CHAPTER 14

PUMP STATIONS

CHAPTER 14 – TABLE OF CONTENTS

14.1 OVERVIEW

14.1.1 Introduction

Stormwater pump stations are necessary to remove stormwater from highway sections that cannot be drained by gravity. Because of high costs and the potential problems associated with pump stations, their use is recommended only where other systems are not feasible. When operation and maintenance costs are capitalized, a considerable expenditure can often be justified for a gravity system. Alternatives to pump stations include deep tunnels, siphons and recharge basins, although recharge basins are often aesthetically unpleasing and can create maintenance problems. General guidance and information on all aspects of pump stations can be found in HEC 24 *(4).*

14.1.2 Policy

The following policies are specific to pump station design:

- Pump stations shall only be used where a gravity system is not practical or feasible.
- The design frequency for pump stations shall be the same as the frequency for the roadway system being drained.
- Pump stations must be designed to satisfy the spread criteria limitations of the roadway section being drained.
- Pump stations shall be designed in accordance with the Hydraulic Institute criteria and the guidance provided in this Chapter. See References *(1)*, *(2)*, *(3)*.

14.1.3 Design Considerations

Pump station design presents the designer with a challenge to provide a cost-effective drainage system that meets the needs of the project. There are a myriad of considerations involved in their design. Below is a listing of some of them:

- wet-pit vs. dry-pit,
- type of pumps,
- number and capacity of pumps,
- motor vs. engine drive,
- peak flow vs. storage,
- force main vs. gravity,
- above vs. below grade,
- monitoring systems,
- backup systems, and
- maintenance requirements.

Many of the decisions regarding the above are currently based on engineering judgment and experience. To assure cost-effectiveness, the designer should assess each choice and develop economic comparisons of alternatives on the basis of annual cost. However, some general recommendations can be made that will help minimize the design effort and the cost of these expensive drainage facilities. These are discussed in this Chapter.

For further information on the design and use of pump stations, see HEC 24 *(4)* and the other references at the end of this Chapter. The Hydraulic Institute, 9 Sylvan Way, Parsippany, New Jersey, 07054-3802 has developed standards for pumps *(1)*, *(2)*, *(3)*. Pump station design should be consistent with these standards.

14.2 SYMBOLS AND DEFINITIONS

To provide consistency within this Chapter and throughout this *Manual*, the following symbols will be used. These symbols were selected because of their wide use in the technical publications.

TABLE 14-1 — Symbols and Definitions

Symbol	Definition	Units
A	Minimum distance from back wall to trash rack	ft
B	Maximum distance between a pump and the backwall	ft
C	Average distance from floor to pump intake	ft
D	Pump diameter	in
DHW	Design high-water elevation	ft
H_f	Friction head	ft
H_l	Losses through fittings, valves	ft
H_s	Maximum static head	ft
H_t	Storage depth	ft
H_v	Velocity head	ft
H_x	Depth for storage volume	ft
INV	Inlet invert elevation	ft
L	Length of wet well	ft
N	Number of equal size pumps	-
Q_i	Inflow	ft^3/s
Q_p	Total capacity of all pumps (peak discharge rate)	ft^3/s
S	Minimum submergence at the intake	ft
t_c	Minimum allowable cycle time	s
TDH	Total dynamic head	ft
V_t	Total cycling storage volume	ft^3
V_x	Individual pump cycling volumes	ft^3
W	Minimum required distance between pumps	ft
Y	Minimum level floor distance upstream of pump	ft

14.3 DESIGN CONSIDERATIONS

14.3.1 <u>Location</u>

Economic and design considerations dictate that the pump station be located relatively near the low point of the highway. Desirably, a frontage road or overpass is available for easy access to the station. The station and access road should be located on high ground so that access can be obtained if the highway becomes flooded. Soil borings should be made during the selection of the site to determine the allowable bearing capacity of the soil and to identify any potential problems.

Architectural and landscaping decisions should be made in the location phase for above-ground stations so that the station will blend into the surrounding community. Following are some considerations that should be used in the location and design of pump stations:

- Modern pump stations can be architecturally pleasing with a minimum increase in cost.
- Clean functional lines will improve the station's appearance.
- Masonry or a textured-concrete exterior can be very pleasing.
- Screening walls may be provided to hide exterior equipment and break up the lines of the building.
- A small amount of landscaping can substantially improve the overall appearance of the site.
- It may be necessary or desirable to place the station entirely underground.
- Ample parking and working areas should be provided adjacent to the station for maintenance and repair vehicles.

