits the removal of trash from the blades without opening the pump.

An interesting development of the propeller-type pump has been its manufacture out of a welded steel pipe with a propeller similar to a boat propeller attached to a shaft and welded inside the pipe casing (fig. 26). This type of pump has been made at some local machine ships. A considerable number have been sold to farmers for drainage of small tracts less than 200 acres. The advantage of this type of pump is its low cost. The disadvantage is that little is known about the performance characteristics and efficiency of individual pumps. Some have been built with such light outside casings that small blocks of wood passing the screen have broken through the casing. In other cases the annual cost of pumping with such units was so low that a higher efficiency would not have too much effect on annual savings.

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 as shown in figure 21. It operates efficiently against somewhat higher heads than the true screw pump. With one change in speed the Hartwell mixed-flow pump operated at 70- to 80-percent efficiency at all heads from 6½ to 26 feet, and the discharge did not decrease excessively at the higher heads (fig. 27). The open-type impeller of the mixed-flow pump facilitates the passage of trash.

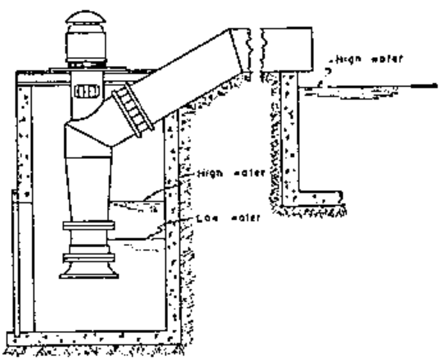


Figure 28.—Submerged pump discharging above high water. [Courtesy of Food Machinery Corp., Peerless Pump Division, Los Angeles, Calif.]

Vertical Submerged Pumps

Submerged single-suction pumps have been used in some large drainage plants. However. their chief application has been in smaller plants for the drainage of individual farms or smaller areas (figs. 28 and 29). Usually the motor or engine to drive the propeller is mounted at the head of the unit. Such pumps may be equipped with any type of impeller driven by a vertical shaft. The manufacture, design, and efficiencies of such units vary greatly. One 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. This is a very important advantage for the farm or smaller pumping plant. Many small plants would not be economical if they required constant attendance.

The greatest disadvantage of vertical types is the inaccessibility of the pump for cleaning. When this type is installed, provision should be made to clean the pumps by (1) closing the suction bay, preferably

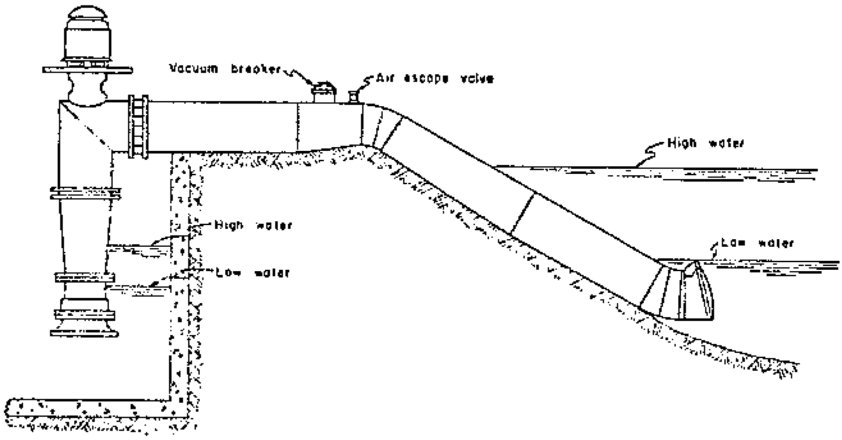


FIGURE 29.—Submerged pumping discharge below low water. [Courtesy of Food Machinery Corp., Peerless Pump Division, Los Angeles, Calif.]

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. The difficulty in cleaning this type of pump makes it especially desirable to provide an effective screen.

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 24 and 27 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 motors mounted on the same shaft (pl. 2, *B*). Belt-connected units could obtain speed changes only by changing pulleys. The Hunt pump (fig. 24) 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

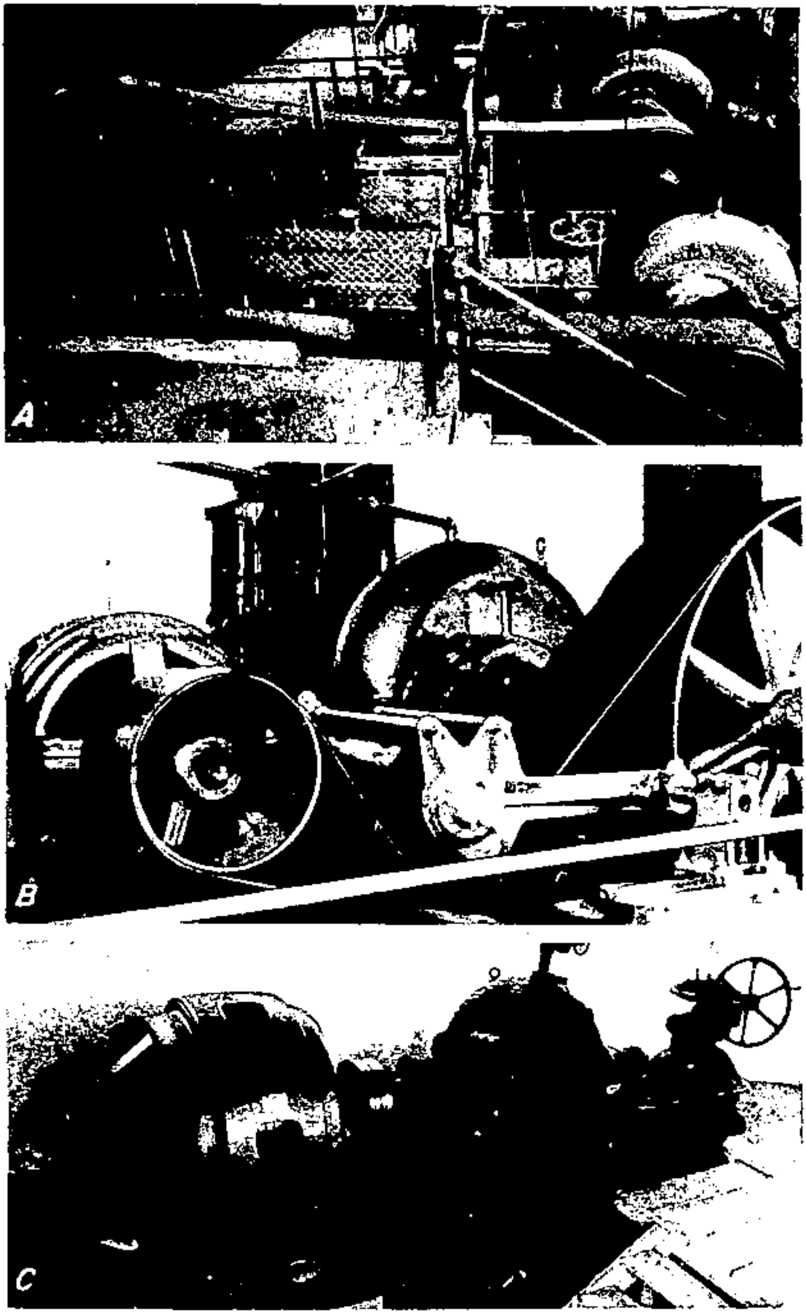
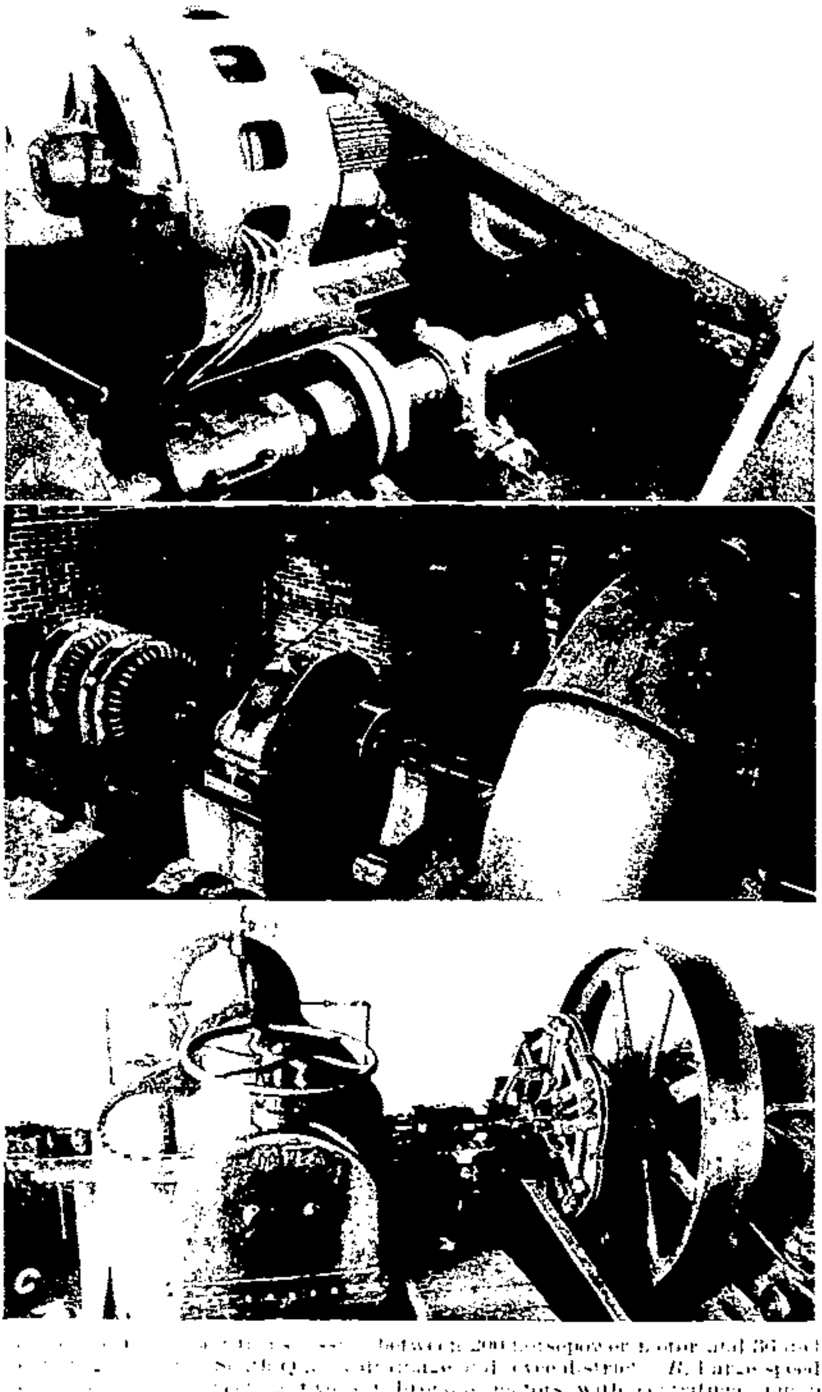


