



# Pumped Outlets for Drainage Systems

by A. T. Corey and M. N. Shearer<sup>1/</sup>

This report represents guidelines for the design of pumped outlets for drainage systems, especially for outlets under conditions prevalent in the Willamette Valley. The design includes several aspects; namely, sump capacity and construction, pump selection, power requirements and switching system. In addition, guidelines are presented for estimating the cost of operation.

## SUMP CAPACITY

### Pump Capacity

The storage capacity required for the sump depends upon the pump capacity, which is usually selected to approximate the "peak" tile flow rate expected. In selecting the pump or pumps, it is appropriate to use the same flow rate as is used to design the main at the outlet. To arrive at an economical design, this rate may be somewhat less than the absolute peak discharge expected, but it should be near the maximum expected over a significant period of time--say 24 hours. The latter is called the "design discharge."

The design discharge depends on the depth of soil, conductivity of the soil, and the spacing of the collector drains. For the shallow, "tight" soils in the Willamette Valley, e.g. Dayton, Amity and Concord, a drainage coefficient of about 0.25 inch/day is probably justified where the spacing is in the range of 40 to 60 feet. This corresponds to a design discharge of about 5 gallons per minute (gpm) per acre of land drained. To obtain this discharge in gpm/acre, multiply the drainage coefficient  $w$  in inches/day by 18.86.

In cases where deeper and more permeable soils are drained, or where closer spacing is justified, a larger design discharge is required. An approximate formula relating the drainage coefficient to the pertinent factors is

$$w = 8K \frac{D^2}{L^2}$$

where  $K$  is the soil conductivity in inches/day,  $D$  is the depth of the soil above the restrictive layer, and  $L$  is the spacing. Here  $D$  and  $L$  should be in the same units.

Note that cutting the spacing in half should increase the drainage coefficient by a factor of 4. This implies that a spacing of 20 feet in an Amity soil, for example, would require a  $w = 1$  inch/day. However, the amount of water would have to be supplied by rain, and rainfalls of this intensity and duration are infrequent in the Willamette Valley. Therefore, it is probably not

---

<sup>1/</sup>Professor of Agricultural Engineering and Extension Irrigation Specialist, Oregon State University.

economical to use drainage coefficients greater than say 0.5 in/day in this area. Consequently, design discharges ranging from 4 to 10 gpm per acre would seem to be justified.

It is not economical to over-estimate the capacity required, since it is possible for the pump to run continuously when necessary and since larger capacity pumps significantly increase both initial cost and power requirements.

### Cycling Time

Cycling time is defined as the period between successive starts of the pump. Small cycling times are not desirable since this will mean more starts during a given period and result in a shorter useful life for the equipment. When an electrical circuit is open or closed, arcing occurs at contact points. Also, a momentary surge of high voltage occurs before the motor reaches operating rpm. The arcing and surging greatly reduce the useful life of the equipment.

Consequently, it is customary to specify a maximum allowable number of starts per hour  $n$  when designing the sump system. An allowable  $n$  of 10 has been suggested in the ASAE yearbook for 1979-80 by the ASAE Pump Drainage Committee. The number of starts per hour is kept within allowable limits by providing sufficient storage capacity in the sump as explained below.

### Storage Capacity

The storage capacity  $S$  is defined as the change in volume of water in the sump during a pumping cycle.  $S$  is equal to the change in elevation  $h_c$  (of water in the sump) times the area of the water surface  $A_s$  in the sump, i.e.,

$$S = h_c A_s.$$

Note that a greater  $S$  can be obtained with a given  $h_c$  by using a shallow sump with a larger surface area than by using a deep sump with a smaller surface area.

It is desirable to minimize  $h_c$  for a given  $S$  since pump discharge is more uniform when the lift does not vary over a large range. Consequently if a cylindrical conduit such as a corrugated pipe is to be used for the sump storage, it is better to orient the storage conduit in a horizontal rather than a vertical position.