14.3.2 Hydrology

Because of traffic safety and flood hazards, pump stations serving major expressways and arterials are usually designed to accommodate a 50-yr storm. It is desirable to check the drainage system for the 100-yr storm to determine the extent of flooding and the associated risk. Every attempt should be made to keep the drainage area tributary to the station as small as possible. Bypass or pass through all possible drainage to reduce pumping requirements. Avoid future increases in pumping by isolating the drainage area; i.e., prevent off-site drainage from possibly being diverted to the pump station. Hydrologic design should be based on the ultimate development of the area that must drain to the station.

Designers should consider storage, in addition to that which exists in the wet well, at all pump station sites. For most highway pump stations, the high flows of the inflow hydrograph will occur over a relatively short time. Additional storage, skillfully designed, may greatly reduce the peak pumping rate required. An economic analysis can be used to determine the optimum combination of storage and pumping capacity. Because of the nature of the sites where highway-related pump stations are located, it is most often necessary that the storage be located well below normal ground level.

If flow attenuation is required for purposes other than reducing the size of the pump facility and cannot be obtained upstream of the station, consideration may be given to providing the storage downstream of the pump station. This will require large flows to be pumped and, thus, pump installation and operation costs will be higher.

If storage is used to reduce peak-flow rates, a routing procedure must be used to design the system. To determine the discharge rate, the routing procedure integrates three independent elements — the inflow hydrograph, the stage-storage relationship and the stage-discharge relationship.

14.3.3 Collection Systems

Storm drains leading to the pumping station are usually designed on a flat grade to minimize depth and cost. A minimum grade that produces a velocity of 2.5 ft/s in the pipe while flowing one-quarter full is suggested to avoid siltation problems in the collection system. Minimum cover or local head requirements should govern the depth of the uppermost inlets. The inlet pipe should enter the station perpendicular to the line of pumps. The inflow should distribute itself equally to all pumps. Baffles may be required to ensure that this is achieved.

The collector lines should preferably terminate at a forebay or storage box. However, they may discharge directly into the station. Under the latter condition, the capacity of the collectors and the storage within them is critical to providing adequate cycling time for the pumps and must be carefully calculated.

Storm drainage systems tributary to the pump station can be quite extensive and costly. Linear or intermediate storage along the storm drain may be used to reduce peak flows and pipe sizes. For some pump stations, the storage available in the collection system may be significant. However, it is often necessary to provide additional storage near the pump station. This may be done by oversizing the collection system or designing an underground vault.

In a wet-pit station where submersible or screw-type pumps are proposed, consideration of storage in the collection lines may be less critical, due to the less stringent submergence requirements of these alternative types of pumps. Also, submersible pumps can withstand more frequent cycling.

It is recommended the use of grate inlets as screens to prevent large objects from entering the system and possibly damaging the pumps. This approach has additional advantages of possibly eliminating costly trash racks and simplifying debris removal because debris can be more easily removed from the roadway than the wet well.

14.3.4 Station Types

Basically, there are two types of stations — wet-pit and dry-pit.

14.3.4.1 Wet-Pit Stations

In the wet-pit station, the pumps are submerged in a wet well or sump with the motors and the controls located overhead. With this design, the stormwater is pumped vertically through a riser pipe. The motor is commonly connected to the pump by a long drive shaft located in the center of the riser pipe. See Figure 14-1 for a typical layout. Another type of wet-pit design involves the use of submersible pumps.

Submersible pumps are now available in large sizes and should be considered for use in all station designs. Rail systems are available that allow removal of pumps without entering the wet well.

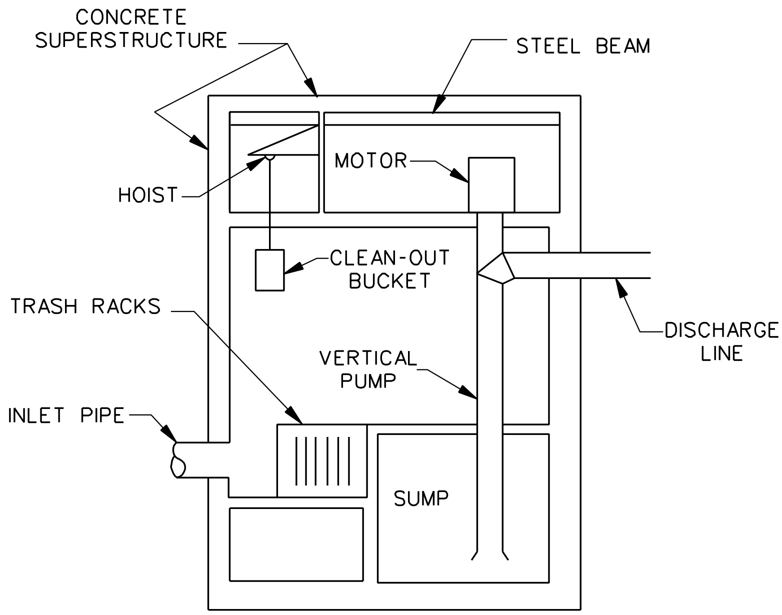


FIGURE 14-1 — Typical Wet-Pit Station

Source: HEC 24 *(4)*.