PLATE 3.—A, Typical belt-connected pumping units, Lacey, Langellier, and West Mata[illegible] drainage and levee district. B, Belt transmission using [illegible] pulley where distance between motor and pump shafts is short. [illegible] and levee district. C, Low-speed induction motor direct-[illegible] bottom-suction centrifugal pump, for use at high lifts [illegible] Lake drainage and levee district.



[illegible] between 200-horsepower motor and 36-inch [illegible] S. 25 Q[illegible] [illegible] *B*, Large speed [illegible] with [illegible] *C*, Clutch connection between [illegible] pump. Union [illegible] gives efficiencies in excess of 70 percent. A 200-revolution-per-minute pump speed probably would have given 70-percent efficiency at heads from 10 to 17½ feet, and substituted for the 225-revolutions-per-minute speed would have enabled the plant to operate at 70-percent efficiency or better at all heads from 10 to 25 feet. The speed adjustment of the Hartwell pump (fig. 27) 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.

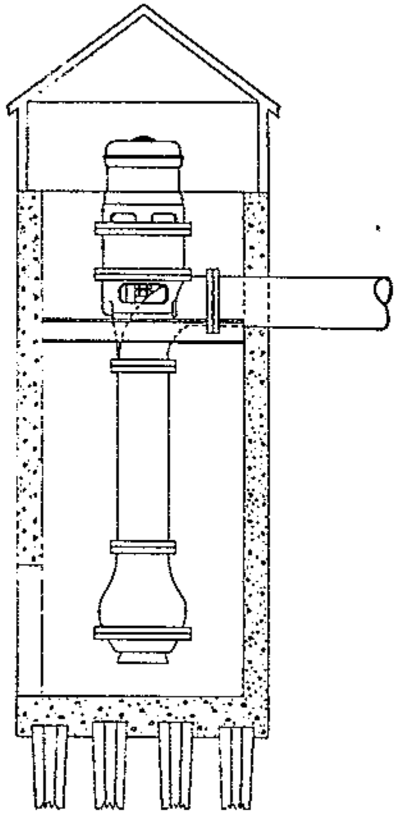


FIGURE 30. — Submerged vertical pump direct-connected to vertical motor resting on piling.

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 less floor space is occupied by each unit. Low-speed induction and synchronous motors that can be direct-connected to pumps are 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 plant, however, a special chain hoist enabled 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, 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 generally seems 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.

A clutch connection is sometimes 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. For high-speed Diesel engines a twin-disc reduction gear and a chain gear to further reduce the speed may be used. One unit of this type has operated satisfactorily for 8 years where the diesel engine operated at about 1,500 r. p. m. and the pump at about 180 r. p. m. Accurate alignment in accordance with the manufacturer's directions is essential for good results.

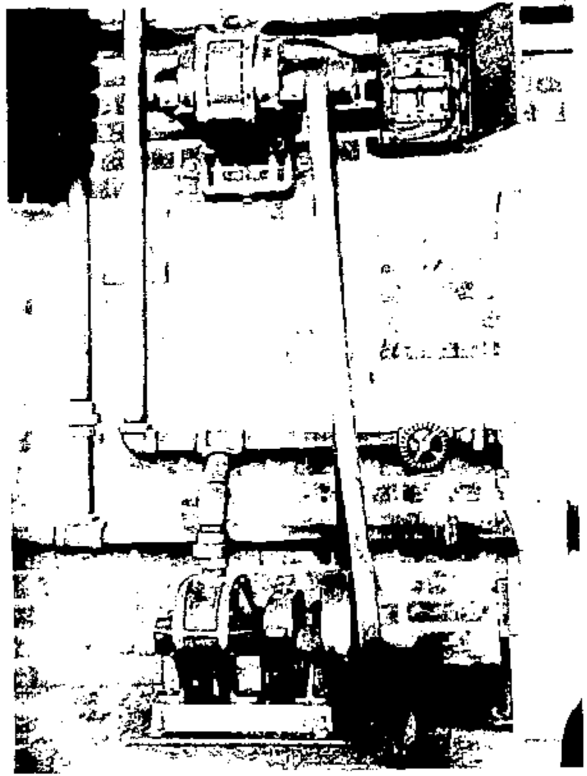
Size and Number of Pumps

Discharge openings of drainage pumps in the upper Mississippi Valley range from 18 to 60 inches in diameter, 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 propeller 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 propeller or mixed-flow pump, as those sizes often have comparable capacities at the maximum lift. Of 24 pumps installed in 3 years, 1928–30, 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 larger pumps.

It is an advantage to have two or more units in a plant so that a breakdown 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



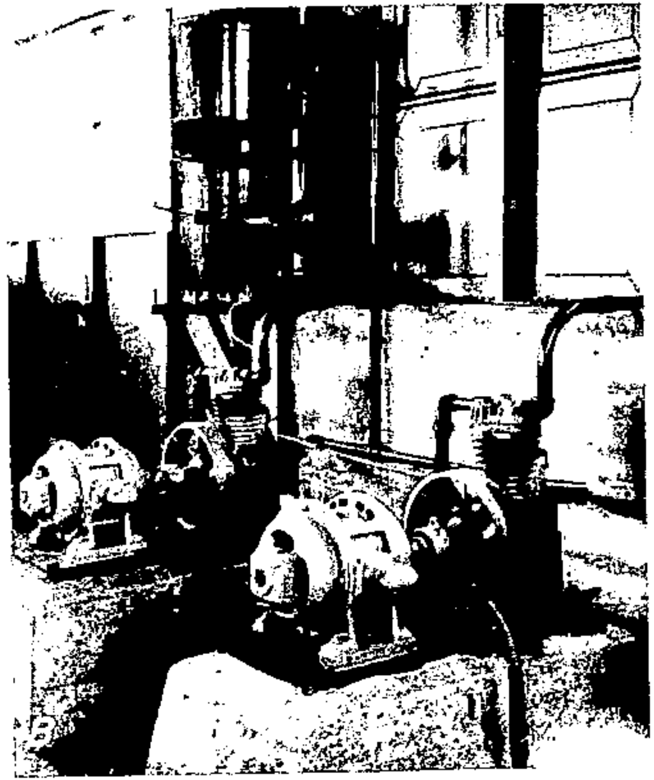


PLATE 5. *A*, Wet-vacuum priming pump driven by a small electric motor. Marion County drainage and levee district. *B*, Dry-vacuum priming pumps with tank in suction line to trap out water. Consolidated drainage and levee district.