When all of the storage is in the sump the necessary storage capacity  $S$  may be calculated by a formula derived by C. F. Larson and E. R. Allred (1956). Their formula is

$$S = \frac{2P}{n}$$

where  $S$  is given in  $\text{ft}^3$  and  $P$  is the pumping rate in gpm. The reasoning behind this formula is presented in the reference cited. Briefly, it is based on the

fact that the shortest cycling time will occur when the tile discharge rate is equal to one-half the pumping rate. Sufficient storage must be provided to take care of this worst condition since it is inevitable that the tile discharge will slow down to this rate sometime after each peak discharge is reached.

However, most of the time the cycling period will be longer. For example, when the tile discharge rate equals or exceeds P, the pump will run continuously and when the tile discharge stops altogether the pump will not run at all. The greatest frequency of starts occurs when the tile discharge is half-way between these two conditions, that is, when the inflow rate equals P/2. Note that over-designing the pump capacity also increases the sump storage requirement, or saying this in another way, it increases the frequency of starts for a given sump storage.

When storage in the main is provided by permitting submergence of the outlet, the greatest frequency of starts occurs when the inflow rate is greater than P/2--in some cases when the inflow rate almost equals P.

### Sump Construction

A drainage sump may be a pit, tank, section of pipe, ditch or low area serving as a collection point from which the drain water is pumped to a waterway. Under conditions existing in the Willamette Valley, a closed section of pipe will usually be preferable to an open ditch or pit. The latter might collect rain or surface runoff, thus reducing the effective storage. Furthermore, open ditches are a hazard for stock and an obstacle for farm machinery.

A design suggested by the ASAE Pump Drainage Committee is shown in Figure 1. Note that the pump intake is in a short section of 3½ foot diameter conduit in a vertical position. This is to provide a suitable submergence and clearance for the pump intake so that the pump can operate efficiently. The minimum submergence recommended for vortex suppression is given approximately by

$$\text{Sub.} = \frac{P}{705 D^2}$$

where P is the pumping rate in gpm and Sub. is the submergence in feet and D is the diameter of the suction bell in feet.

Note that the main portion of the sump storage is in the horizontal section of corrugated pipe. Note also, that the design shown in Figure 1 is entirely closed so that the collection of surface runoff and interference with farm operations is minimized.

In the Willamette Valley, however, it should frequently be possible to use the main drain for most of the storage so that the horizontal section of corrugated steel pipe can be eliminated. This is because drainage coefficients appropriate for this area are relatively small.

Using the main drain for sump storage should be feasible where the main is slightly below the elevation of the laterals, which is common practice in this area. A greater storage for a given  $h_c$  is obtained with the mains on

smaller slopes, although it is not recommended that slopes much smaller than 0.05 percent be used. In some cases it may be feasible to provide a main somewhat larger than needed for the design discharge in order to provide the necessary storage.

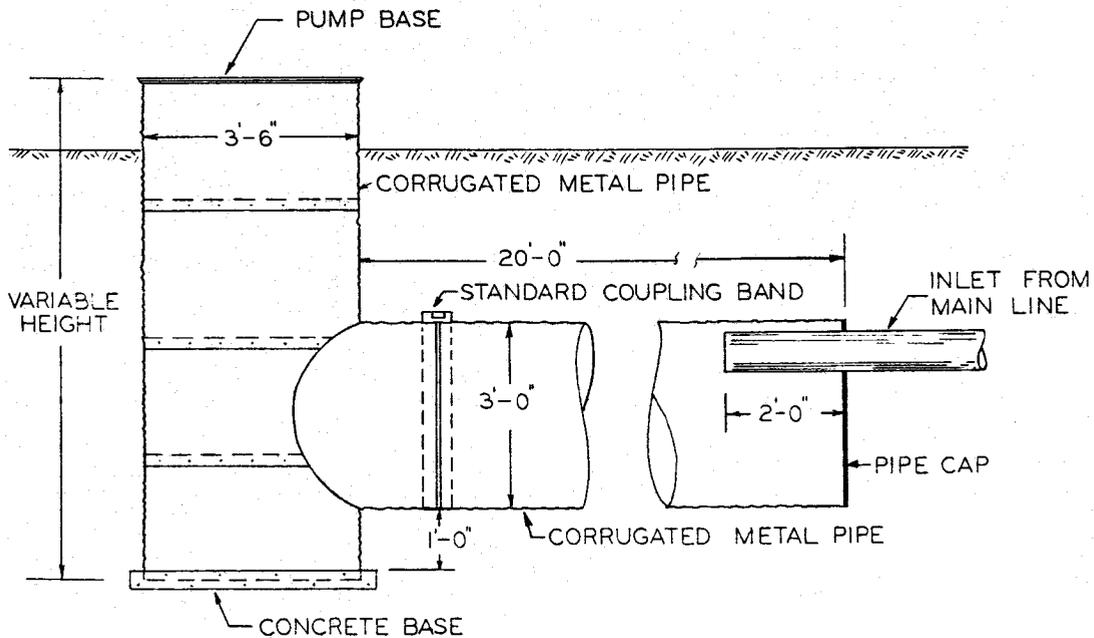


Figure 1. Suggested sump design adapted from ASAE standards.

### Calculation of Sump Storage Taking into Account the Main Storage

Storage available in a main is the volume of the main which is not needed to carry the main discharge but which can be filled with water by an appropriate submergence of the outlet. In calculating this storage it is necessary to consider:

- (1) the volume of that portion of the mainline which will be filled with water when the sump level is at its highest elevation,
- (2) the fraction of the mainline volume that is necessary to carry the discharge from the laterals and which cannot be drained when the sump level is at its lowest elevation.

Until water in the sump has reached a level above the top of the main at the outlet, the main will not be completely filled unless the discharge equals the maximum capacity, for a main of the given size and slope. As the top of the outlet is submerged, a portion of the main will become completely filled regardless of the discharge. This portion is called the submerged length  $L$  and is calculated by

$$L = \frac{100 d}{\text{percent slope}} \quad \text{or} \quad d = \frac{L \times \text{percent slope}}{100}$$

where  $d$  is the depth of submergence of the top of the outlet. Both  $L$  and  $d$  are given in feet. Of course,  $L$  cannot be greater than the existing length of the main.

In addition to the fully submerged length, another portion of the main will be partially submerged, if the existing main is longer than the calculated  $L$ . The latter portion may not be significant, however, since a fraction of the main volume is needed to carry the discharge and cannot be emptied in any case. Consequently, the partially submerged length is not considered in the calculations described below.

As the water level at the sump is dropped below the bottom of the main outlet by pumping, a volume of water is removed from the main. This volume represents the storage available from the main. The magnitude of this storage is not a constant but depends upon the discharge from the laterals. For example, if the discharge equals the maximum main capacity  $C$ , the storage is zero, whereas the storage approaches the full volume of the submerged length of main as the discharge approaches zero.

It is necessary to have at least a minimum storage in a sump at the outlet in order to provide the recommended submergence and clearance for the pump intake. The sump storage  $S_s$  does not depend on the discharge  $I$  in the main. Of course, if  $S_s$  is large enough, it is not necessary to depend upon the main for storage. However, it will often be economical to consider the main storage to reduce the size and cost of sump construction.

A formula for calculating the necessary sump storage taking into account the main storage is

$$S_s = \frac{8I_c}{n} \left( \frac{P-I_c}{P} \right) - L A \left( \frac{C-I_c}{C} \right).$$

In this formula, the symbols have the following meanings:

$S_s$  = Sump storage in  $\text{ft}^3$ .

$I_c$  = A critical main inflow in gpm that would cause the greatest frequency of pump starts,

$n$  = The allowable number of pump starts per hour,

$P$  = The pump capacity in gpm - also the maximum sustained main discharge to be expected,

$L$  = The submerged length of main,

$A$  = The cross-sectional area of the main,

$C$  = The maximum discharge capacity of the main.

This formula is derived using two simplifying approximations:

- (1) The fraction of the main volume needed to carry the discharge is  $I/C$ , which is a conservative assumption.
- (2) Storage in a partially submerged portion of the main is neglected. This also is a conservative assumption.

To use the formula for sump size, accounting for main storage, it is necessary to first find the critical drain inflow  $I_c$ . This is calculated by the formula

$$I_c = \frac{nP}{16} \left( \frac{8}{n} + \frac{LA}{C} \right).$$

Note that when  $L = 0$ ,  $I_c = P/2$ . For this case,

$$S_s = \frac{2P}{n},$$

which is the formula of Larsen and Allred (1956).