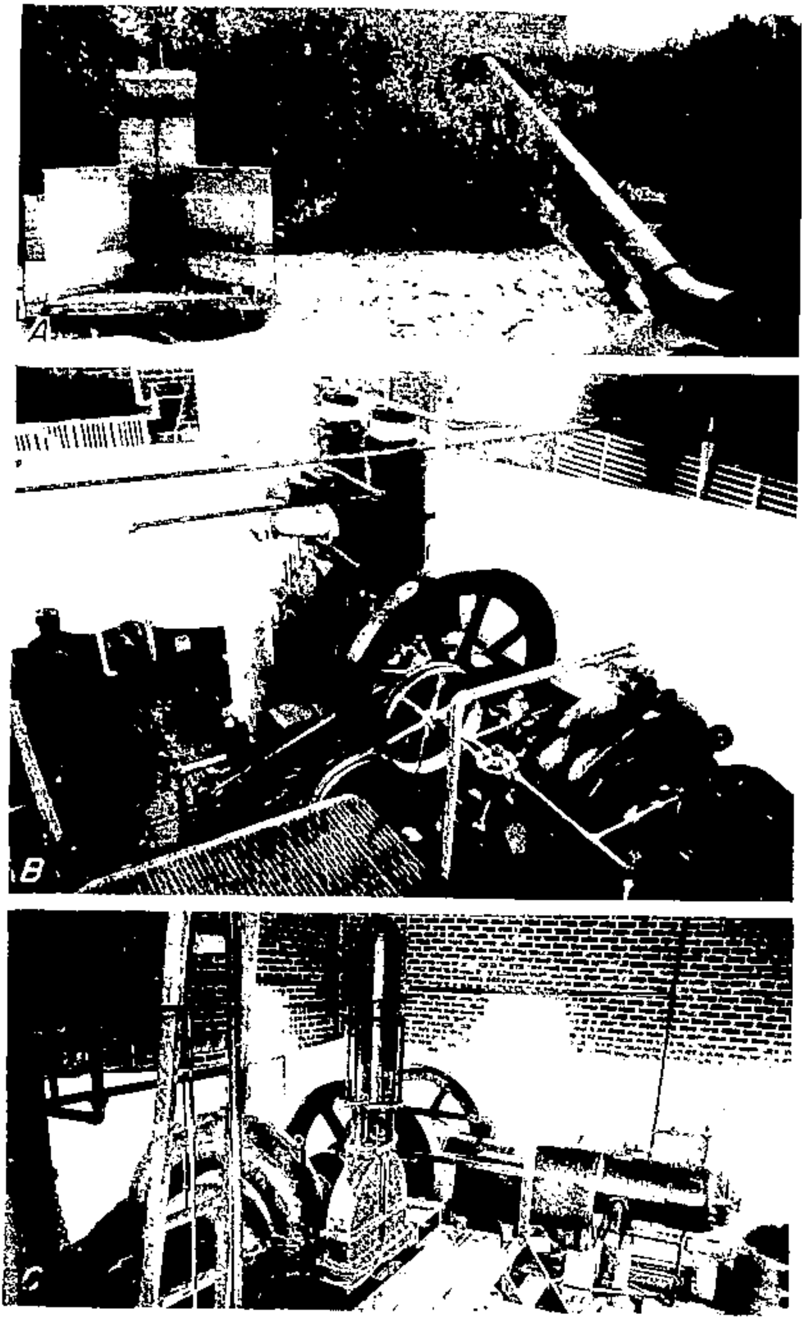


PLATE 6.—A, Erosion in discharge bay at outlet of discharge pipe and gravity sluiceway. Mauvaisterre drainage and levee district. B, Vertical 2-cylinder 100-horsepower semi-Diesel engine direct-connected to 24-inch centrifugal pump; also belt-operated gate valve. Valley City drainage and levee district. 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.

head. Lower velocities are desirable if the plant operates at such a high annual plant factor[6] 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 and pipe 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 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.

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 virtually 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 also may be used. 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. To assure this, special packing 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. They are also 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.

[6] The annual plant factor is the 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.

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. This may not prevent damage, however, 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 virtually 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 every few years. The priming equipment must be 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

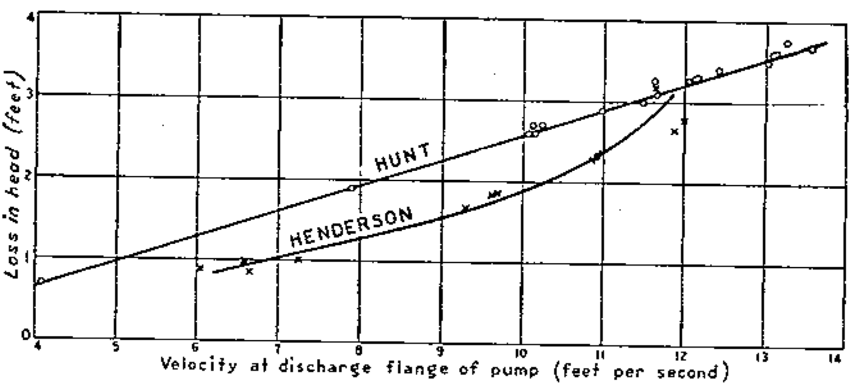


FIGURE 31.—Loss of head in typical well-designed suction and discharge pipes.

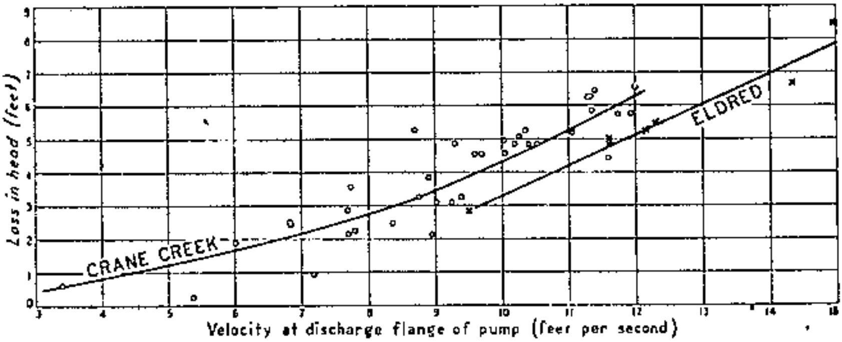


FIGURE 32.—Loss of head in suction and discharge pipes where discharge pipe was not expanded.

somewhat larger than the pump connections so that friction losses may be reduced to an economical point, and the changes in size should be gradual.

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 (*11*) 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 31 and 32, and the descriptions of those pipes are given in table 8. The head losses were computed by deducting the static lifts from measured total heads on the pumps. They include the losses due to any trash and other obstructions in the pipes, to air entering the discharge pipe where it is under vacuum and to eddies where pipe sizes change. as well as the entrance, friction, and exit losses. Tests were selected where the pipes were apparently free from obstructions or excessive amounts of air entering. Theoretically most of these hydraulic 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.

The Henderson County pipes were expanded more than any of the other units shown, and the loss in head was least (table 8). The suction pipes were expanded in the ratio of 4.4 : 1, between pump flange

TABLE 8.—*Descriptions of riveted-steel suction and discharge pipes for which losses of head are shown in figures 31 and 32*

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	10	140	91
Diameter ... inches	48–54	36–42	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	1.9	2.6	3.3	4.3

[1] Indeterminate; see p. 52.
[2] Not determined; see p. 52.
[3] Gate valve at pump; flap gate at end of pipe.
[4] Flap gate at end of pipe.

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 unit 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 approximate dimensions 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 of the same size throughout 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 33. Analysis of the total head on the pump is given on page 40. The loss in discharge pipes was determined from the Scobey formula (*10*), with $K_s=0.51$ applicable to pipe three-sixteenths to seven-sixteenths inch thick, having all seams held by rivets with projecting heads, and pipes approximately 15 years old conducting nonaggressive waters. Losses in welded pipe would be somewhat less at 15 years of age but would equal those shown at a greater age, depending on the roughness of the pipe interior.

Expanding the suction pipe permits lowering of the suction bay by pumping 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.

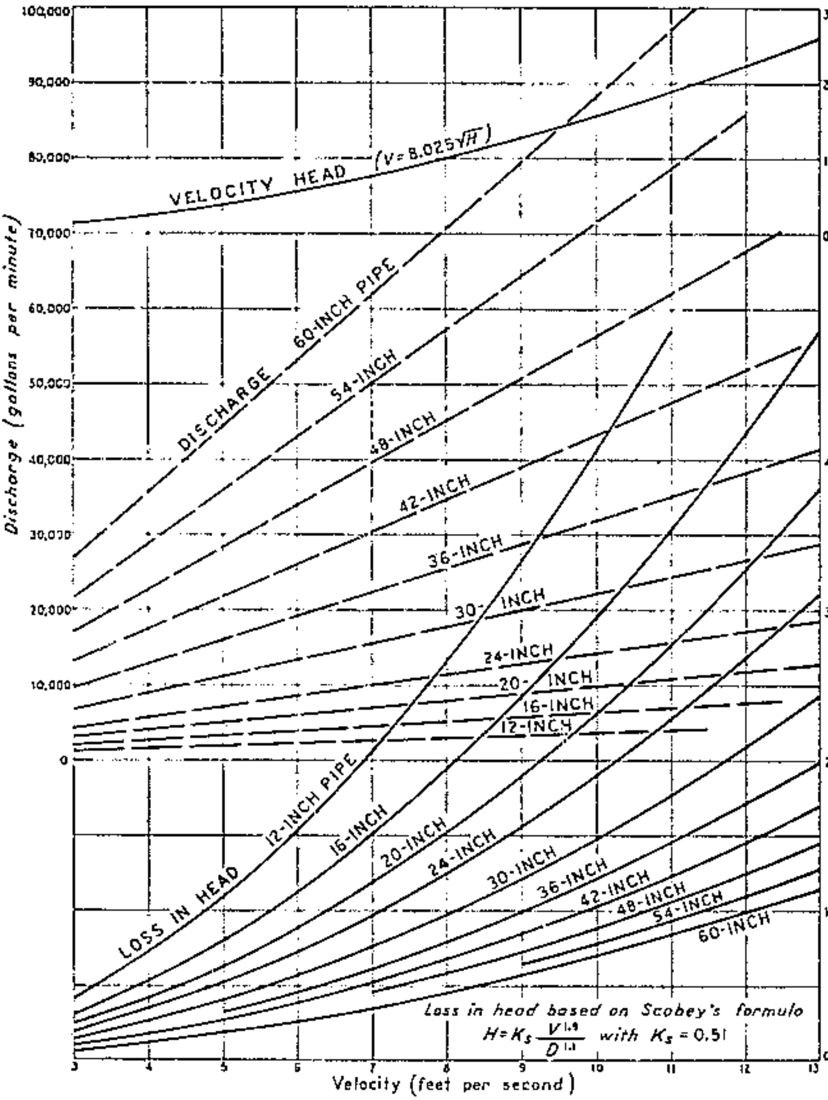


FIGURE 33.—Velocity head, discharge, and friction-head loss in riveted steel pipes.

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 (*17*) 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 33. Part of the velocity head may be recovered by expanding the discharge pipe. The friction loss in a well-designed suction pipe expanded 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. Unless the pump is submerged 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, according to the limited data available on this subject.

The end of a discharge pipe often is belled to reduce the exist 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 reductions in head loss. 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 one-fourth to seven-sixteenths inch thick usually has been used for both the suction and the discharge pipes because curves and expanding sections, could readily and economically be manufactured from it. 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 one-fourth inch thick, or more than 54 inches in diameter and of plate metal three-eighths inch thick.

Welded pipe costs less than riveted steel pipe and has been generally used in recent years. 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 a durable 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. Corrugated and light metal pipe has been used for numerous small plants.

Reinforced concrete has been used in a few instances 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. In a few of these plants hoods were placed over the entrances to obtain lower water levels. Trash wedging in such a suction bay cannot be removed easily. Concrete construction for suction pipes is not usually recommended.