It may sometimes be desirable to calculate the length  $L_m$  of submerged drain sufficient to completely eliminate the need for sump storage. In this case the sump would be needed only to provide the recommended clearance and submergence for the pump intake. In the case for which the main capacity is equal only to the pumping rate, i.e.,  $C = P$ , the calculation for  $L_m$  is given by

$$L_m = \frac{8P}{nA}.$$

#### Storage in Minimum Sumps

In order to provide the recommended pump intake submergence, side clearance, bottom clearance, vortex suppression, and also to permit the water level to rise above the top of the outlet and drop to below the outlet, a minimum storage results. The volume of this storage depends on the diameter of the main at the outlet, because this determines the change in elevation necessary to submerge and clear the outlet.

The ASAE Yearbook (1979-80) recommends that if a vertical section of corrugated metal pipe is used for this purpose it should have a minimum diameter of 3.5 feet ( $A_s = 9.62 \text{ ft}^2$ ). Consequently, the minimum storages for sumps with the most common main sizes would be

- 8-inch - - - - -  $0.667 \times 9.62 = 6.4 \text{ ft}^3$
- 10-inch - - - - -  $0.833 \times 9.62 = 8.0 \text{ ft}^3$
- 12-inch - - - - -  $1.0 \times 9.62 = 9.6 \text{ ft}^3$

Any submergence of the top of the outlet will, of course, add to these minimum storages.

#### Example Problems

- A. Dayton Soil - Area = 50 acres
  - lateral spacing = 50 ft.
  - estimated  $w = 0.25$  inches/day
  - main slope - 0.1 percent
  - main sizes: 200 feet--disposal - 10-inch
    - 50 feet--main - 10-inch
    - 150 feet--main - 8-inch
    - remainder - main - 6-inch

Design Calculations

1. Maximum inflow rate to disposal line

$$I_m = 0.25 \times 18.86 = 236 \text{ gpm}$$

The factor 18.86 accounts for, ft<sup>2</sup>/acre, gallons/ft<sup>3</sup>. inches/ft and minutes/day.

2. Pump Capacity

A pump is chosen that has a capacity close to 236 gpm. Assume that one can be found that is rated at 240 gpm. Thus, P = 240.

3. Main Capacity

A 10-inch main or a 0.1 percent slope is calculated using Manning's equation with n = 0.015. To obtain the capacity in gpm this equation takes the form

$$C = 13747 D^{2.67} S^{.5} \text{ or}$$

$$C = 13747 (.833)^{2.67} (.001)^{.5} = \underline{267} \text{ gpm}$$

This value is not much greater than P; thus for purposes of calculation, we assume

$$C = P = 240 \text{ gpm}$$

4. Necessary Sump Storage

Since 250 feet of 10-inch main is available, we tentatively consider submerging only this portion of the main. In this case, the required sump capacity is found from

$$I_c = \frac{10(240)}{16} \left[ \frac{8}{10} + \frac{250(.54)}{240} \right] = 240 \text{ gpm}$$

$$S_s = \frac{8(204)}{10} \left[ \frac{240-204}{240} \right] - 250(.54) \left[ \frac{240-204}{240} \right] = 4.3 \text{ ft}^3$$

We see that the required storage is less than that provided by a minimum sump for a 10-inch main.

5. Necessary Submergence of Outlet

The water level above the outlet to submerge 250 feet of main is given by

$$d = \frac{250 \times 0.1}{100} = 0.25 \text{ ft.}$$

6. Total Required Length of Corrugated Pipe

(a) Intake submergence -

$$\frac{P}{705 D^2} = \frac{240}{705 (.33)^2} = \underline{3.2} \text{ ft.}$$

assuming an intake diameter of 4 inches (.33 ft)

(b) Bottom clearance -

$$\frac{D}{3} = \frac{0.33}{3} = 0.11, \text{ say } \underline{0.2} \text{ ft.}$$

(c) Water level change -

$$h_c = d + \text{diameter of main} = .25 + .83 = \underline{1.1} \text{ ft.}$$

(d) Freeboard for pump support - we must ensure that the pump motor remains dry.

Allow 1 ft. for freeboard.

(e) Required length of corrugated pipe -

$$L_p = 3.2 + 0.2 - 1.1 + 1.0 = \underline{5.5} \text{ ft.}$$

7. Alternative Design

Since the minimum sump supplies more than the required storage, we consider the consequence of submerging only the first 200 feet of main into which no laterals empty. In this case:

$$I_c = \frac{10(240)}{16} \left[ \frac{8}{10} + \frac{200(.54)}{240} \right] = 187.5$$

$$S_s = \frac{8(187.5)}{10} \left[ \frac{240-187.5}{240} \right] - 200(.54) \left[ \frac{240-187.5}{240} \right]$$

$$= \underline{9.2} \text{ ft}^3$$

$$= \frac{200 \times .1}{100} = 0.2 \text{ ft.}$$

$$h_c = 0.2 + 0.83 = 1.03$$

The minimum sump storage  $S_m$  is

$$1.03 \times 9.62 = \underline{9.9} \text{ ft.}^3$$

We see that it is necessary to submerge only that portion of the disposal line which is free of laterals, but the factor of safety is smaller than if the entire 250 feet is submerged.

B. Amity Soil (fairly good natural drainage)

Area = 40 acres

spacing = 60 ft.

estimated w = 0.33 inches/day

main slope = 0.1 percent

length disposal line - 150 ft (no laterals)

The laterals do not drop into the main; consequently we would prefer to completely submerge only the disposal line.

Design Calculations

1. Maximum Inflow Rate to Disposal Line

$$I_m = 0.33 \times 40 \times 18.86 = \underline{249} \text{ gpm}$$

2. Pump Capacity

$$P = 250 \text{ gpm}$$

3. Main Capacity

$$C = 13747 (.833)^{2.67} (.001)^{.5} = \underline{267} \text{ gpm}$$

A 10-inch disposal line is adequate.

4. Required Sump Storage

$$I_c = \frac{10(250)}{16} \left[ .8 + \frac{150(.54)}{267} \right] = 172 \text{ gpm}$$

$$S_s = \frac{8(172)}{10} \left[ \frac{250-172}{250} \right] - 150(.54) \left[ \frac{267-172}{267} \right] = \underline{14.1} \text{ ft.}^3$$

We see that the required  $S_s$  is greater than that provided by a minimum sump.

5. Alternatives

(a) Enlarge sump

(b) Enlarge mainline

We consider consequence of installing a 72-inch disposal line.

6. Required Sump Storage

$$C = 13747(1)(.001)^{.5} = 435 \text{ gpm}$$

$$I_c = \frac{10(250)}{10} \left( \frac{8}{10} + \frac{150(.78)}{435} \right) = 167 \text{ gpm.}$$

$$S_s = \frac{8(167)}{10} \left( \frac{250-167}{250} \right) - 150(.78) \left( \frac{435-167}{435} \right)$$

$$S_s = 44.4 - 72.1 = \underline{27.7} \text{ ft.}^3$$

The negative value of  $S_s$  implies that the 12-inch main of length 150 feet provides far more than the required storage capacity with zero sump capacity.

C. 100 Acres

$$w = 0.25$$

Disposal line = 150 ft (no laterals)

Slope = 0.1 percent

#### Design Calculation

$$I_m = 0.25 \times 100 \times 18.86 = 471.5 \text{ gpm}$$

P = 475 gpm - needs 14-inch tile

$$C = 13747 (1.1667)^{2.667} (.001)^{.5} = 656 \text{ gpm}$$

$$I_c = \frac{10 \times 475}{16} \left( \frac{8}{10} + \frac{150(1.07)}{656} \right) = 310 \text{ gpm}$$

$$S_s = \frac{8(310)}{10} \left( \frac{475-310}{475} \right) - 150(1.07) \left( \frac{656-310}{656} \right) = 1.5 \text{ ft.}^3$$

A minimum sump will provide the necessary storage plus an adequate factor of safety.

#### Alternative Design

Since in this case, the disposal line is relatively short, we might consider the consequence of doubling the slope to reduce the size required. Of course, we will check to see that this does not increase the depth of excavation near the outlet to the extent that it cannot be installed with the usual trenching equipment. Assuming that this will create no particular problem, we check the consequences.

First we note that the pumping head will be increased by 0.15 feet, which is of little consequence.