Valves or Gates in the Discharge Line

Some type of valve or gate must usually be installed in the discharge line of a pumping plant to facilitate priming and to prevent 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 34. If there is no effective gate or valve in the dis-

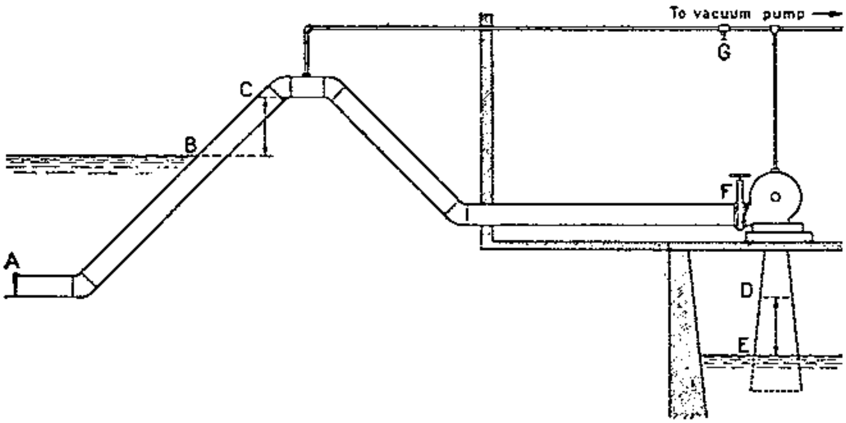


Figure 34.—Typical conditions during the priming of drainage pump.

charge 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. 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 propeller pump and the mixed-flow pump can be started before they are 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 little 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. Owing to its cost, the use of such a valve has been limited to a few plants.

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 backflow into the district and no gate valve is to be used, it may be desirable that a manhole be placed inside the levee near the top point. Otherwise, it may become necessary to cut into the pipe with an acetylene cutting torch in an emergency.

Gate valves smaller than 36 inches, 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 used most 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 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 has yet been used for drainage pumping in the upper Mississippi Valley.

The factors affecting choice of power type for drainage pumping in the upper Mississippi Valley have been discussed in another publication (*11, pp. 96–97*). In 1930, pumping with oil engines costs somewhat less than pumping with electric power, except for small annual pumping, but electric power was generally more convenient and dependable and plant operation was simpler. In a district having 2 or 3 separate plants the smaller ones might be automatic in operation and hence require only one operator if electric power is used; or the smaller plants might be operated by nearby farmers (7) 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

$$BHP = \frac{0.0002526 H_t Q}{e_p e_t}$$

in which,

BHP = brake horsepower required
H_t = 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. Pump characteristic curves and a proposed prime mover should be considered for: Load at the shut-off point, the starting load, and load at total heads below the maximum. The unit should be designed to operate satisfactorily under all operating conditions.

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 three or four units of equal size rather than one or two larger ones. When three or more units are used, one can be sealed off for a year much more frequently than when only two 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 may be 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 during part of each year.

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 have been 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 incentive 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 Missisippi 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 curent 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 curent 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. These, however, add to the cost of the motor. Since 1928 several drainage pumping plants in the upper Mississippi Valley have installed synchronous motors. Some installations include two synchronous motors of different speeds mounted on the same shaft, so the speed of the pump can be changed without changing pulleys. Usually both motors have been of the same horsepower 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 one example 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 would average from 5 to 10 percent higher, partly because the pulleys often are not changed when necessary for economical pumping. The savings 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 for use over 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 one 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. It is also 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 alinement 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 satisfactory when more economical than other types, but for engines of more than 150 horsepower it is 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. Antirust compounds should be considered for use. 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 makes it necessary to increase the power needed 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° Baumé. 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 sulfurous-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. This frequently is a soft swamp not suitable for foundations. 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 should 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, 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 35, would avoid such difficulties. It is believed

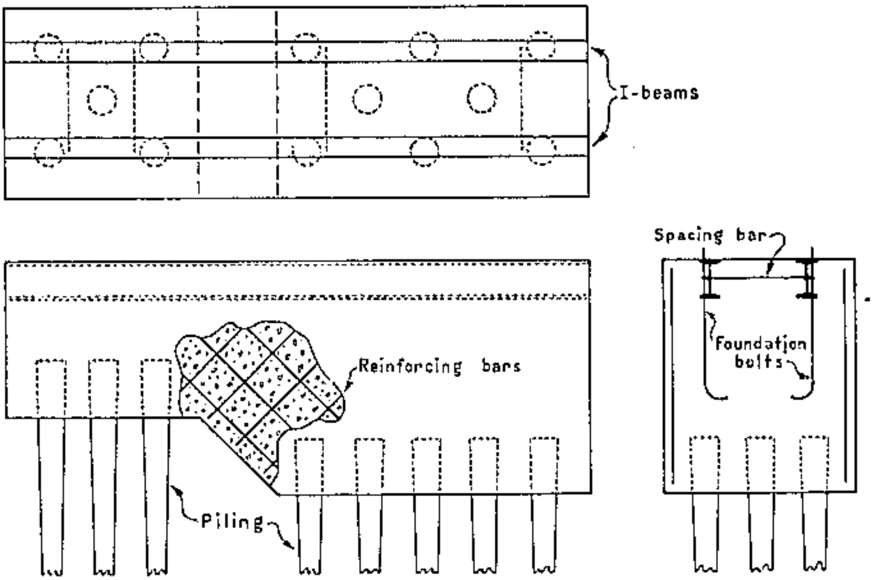


FIGURE 35.—Suggested reinforced concrete foundation for direct-connected pumping unit.

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 to be 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 36. An arrangement economical of floor space is illustrated in figure 37. The belt connection facili-

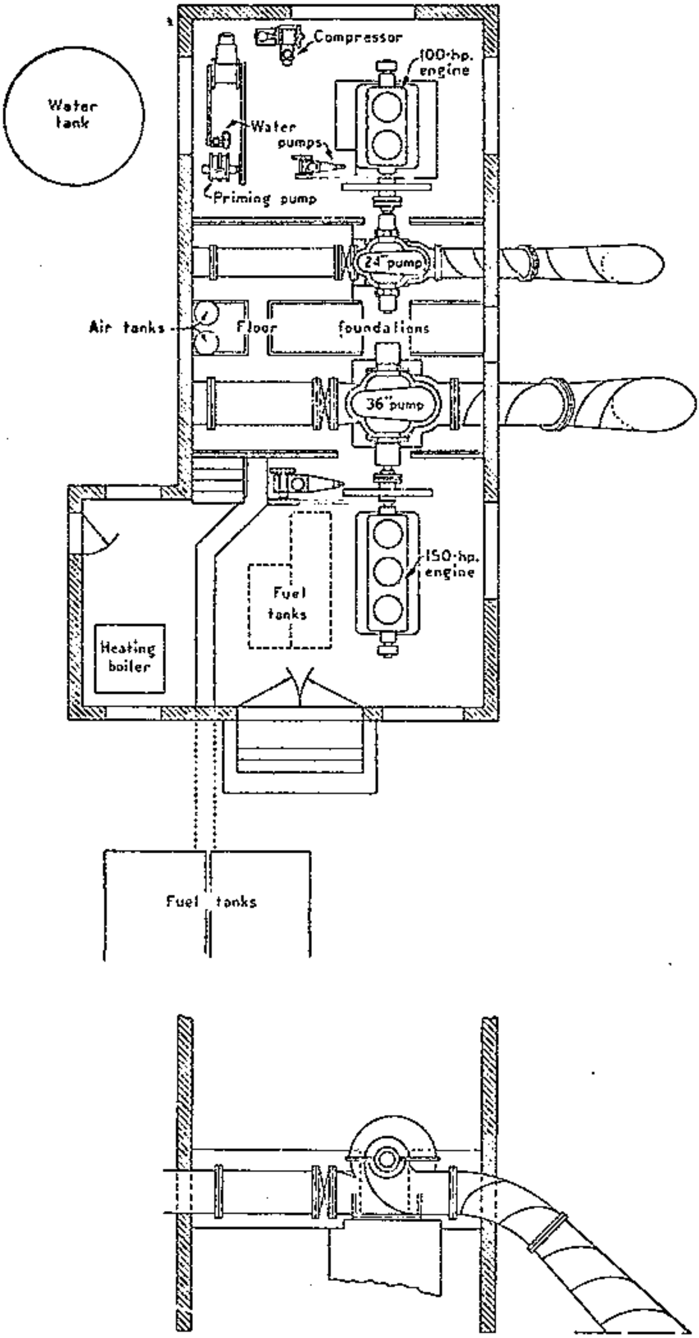


FIGURE 36.—Plan of Valley City pumping plant, comprising 2 semidiesel engines direct-connected to centrifugal pumps and accessory equipment.