#### Design Calculations

$$P = 475 \text{ gpm (same)}$$

$$C = 13747 (1.0)(.002)^{.5} = 614 \text{ gpm for 12-inch tile}$$

$$I_c = \frac{10(475)}{16} \left( \frac{8}{10} + \frac{150(.78)}{614} \right) = 294 \text{ gpm}$$

$$S_s = \frac{8(294)}{10} \left( \frac{475-294}{475} \right) - 150(.78) \left( \frac{614-294}{614} \right) = 29 \text{ ft.}^3$$

S = 29 ft.<sup>3</sup> is too much for minimum sump

Try slope of 0.4 percent

$$C = 13747 (1.0)(.004)^{.5} = 869 \text{ gpm}$$

$$I_c = \frac{10(475)}{16} (.8 + \frac{150(.78)}{869}) = 277 \text{ gpm}$$

$$S_s = \frac{8(277)}{10} (\frac{475-277}{475}) - 150(.78)(\frac{869-277}{869}) = 12.7 \text{ ft.}^3$$

We see that by allowing an  $h_c$  given by

$$h_c = \frac{12.7}{9.62} = 13.2 \text{ ft.}$$

we can provide the necessary  $S_s$ . The necessary submergence of the main outlet will be

$$d = \frac{250 \times 0.4}{100} = 1.0 \text{ ft.}$$

Consequently, the  $h_c$  necessary to clear the bottom of the outlet will be 2.0 ft. This will provide plenty of storage but it also will increase the pumping head variation.

### PUMPING SYSTEMS

#### Pump Type

Traditionally axial flow or propeller pumps shown in Figure 2 are considered to be especially desirable as drainage pumps because of their ability to move large quantities of water (1000 to 25,000 gallons per minute) against low heads at high efficiencies (60 to 75 percent). Pumping water from tile drain systems in Western Oregon, however, presents a different problem. Estimated flow rates for pumping installations may vary from 100 to perhaps 1,000 gallons per minute -- with total dynamic heads of less than 10 feet.

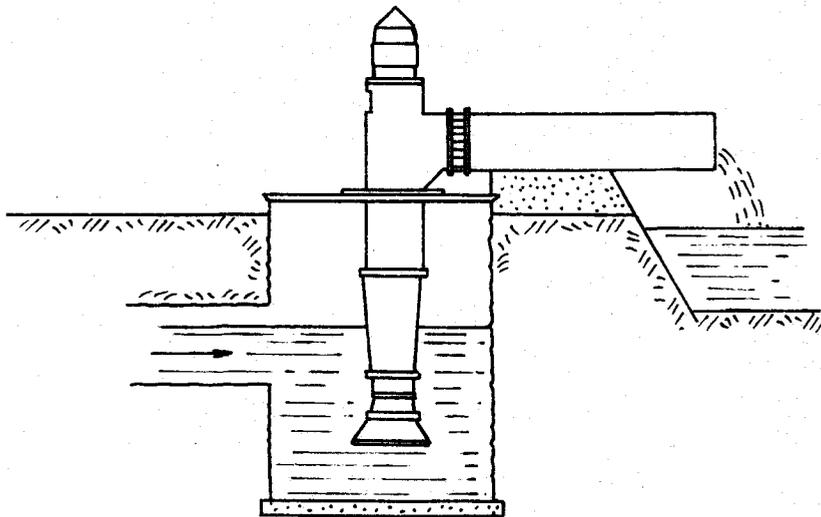


Figure 2. Typical propeller pump installation

Sump or de-watering pumps shown in Figure 3 are better adapted to this load even though their efficiencies (30 to 50 percent) may be considerably lower than efficiencies of propeller pumps. They are available from nearly all pump suppliers as complete units in capacities desired and require considerable less investment. When areas larger than 40 acres are drained through a common main drain, larger pumps may be selected or more than one pump may be placed in the same or in an adjoining sump. Care must be taken to ensure adequate clearances around pumps. (See Sump Design Discussion). Some dewatering pumps are made to operate dry with no damage to the pump.

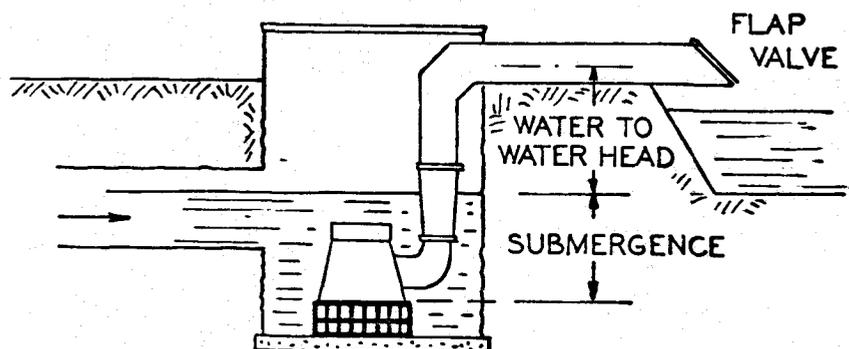


Figure 3. Typical dewatering or sump pump installation discharging above grade into ditch.

When it is difficult to find a pump which matches the exact pumping requirement, it may be more satisfactory to use two or more pumps. Multiple pumps cost more than a single pump of the same capacity; however, it may be more convenient and easier to meet the flow and head requirements. If two pumps are used, their heads may be calculated separately and the T.D.H. should match as closely as possible. Flow rates need not be equal. Separate discharge lines into the ditch should be used.

Multiple pumps provide greater flexibility in flow rates and can keep cycling to a minimum. In addition a number of small, single phase motors can be started one at a time with minimum load effects on electrical supply lines. Total pumping capacity of single or multiple pumps should equal estimated maximum tile discharge rates.

There is considerable difference in quality and cost of sump pumps. In Western Oregon, pumps will operate from 2500 to 3000 hours annually. With this operational requirement it will usually be most economical in the long run to purchase pumps with high quality long life bearings and seals even though their initial costs may be higher. During the dry season pumps should be removed from the sumps, serviced, and stored in a dry place.

#### T.D.H. Determination

Total dynamic head for drainage pumps are low, therefore, all heads in the

system must be considered, i.e.

$$\text{T.D.H.} = H_e + H_v + H_s + H_f + H_d$$

Where:

T.D.H. = Total dynamic head

$H_e$  = Entrance loss at the pump

$H_v$  = Velocity head in the discharge pipe

$H_s$  = Vertical distance between water surface in the sump and water surface in the discharge drain or pipe centerline.

$H_f$  = Friction loss through the discharge pipe, angles, and flap gate if used on the pipe discharge.

$H_d$  = Discharge loss as the water leaves the pipe and enters the drain ditch.

Pump suppliers will make this calculation; however, the purchaser must supply the flow rate desired and the elevation heads ( $H_s$ ) for both the high and low float settings. All of the losses except the  $H_s$  are affected by velocity of the water. It is desirable because of these effects to reduce velocities to about 5 feet per second as soon as possible. This can be done as the water leaves the pump and enters the discharge pipe by using a 10 to 12 degree conical increaser. This can reduce the total dynamic head from 10 to 35 percent. A good sump supplier can estimate the amount of savings more precisely for you.

If the pump discharges water into the atmosphere and drops it into a ditch as shown in Figure 3, the elevation head is calculated between the center line of the discharge pipe and the water surface in the sump.

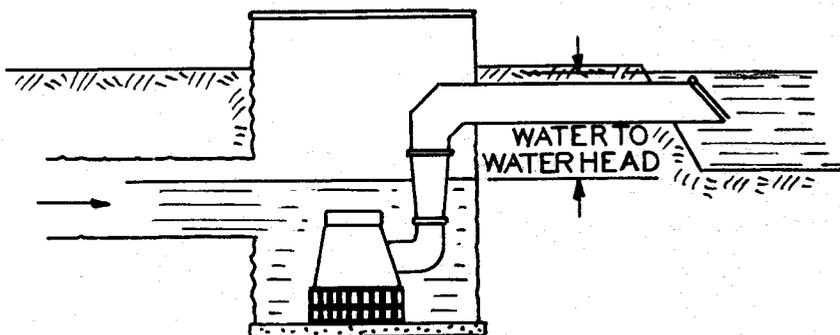


Figure 4. Typical dewatering or sump pump installation discharging below grade into ditch.