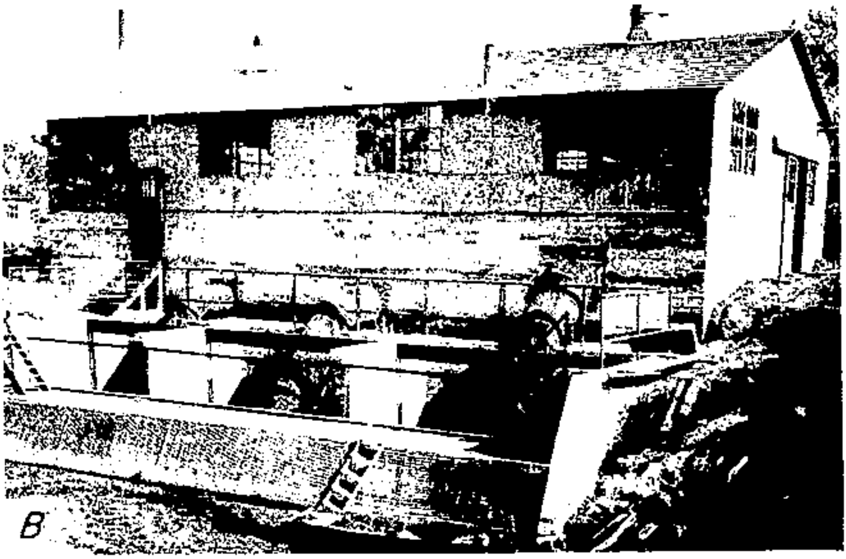
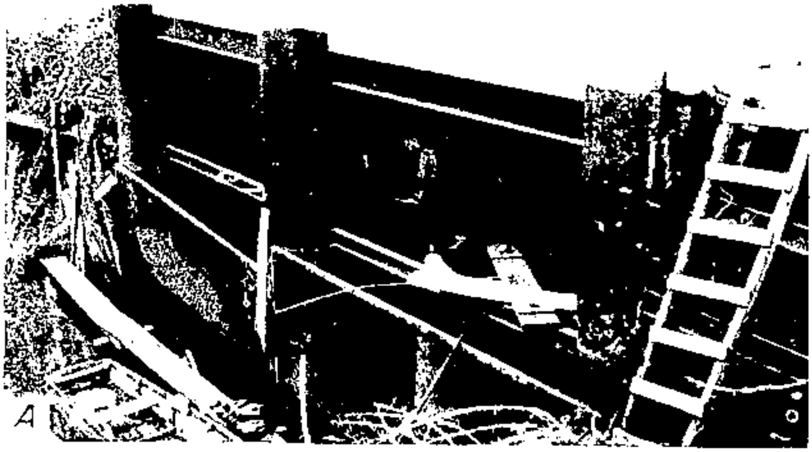


PLATE 7.—A, Simple brick pumphouse. Des Moines-Mississippi drainage and levee district. B, Metal pumphouse on reinforced concrete foundation carried above pumping floor; also well-designed trash screen. Lacey, Langellier, and West Matanzas drainage and levee district. C, Low-cost pumphouse of concrete blocks. McGee Creek drainage and levee district.



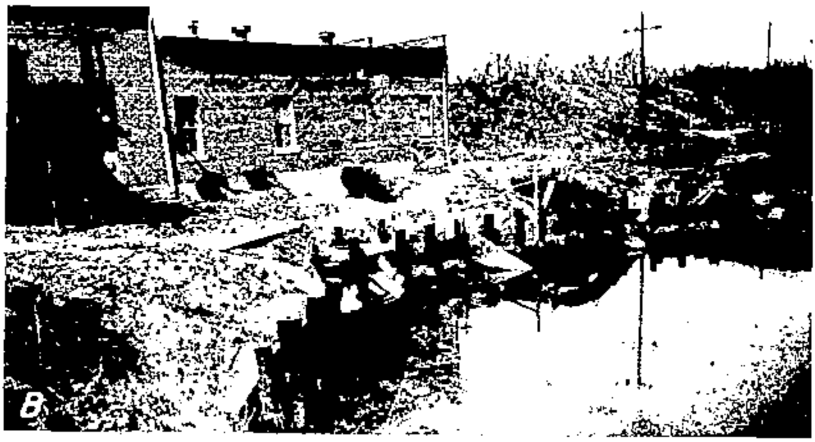


PLATE 8.—A, Concrete suction bay reinforced by concrete-filled caissons which support roof of pumphouse over the bay. [illegible] drainage and levee district. B, Inexpensive suction bay of wooden planks and piling to prevent caving of side of drainage ditch. Big Swan drainage and levee district. C, Trash screen upstream from suction bay supported by substantial wooden trestles. Hunt drainage and levee district.

tates 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

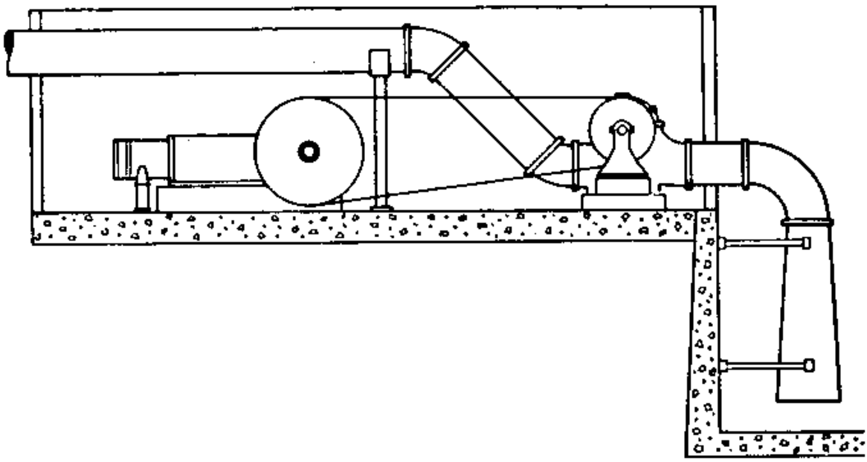


FIGURE 37.—Discharge pipe from centrifugal pump belt-driven by horizontal oil engine in McGee Creek plant is carried over the engine to reduce floor area required.

is desirable. For such plants a steel-frame building with stucco or sheet-metal walls is well suited (pl. 7, *B*). For 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, no pumping is usually 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 pumphouse 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.

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. 38). 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 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 be at least equal to the entrance diameter of the pipe. Between the back wall and the suction pipes a clearance of one-half to three-fourths the

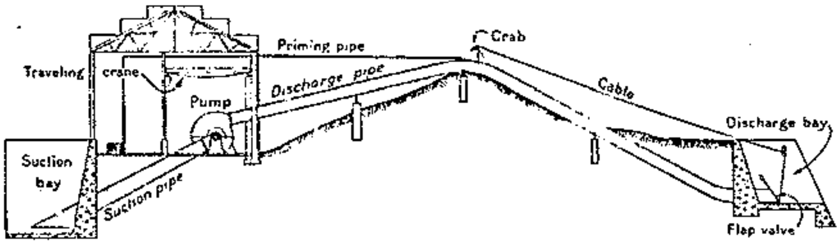


FIGURE 38.—Elevation of well-designed pumping unit in Hartwell plant: Reinforced suction and discharge bays; horizontal entrance opening to inclined suction pipe; discharge pipe passing over top of levee, with long-radius bends; crab for lifting flap gate out of flow from pipe.

entrance diameter seems adequate. The spacing of suction pipes at the entrance ends should be, on centers, not less than four or five 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 bottom of the caissons, 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 may cost more than 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 short 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 such 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*), although 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. This would have cost less than extending the pipe 5 to 10 feet farther from the levee. The gravity-sluice outlet structure was undermined. This 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 way 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 these and other levee failures. Had the sluiceways 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 runoff capacity of 0.25 inch per 24 hours over the watershed area is believed adequate for a gravity sluiceway built to supplement a pumping plant. When the runoff 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 (*17*).

An iron or steel gate should be constructed on the discharge side of the sluiceway, and should 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 18 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*), a cut-off wall, sheet piling, or riprap should be used. Here erosion occurred even though the average velocity through the culvert did not exceed 4 to 5 feet per second.

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 one-fourth 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 this area 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, *B*). 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 trash and debris.

FARM PUMPING PLANTS

Small drainage pumping plants are desirable under many conditions. A high farm income has encouraged drainage by all means, including use of pumps. An increasing number of small plants have been installed by landowners since 1945. Farm-drainage pumping plants often permit the drainage of isolated areas more cheaply than use of a tile line or open ditch. Pumps sometimes secure adequate drainage for land that is too low for gravity drainage. Small plants can often be used to advantage in large gravity drainage systems or to drain low areas in larger pumping districts. They can often be used in conjunction with irrigation systems to provide for adequate water control at all seasons.

Many of these small drainage pumping plants have been installed to drain fields ranging from 20 to 160 acres. Widespread installation of rural power lines has encouraged this development because electric operations of small plants has several advantages. One advantage is that automatic operation is simplified where electric power is available. Often these plants are limited in maximum size by the voltage available at the farm. A voltage of 220 single phase permits operation of 5- to 7-horsepower motors in several areas. Many small plants are operated by small gasoline or Diesel engines or by farm tractors. The rapid mechanization of farms has likewise encouraged farm-pumping installations.

Such installations have been characterized by efforts to save on first cost of the plant. Economy in first cost has been secured by use of inexpensive propeller-type pumps (p. 45), by simple buildings, inexpensive foundations and a minimum of accessory equipment. Small pumps and motors require only a light foundation. Often, a small concrete footing will provide for an adequate foundation on dry ground. In some cases steel I-beams have been laid across ditches on which to rest the equipment. Corrugated iron buildings with wood frames have been used for housing equipment. Pit-type plants have also been used to advantage.

By setting the discharge pipe above high water, as shown in figure 28, priming equipment is eliminated, use of automatic equipment is simplified, and the first cost of the plant is kept down. However, by setting the discharge pipe below low water, as shown in figure 29, the total head on the pump is reduced and the pump discharge is increased. Advantages of both methods of pump discharge need to be considered on the basis of the specific performance of the pumping unit.

One of the most important considerations in building such plants is to decide whether to provide the least expensive equipment and thus take additional risks on breakdowns and interruption in service. The kind and value of crops grown, the area of land flooded or damaged by high-water table in case of failure of the pumping plant, and the possibility of prompt repair or replacement of the pump and equipment should be considered. Small units can be replaced by other effective equipment more readily in an emergency than large pumping equipment. Hence, a risk often seems well justified in installing small plants.

Farm pumping plants require a higher runoff coefficient than larger drainage plants because storage is not usually available to the same degree as in large pumping districts. It is important to build an adequate drain to get the water to the plant rapidly enough for satisfactory pump operation. Usually an open drain adjoining the plant should be deepened and widened to provide temporary storage of water.

Examples of Small Plants

Several of the following plants have installed light propeller-type pumps to save on first cost. The discharge of many such units drops materially when the lift increases from 3 to 5 feet. Particular care is required in adapting such pumps to low-lift installations.

1. A light drainage pump, operated by a gasoline engine, is shown in plate 1, *A*. This unit is located in the Florida Everglades and

pumps from the drainage ditch on the right to the outlet to the left. A V-belt pulley mounted on the head drives the pump shaft. The propeller is set under water and no priming is required. A screen attached to the suction prevents trash from entering. The engine is set on skids and could easily be moved.

2. Plate 9, *A* shows a reversible pump with a propeller-type blade. having a capacity of 6,000 to 8,000 g. p. m. By turning one of two concentric cylinders, which form the pump barrel, 180°, the flow is reversed while the pump shaft continues to operate in the same direction. This unit is located in the Florida Everglades and is used for

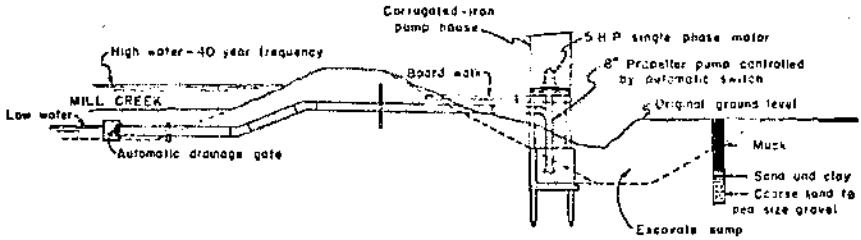
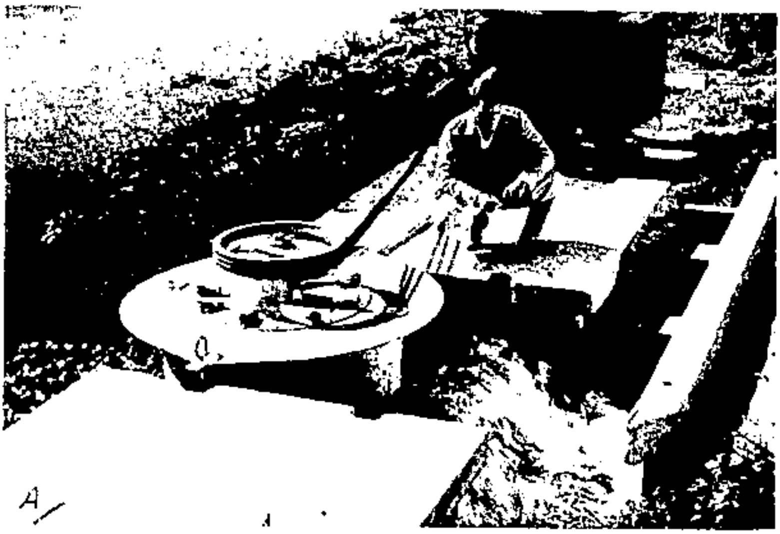


FIGURE 39.—Elevation of well-designed farm pumping plant, Central La Peer Soil Conservation District, Mich.

water control in the organic soils. After heavy rains it is used for drainage; when the water table needs to be higher it serves for an irrigation unit. The static lift seldom exceeds 4 feet.

3. Figure 39 shows the cross section lay-out of a well-designed pumping plant for drainage of 46 acres of muck land. The pump is a vertical propeller-type rated at 1,270 g. p. m. at 9-foot total head when operating at 1,760 r. p. m. The plant has a runoff capacity of 1.16 inches per 24 hours from a 58-acre watershed. The pump is driven by a 5-horsepower repulsion induction motor mounted on the head and connected directly to the pump shaft. A view of the plant is shown in plate 9, *B*. The Central La Peer Soil Conservation District furnished plans for this plant to the owner. Keith H. Beauchamp, drainage engineer, Soil Conservation Service, designed the plant.

4. Plate 9, *C* is a view of an electric-driven plant draining 155 acres, located in Lucas County, Ohio. The 7½-horsepower motor is connected by three V-belts to the vertical submerged impeller-type pump. The pump capacity was estimated for design purposes at 2,600 g. p. m. at a low head, equivalent to a runoff capacity of 0.89 inch per 24 hours. An approximate rating was made when the water in the suction bay was at low stage and the pump discharge was estimated to be only half the designed capacity. However, as the suction bay rises it is probable that the 2,600 g. p. m. capacity is reached, indicating a great variation in capacity with a small difference in head. The pump discharged into a concrete-lined flume which ran into the outlet drain (pl. 9, *C*). No advantage is taken of the vacuum which would be created by a discharge pipe discharging below the water surface in the outlet channel. However, no vacuum pump or vacuum-breaking device is required and automatic operation of the plant is simplified. Automatic operation is secured by means of a float-controlled switch.



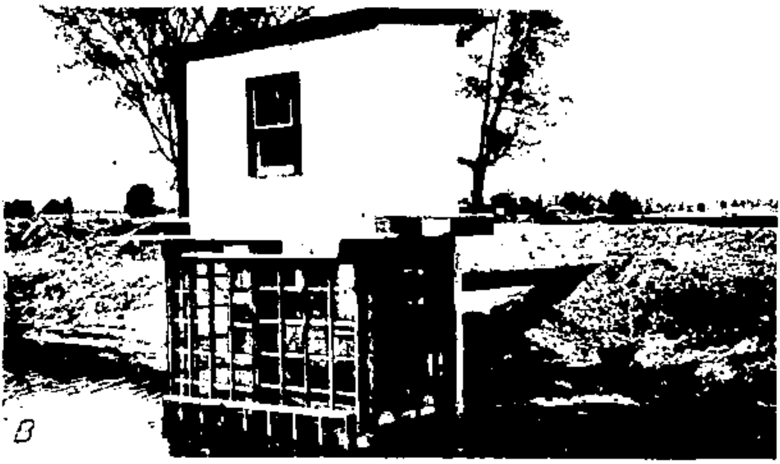


PLATE 9.—A, Reversible propeller pump used for drainage or irrigation by [illegible], Florida Everglades. B, Farm drainage pumping plant, owned by M. V[illegible] Des Berges, Imlay City, Mich. C, Farm drainage pumping plant owned [illegible] P. Wal[illegible], Lucas County, Ohio.

PLATE 10.—A, Farm drainage pumping plant owned by Vanarsky, Tuscola County, Mich. B, Farm drainage pump located in pit draining land in background by tile system, owned by I. Jones, Fulton County, Ohio. C, Farm drainage pump set on headwall across ditch draining L. Ainsworth farm, Tuscola County, Mich.

The soil, a silty clay, is drained by tile lines spaced at 50-foot intervals. A mile of open ditch serves as outlet for the tile lines.

The cost of the plant, which was completed in 1948, was $475 for pump house, foundation, and outlet flume. The owner did considerable work on the plant. The cost of the pump and motor was $394. The 7½-horsepower motor is a 220-volt single phase. The 12-inch pump is a light-submerged propeller-type (p. 45).

5. Plate 10*A* shows a smaller plant containing an 8-inch propeller-type pump (p. 45) located in Tuscola County, Mich. The pump is belt-connected to a gasoline engine. The frame shed and pump rest on I-beams across the ditch. This plant drains 120 acres.

6. Plate 10, *B* shows the top of a pump pit set underground which drains a low area of 25 acres through a tile located in Fulton County, Ohio. Before this old lake bed was pumped out it was wet; and about 5 acres could not be farmed. The pit contains a 2-inch sump pump, driven by a 2-horsepower motor. The cost of this pump and motor was about $300. The cost of concrete block sump pump was about $100 cash expenditure. This does not include the work of the owner who did most of the building. During the 11-month period, January to November 1948, the plant used 1,573 kilowatt-hours and the power cost was $53.70.

7. Plate 10, *C* shows an 8-inch locally manufactured propeller-type pump drawn by a 5-horsepower single-phase electric motor. An automatic float operates the motor switch. The pump cost $175 and the motor $175. The unit is supported on the concrete block head-wall over which the water is pumped to drain the farm lying upstream. This unit drains 120 acres.

CONSTRUCTION OF THE PUMPING PLANT

Competitive bidding for construction of large 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 bring competition from manufacturers and dealers in machinery, many of whom will not take contracts involving foundation and similar work.

A cofferdam of wood or steel piling is usually 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 is to excavate the sand 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 underfloor is nearly watertight, the cofferdam can be emptied without difficulty and the suction-bay floor constructed. The underfloor should not be considered a part of the suction-bay floor because a thin concrete slab laid under water may have little strength. The uplift pressure on the underfloor when the suction bay is dewatered should be estimated. Provision should be made to protect the structure against such upward pressures by means of weighting or bracing the floor. 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 whether 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 larger pumps because some manufacturers are not equipped to test them. Because the equipment required to test oil engines is heavy and expensive to ship, it may be 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 be within 10 minutes but never more than 15 minutes. The joints of the suction and discharge pipes should be inspected to be sure they are airtight; some 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, by, for instance, adjusting speeds and by repairing or giving better care to the equipment. Occasional tests of the plant are the best means of determining that the plant is operating at proper efficiency. In one case a test revealed a pumping unit was operating at low efficiency. Further checking for causes revealed a large block of wood in the pump impeller which had not been detected by the plant operator because it created no noticeable noise or vibration. After the block was removed the pump operated near its designed efficiency. The methods and equipment recommended for testing pumping plants have been described elsewhere (*11*). This information may also be secured on request from the Soil Conservation Service.

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 operators. Keeping the plant clean reduces the fire hazard and, in addition, improves its appearance.

Even though there is a well-maintained trash screen, sticks and trash often lodge in the pumps or pipes. This decreases the plant efficiency and may endanger the equipment. Although wood blocks 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 should be kept tight enough 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. When they are primed, 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. This 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 greatly reduces the efficiency of a pump. Occasional tests will show when the efficiency is abnormally low. Then inspection can be made to determine the character of repairs needed, as, for instance, the 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 46. 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. Some operators who are paid by the month do their work in the least possible time 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) kilowatt-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.