If pump discharge is submerged as shown in Figure 4, the elevation head is calculated between the water surface in the ditch and the water surface in the sump. The saving in pumping head by using submerged discharge can amount to 20 percent of the total dynamic head so that it is worth planning for.

## Power Requirements

Sump pumps and electric motors are usually sold as units so there should be no motor overload problem under very low head conditions. Electric motor size required is determined by the following formula:

$$\text{W.H.P.} = \frac{Q \times \text{T.D.H.}}{3960 \times \text{Eff.}}$$

Where:

W.H.P. = water horsepower, the rated horsepower of the motor  
 Q = flow rate in gallons per minute  
 T.D.H. = total dynamic head in feet  
 3960 = a conversion constant  
 Eff. = efficiency of the pump at operating conditions

Example No. 1: Forty acres of a tile drained field discharges a maximum of 7 gallons per minute per acre. The maximum water surface in the drain ditch is at ground surface level. The maximum total dynamic head is 7.5 feet. The pump operates at 35 percent efficiency. What size electric motor is required?

$$\begin{aligned} \text{W.H.P.} &= \frac{Q \times \text{T.D.H.}}{3960 \times .35} \\ &= \frac{7 \times 40 \times 7.5}{3960 \times .35} \\ &= 1.5 \text{ horsepower} \end{aligned}$$

This size electric motor can frequently be hooked directly into a farm electric system if it is not too far from the meter. The maximum reasonable distance is about 400 feet; however, it can be farther if larger wire is used. The advantage of using an existing power delivery is that line extension charges, if any, are eliminated and it is not necessary to have the power disconnected and turned on each year to avoid the monthly minimum charges during the non-pumping season.

Each situation must be considered individually. Advisory assistance is available from power supplier representatives and pump suppliers.

## Pumping Cost

During normal seasons, if the pump is sized as suggested, it will operate from 2,500 to 3,000 hours per year. The cost per kilowatt hour will be influenced by power line extension policies of the electrical utility and rate schedules.

Assuming a rate, for calculation purposes only, of 1 cent per kilowatt hour, annual pumping cost is determined as follows:

$$\text{Annual power cost} = \frac{\text{WHP} \times \text{hrs} \times .01 \times .746}{\text{Eff.}}$$

Where: WHP = Water horsepower, the energy required to drive the pump  
Hrs. = Annual hours of operation  
.01 = Assumed average cost per kilowatt hr. for electricity  
.746 = Converts hp to KW hr.  
Eff. = Electric motor efficiency

Example: A 1.5 hp motor is required for 40 acres. Pump operates 2500 hrs. per year. Motor Eff. = .78, power cost- .01 per KWhr.

$$\begin{aligned} \text{Annual cost} &= \frac{\text{WHP} \times \text{hrs} \times .01 \times .746}{\text{Eff.}} \\ &= \frac{1.5 \times 2500 \times .01 \times .746}{.78} \\ &= \$35.86 \text{ per 40 acres, or } \$0.90 \text{ per acre.} \end{aligned}$$

If the average cost of power is \$0.03 per KWhr instead of \$0.01 it will be three times this amount or \$2.69 per acre. If you can hook up to your existing meter, you can determine from your current monthly bills your average power cost per KWhr.

### Switching System

There are a number of switching techniques available to control maximum and minimum water levels in the sump. Probably the most common is the float arrangement. It is best to discuss this with the pump supplier as some pumps come with the switching mechanism attached and others are purchased separately. It is important that electrical codes be followed explicitly. All controls must be functional during a wet period in a wet situation.

### REFERENCES

1. ASAE Pump Drainage Committee (1979). Design of Agricultural Drainage Pumping Plants. Agricultural Engineers' Yearbook (1979-80), pp. 464-469.
2. Larson, C.L. and Allred, E.R. (1956). Planning pump drainage outlets. Agricultural Engineering, The Journal of the ASAE, Jan. 1956, pp. 38-40.
3. Larson, C.L. and Mnabeck, D.M. (1961). Factors in drainage pumping efficiency. Agricultural Engineering, The Journal of the ASAE, June 1961. pp. 296-297.
4. Larson, C.L. and Manbeck, D.M. (1962). Tile flow and power use data for pump drainage systems. Transactions of the ASAE, Vol. 5, No. 2, 1962, pp. 207-209.