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APPENDIX

The pumping equipment available for land drainage on the Illinois River and on the Mississippi River between Dubuque and St. Louis, in 1930, is shown in table 9. The locations have been shown in figure 1.

The capacities and types of pumps and power equipment of many of the plants were 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, however, to any engineer in making a study of pumping plants in the area.

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

Table 10 (*11*) shows average annual operations of drainage pumping plants in the upper Mississippi Valley. Data were for 1930 and prior years. The quantity of water pumped varied considerably in accordance with conditions previously discussed. The average over-all efficiency for most plants ranged from 40 to 55 percent. The Scott County stream plant while pumping during high-water periods maintained an over-all efficiency of 63 percent. For any plant the over-all efficiency varies as the static lift, being higher as the static lift increases. This is because hydraulic pipe losses are a higher proportion of the total head when a pump operates at a low lift.

Table 11 (*8*) shows costs and conditions of operations in Louisiana pumping districts. The average rainfall of southern Louisiana is about 60 inches. With a few exceptions the annual rainfall measured at the districts shown in the table was well below the average for the area. Pumping dropped off materially in years of low rainfall. The abnormally low rainfall recorded in most years should be considered in making use of these records.

TABLE 9.—*Drainage pumping plants in the upper Mississippi Valley, 1930*

Key No.[1]	Drainage district	Water-shed area	Engine or motor: Type	Engine or motor: Began operating	Engine or motor: Rating	Pump: Began operating	Pump: Size and type[2]	Pump: Capacity (estimated)	Runoff depth per 24 hours[3]
	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	6,000	
3	Pekin-LaMarsh	3,600	do	1910	100	1910	26	13,500	.346
				1925	75	1025	20	10,000	
4	Rocky Ford	1,600	do	1914	100	1914	18	10,000	.805
				1919	100	1919	26	17,000	

[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.

TABLE 9.—*Drainage pumping plants in the upper Mississippi Valley, 1930*—Con.

Key No.[1]	Drainage district	Watershed area	Engine or motor: Type	Engine or motor: Began operating	Engine or motor: Rating	Pump: Began operating	Pump: Size and type[2]	Pump: Capacity (estimated)	Runoff depth per 24 hours[3]
	ILLINOIS RIVER DISTRICTS—continued	*Acres*		*Year*	*Horsepower*	*Year*	*Inches*	*Gal. per min.*	*Inches*
5	Spring Lake	22,500	Electric	1915	300	1909	48	52,000	.270
			do	1915	300	1909	48	52,000	
			Diesel	1926	120	1909	24	13,000	
6	Banner Special	6,760	Electric	1916	250	1916	30	30,250	.360
				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	Semi-Diesel	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	8,640	do	1930	200	1930	30M	29,000	.409
12	Langellier			1930	125	1930	30	20,000	
13	West Matanzas			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	.440
18	Coal Creek	7,525	do	1914	100	1914	20	12,500	.204
				1914	100	1914	20	12,500	
				1914	100	1914	20	12,500	
19	Crane Creek	6,230	Steam	1912	250	1912	30	30,000	.255
20	Big Prairie	2,200	Electric	1929	100	1914	30	18,000	.434
21	Beardstown Drainage and Sanitary.	800	do	1920	125	1920	24M	15,000	1.524
				1929	75	1929	20M	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	1910	100	1910	24	12,000	.190
24	Meredosia Lake	5,000	do	1929	200	1929	30	32,000	.488
				1929	100	1911	24	14,000	
25	Coon Run	1,987	Electric	1930	50	1930	20M	10,000	.207
26	Little Creek	2,000	Steam	1911	90	1911	24	16,000	.710
			Semi-Diesel	1923	80	1923	24	12,000	
27	McGee Creek	[illegible]	Steam	1915	225	1915	30	31,000	.401
			Semi-Diesel	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	.266
				1922	100	1922	24	10,750	
29	Mauvaisterre	7,150	Diesel	1926	120	1926	30	20,000	.152
30	Scott County	12,500	Steam	1913	420	1913	45	45,000	.362
			do	1913	155	1913	24	10,000	
			Semi-Diesel	1928	150	1928	30	22,000	
31	Big Swan	15,700	Electric	1916	400	1912	45	45,000	.207
				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	1929	260	1929	30M	41,000	
			Steam	1920	810	1920	60	100,000	
			Electric	1923	50	1923	15	5,000	
			do	1923	150	1923	30	22,000	
33	Hartwell	12,900	Electric	1915	150	1915	30	22,500	.554
				1915	150	1915	30	22,500	
				1915	150	1915	30	22,500	
				1920	458	1920	48M	67,000	
34	Keach (Fairbanks)	18,300	Steam	1910	350	1910	48	55,000	.543
				1927	225	1910	36	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	Semi-Diesel	1923	50	1923	18	8,000	.377
37	Nutwood	17,500	Steam	1910	500	1910	48	52,000	.354
			Semi-Diesel	1925	300	1910	36	30,000	
						1925	36	35,000	

[1] 3 districts drained by 1 pumping plant.

TABLE 9.—*Drainage pumping plants in the upper Mississippi Valley, 1930*—Con.

Key No.[1]	Drainage district	Watershed area	Engine or motor: Type	Engine or motor: Began operating	Engine or motor: Rating	Pump: Began operating	Pump: Size and type[2]	Pump: Capacity (estimated)	Runoff depth per 24 hours[3]
	MISSISSIPPI RIVER DISTRICTS	*Acres*		*Year*	*Horsepower*	*Year*	*Inches*	*Gal. per min.*	*Inches*
38	Carroll County No. 1	3,500	Semi-Diesel	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	Semi-Diesel	1924	100	1924	24	15,000	.300
				1924	200	1924	36	34,000	
41	Drury	7,000	Steam	1908	250	1908	50	70,000	.530
42	Union No. 1	[4]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
			Semi-Diesel	1915	50	1915	18	8,000	
			do	1920	55	1920	20	9,000	
45	Muscatine-Louisa No. 13	55,000	Steam	1910	400	1910	54	85,000	.240
			do	1910	400	1910	54	85,000	
			do	1910	150	1910	30	36,000	
			Electric	1920	200	1920	30	30,000	
			do	1920	100	1920	24	16,000	
46	Louisa-Des Moines No. 4	18,000	Steam	1909	315	1909	60	63,500	.421
				1909	315	1909	60	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	[4]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	Semi-Diesel	1920	100	1929	30	20,000	.482
				1920	125	1920	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	10,640	do	1928	250	1928	36	35,000	.355
				1928	250	1928	30 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	30,000	
58	Union Township	3,700	Semi-Diesel	1923	160	1923	30	22,000	.315
59	Fabius	14,000	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	
62	South River	11,200	Steam	1911	250	1911	30	32,000	.303
				1911	250	1911	30	32,000	
63	Riverland	6,400	Semi-Diesel	1921	125	1921	30	20,000	.331
			Diesel	1930	120	1921	30	20,000	
64	Elsberry	25,000	Steam	1915	300	1915	48	56,000	.238
				1915	300	1915	48	56,000	
65	Sandy Creek	1,125	Semi-Diesel	1923	38	1923	18	8,000	.377
66	Brevator	1,800	do	1920	60	1920	24	10,000	.295
67	East Side Levee and Sanitary:								
	Cahokia plant	2,088	Diesel	1930	90	1930	36 S	20,000	.789
				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	1930	54 S	50,000	

[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] 2 districts drained by 1 pumping plant.

TABLE 10.—*Average annual operations of drainage pumping plants*

Plant	Power type	Years of record: Period	Years of record: Number averaged	Rainfall	Water pumped: Run-off depth	Water pumped: Quantity	Average static lift	Rated horsepower hours operated	Current coal, or fuel oil used [1]	Input to motor or engine	Output of motor or engine	Power used per acre-foot		Average overall efficiency	Average load on motors
				Inches	*Inches*	*Acre-feet*	*Feet*			*Indicated horse-power-hours*	*Brake horse-power-hours*	*Indicated horse-power-hours*	*Brake horse-power-hours*	*Per-cent*	*Per-cent*
Banner Special	Electric	1925	1	27.61	8.98	5,061	10.38	180,100	106,900	143,360	128,790	28.3	25.4	50	71
Coal Creek	do	1914-21; 1923-30	16	35.11	23.94	15,010	12.14	624,226	463,289	621,273	559,755	41.4	37.3	40	90
Green Bay	do	1925	1	38.31	21.65	25,272	10.67	619,410	578,900	776,290	714,200	30.7	28.3	48	115
Hartwell	do	1929-30	2	35.80	13.75	14,778	11.65	530,065	345,825	463,750	428,360	31.4	29.0	51	81
Hunt	do	Apr. 18, 1928, through 1930.	2.70	37.17	16.29	21,498	8.53	743,486	456,580	612,281	561,220	28.5	26.1	41	75
Indian Grave	do	1925-28 and 1930	5	34.23	13.18	23,060	4.74	711,530	549,362	721,964	664,324	31.3	28.8	21	93
Lima Lake	do	1929-30	2	34.10	26.56	36,834	9.83	1,313,705	880,095	1,180,205	1,073,985	32.0	29.2	42	82
South Beardstown	do	1925-30	6	37.85	40.48	28,165	15.35	1,011,792	815,863	1,094,070	976,775	38.8	34.7	54	96
Henderson County	do	May 14, 1928, through 1930.	2.63	40.24	30.95	57,772	6.67	2,103,698	1,078,322	1,446,044	1,302,365	25.0	22.5	37	62
Total or average			38.33	35.60	21.75	25,272	9.45	870,890	586,126	784,360	712,197	31.0	28.2	42	82
Bay Island	Steam	1925-30	6	38.94	18.21	78,895	7.17	2,140,982	3,118.4	1,934,453	1,702,317	24.5	21.6	40	79
Crane Creek	do	Feb. 10, 1925, through 1930.	5.89	39.53	20.38	10,577	12.44	432,198	834.6	333,974	297,457	31.6	28.1	54	69
Henderson County	do	Mar. 1, 1926-May 13, 1928.	2.20	39.67	35.64	66,533	5.96	2,275,404	3,448.4	1,105,118	972,509	16.6	14.6	49	43
Total or average			14.09	39.38	24.74	52,002	7.01	1,616,195	2,467.1	1,124,515	990,761	21.6	19.1	45	61
Eldred	Stand-by steam	1925-30	6	------	3.25	2,520	10.64	127,573	398.0	91,225	74,977	36.2	29.8	40	59
McGee Creek	do	1928-30	3	------	.65	852	6.19	37,230	118.1	33,760	28,697	39.6	33.7	21	77
Scott County	do	1929-30	2	------	2.72	2,831	15.24	143,600	237.1	93,305	82,110	33.0	29.0	63	57
Total or average			11	------	2.21	2,068	12.13	102,801	251.1	72,763	61,928	35.2	29.9	47	69
Eldred	Diesel	1925-30	6	38.30	20.90	16,206	8.35	563,082	29,058	561,978	409,965	34.7	25.3	33	73
McGee Creek	Semi-Diesel	1928-30	3	35.71	13.22	17,298	9.63	631,167	45,990	798,900	599,180	46.2	34.6	29	95
Scott County	do	1929-30	2	31.57	11.09	11,553	13.65	472,935	34,370	580,795	435,610	50.3	37.7	37	92
Total or average			11	35.19	15.07	15,019	10.21	555,728	36,473	647,224	481,585	43.1	32.1	33	87
Fabius	Diesel	1925-28	4	39.71	10.24	12,715	4.83	321,270	23,546	383,550	268,423	30.2	21.1	22	84
Mauvaisterre	do	Apr. 10, 1927, through 1929.	2.73	41.29	7.43	4,444	6.10	133,595	7,779	150,887	113,148	34.0	25.5	25	85
Valley City	Semi-Diesel	Feb. 18-Dec. 31, 1925, and 1928-30.	3.87	34.48	13.64	8,140	11.07	249,568	16,813	287,764	224,452	35.4	27.6	43	90
Total or average [2]			6.60	37.88	10.54	6,292	9.31	191,582	12,293	219,326	168,800	34.9	26.8	37	88

[1] Kilowatt-hours of electric current, tons of coal, or gallons of fuel oil.

[2] Fabius plant data omitted from average because costs are not representative.

TABLE 11.—*Costs and conditions of operation of drainage pumping plants in Louisiana pumping districts*

District	Area	Year	Rainfall	Run-off	Average effective lift	Acre-feet of water lifted 1 foot	Cost							Number of days on which plant operated	Number of times boilers were fired up	Number of 24-hour days per year at full capacity	Hours at full capacity per fire-up	Longest period of continuous operation	Number of times plant was run continuously more than following number of days					
							Fuel	Labor	Repairs and incidentals	Total	Per acre	Per acre-foot of water lifted 1 foot: Total	Per acre-foot of water lifted 1 foot: Fuel						1	2	3	4	6	10
	Acres		*Inches*	*Inches*	*Feet*													*Days*						
Smithport Plantation	647	[1] 1911	57.52	33.35	5.2	9,400	$667	$351	$150	$1,168	$1.80	$0.125	$0.071	115	103	33.0	7.7	2.0	8	1	0	0	0	0
	647	[2] 1912	52.23	40.80	5.5	12,190	795	342	170	1,307	2.02	.108	.066	108	77	49.5	12.9	4.3	12	5	3	1	0	0
	647	1913	60.01	37.77	5.8	11,840	780	215	35	1,030	1.60	.087	.066	86	68	34.2	12.0	3.8	10	3	1	0	0	0
Willswood Plantation	2,400	[3] 1909	43.32	16.33	5.0	16,330	936	600	230	1,766	.73	.108	.057	45	39	20.0	7.3	2.5	3	1	0	0	0	0
	2,400	1910	43.08	11.58	5.0	11,580	616	500	180	1,296	.54	.112	.053	45	41	8.4	4.9	.75	0	0	0	0	0	0
	2,600	1911	52.32	23.42	5.0	25,290	1,267	753	300	2,320	.89	.092	.050	69	61	10.2	7.6	3.5	5	1	1	0	0	0
	2,600	[4] 1912	48.22	34.42	5.0	37,290	2,676	765	514	3,955	1.52	.106	.072	81	63	35.0	8.9	4.0	10	4	1	1	0	0
	2,600	1913	51.52	27.08	5.0	30,180	2,412	848	457	3,717	1.43	.123	.080	68	60	17.5	7.0	2.0	6	2	0	0	0	0
	2,600	1914	42.25	15.05	5.0	16,320	1,210	426	464	2,100	.81	.129	.074	41	39	9.7	6.0	2.2	2	1	0	0	0	0
	2,600	1915	5.05	34.24	5.0	37,100	3,030	751	922	4,703	1.80	.127	.082	63	49	24.5	12.0	4.77	8	3	3	2	0	0
Reclamation district No. 1 Poydras.	2,500	[5] 1912	70.52	28.28	2.1	12,420	1,098	640	244	1,982	.79	.160	.088	114	77	41.8	10.5	14.0	12	9	7	4	1	1
	2,500	1913	63.45	38.82	4.5	36,580	2,314	1,261	347	3,922	1.57	.107	.063	155	131	46.5	8.5	6.5	4	3	2	1	1	0
	2,500	1914	47.70	16.46	5.2	18,320	743	764	408	1,915	.77	.105	.041	73	71	17.8	6.0	2.0	2	1	0	0	0	0
	2,500	[6] 1915	36.14	16.21	5.2	17,572	755	582	180	1,517	.61	.086	.043	62	60	22.8	6.8	1.77	5	0	0	0	0	0
Lafourche drainage district No. 6, subdistrict No. 1.	1,880	1913	47.37	31.00	5.1	24,910	1,526	564	116	2,206	1.18	.089	.061	210	198	40.0	4.8	5.0	5	4	2	1	0	0
	1,880	1914	45.70	25.83	5.3	21,340	871	517	65	1,453	.77	.068	.041	178	159	35.3	5.3	4.8	8	5	2	1	0	0
	1,880	[6] 1915	28.41	21.94	5.4	18,745	1,160	331	25	1,516	.80	.081	.062	92	75	39.0	9.4	3.71	11	7	3	0	0	0
Lafourche drainage district No. 12, subdistrict No. 3.	2,250	1913	52.0[illegible]	20.84	2.9	11,270	930	306	331	1,567	.70	.140	.082	90	90	15.5	4.1	.75	0	0	0	0	0	0
	2,250	1914	58.95	23.40	3.5	14,870	1,140	233	194	1,567	.70	.106	.077	78	71	18.7	6.3	1.60	3	0	0	0	0	0
	2,250	[6] 1915	33.07	11.61	4.2	9,087	1,115	112	265	1,492	.66	.164	.123	41	39	13.5	6.2	1.42	2	0	0	0	0	0
Gueydon drainage district, subdistricts Nos. 1 and 2.	6,500	[7] 1912	41.71	37.26	3.6	75,200	2,640	1,225	795	4,660	.72	.062	.035	196	185	40.0	4.1	4.0	6	3	2	1	0	0
	7,500	[8] 1913	77.61	65.86	5.0	194,000	6,013	1,321	631	7,965	1.06	.041	.031	214	173	53.0	7.4	19.54	13	5	2	1	1	1
	6,500	[9] 1914	59.78	39.86	5.1	109,350	------	------	------	5,175	.79	.047	------	166	162	28.0	4.1	5.2	7	1	1	1	0	0
	6,500	[9] 1915	49.77	24.21	5.2	68,644	1,833	1,180	867	3,880	.60	.056	.027	78	77	19.4	6.0	2.79	6	2	0	0	0	0
New Orleans Lake Shore Land Co.	6,040	[10] 1914	50.60	20.46	4.5	46,760	[11] 3,932	------	------	5,650	------	.121	.084	308	308	16.0	1.2	.6	0	0	0	0	0	0
	6,943	[12] 1915	36.80	20.33	5.2	61,496	[11] 3,603	912	34	4,549	.65	.074	.059	172	------	21.7	------	2.15	3	2	0	0	0	0
Jefferson drainage district No. 3.	5,000	[13] 1913	49.60	28.42	5.1	59,970	2,565	692	480	3,737	.75	.062	.043	121	110	29.0	4.9	2.5	1	1	0	0	0	0
	5,000	[14] 1914	48.16	26.45	4.7	51,630	2,274	595	261	3,130	.63	.061	.044	104	97	18.3	4.5	2.0	5	1	0	0	0	0
	5,000	[12] 1915	44.61	18.18	5.1	38,578	1,600	700	350	2,650	.53	.069	.042	61	59	15.9	5.9	1.83	1	0	0	0	0	0

Lafourche drainage district No. 12, subdistrict No. 1.	835	[13] 1915	30.38	20.25	3.6	5,157	75	160	25	260	.31	.050	.014	41	40	15.9	9.0	1.0	1	0	0	0	0	0

[1] March 1911, to February 1912, inclusive.
[2] March to December, inclusive.
[3] June to December, inclusive.
[4] January to May and October to December, inclusive.
[5] Mar. 9 to Dec. 31.
[6] January to September, inclusive.
[7] Mar. 17 to Dec. 31. Area 5,600 to Aug. 23, 1912, then increased to 6,500.
[8] Area increased to 7,500 in July; decreased to 6,500 Dec. 31, 1913.
[9] Area 6,500 acres Dec. 1, 1915; 7,500 acres after Dec. 1, 1915.
[10] Average for year, electric plant.
[11] Electric power.
[12] February to December, inclusive.
[13] Mar. 12 to Dec. 31.
[14] January to September, inclusive.

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