14.3.4.2 Dry-Pit Stations

Dry-pit stations consist of two separate chambers — the storage box or wet well and the dry well. Stormwater is stored in the wet well that is connected to the dry well by horizontal suction piping. Centrifugal pumps are usually used. Power is provided by either close-coupled motors in the dry well or long drive shafts with the motors located overhead. The main advantage of the dry-pit station for stormwater is the availability of a dry area for personnel to perform routine and emergency pump and pipe maintenance. See Figure 14-2 for a typical layout.

14.3.5 Pump Types

The most common types of stormwater pumps are axial flow (propeller), radial flow (impeller) and mixed flow (combination of the previous two). Each type of pump has its particular merits.

AXIAL FLOW PUMPS. Axial flow pumps lift the water up a vertical riser pipe; flow is parallel to the pump axis and drive shaft. They are commonly used for low-head, high-discharge applications. Axial flow pumps do not handle debris particularly well because the propellers will bend or possibly break if they strike a relatively large, hard object. Also, fibrous material will wrap itself around the propellers.

RADIAL FLOW PUMPS. Radial flow pumps utilize centrifugal force to move water up the riser pipe. They will handle any range of head and discharge but are the best choice for high head applications. Radial flow pumps generally handle debris quite well. A single-vane, non-clog impeller handles debris the best because it provides the largest impeller opening. The debris-handling capability decreases with an increase in the number of vanes because the size of the openings decrease.

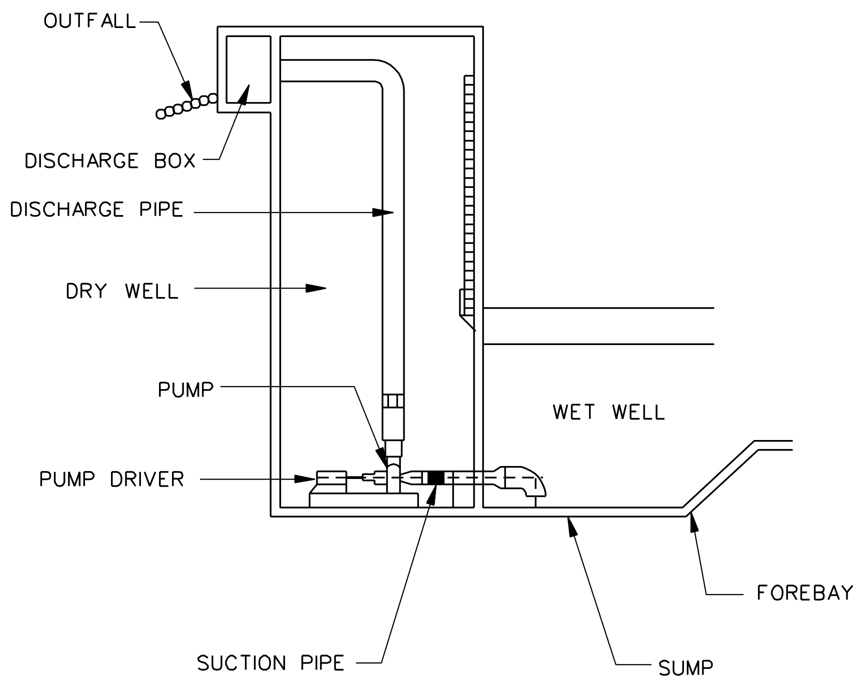


FIGURE 14-2 — Typical Dry-Pit Station

Source: HEC 24 *(4)*.

MIXED FLOW PUMPS. Mixed flow pumps are very similar to axial flow, except they create head by a combination of lift and centrifugal action. An obvious physical difference is the presence of the impeller “bowl” just above the pump inlet. They are used for intermediate head and discharge applications and handle debris slightly better than propellers.

These pumps can be driven by motors or engines housed overhead or in a dry well or by submersible motors located in a wet well. Submersible pumps frequently provide special advantages in simplifying the design, construction, maintenance and, therefore, cost of the pumping station. Use of anything other than a constant-speed, single-stage, single-suction pump would be rare.

The selection procedure is to first establish the criteria and then to select from the options available a combination that clearly meets the criteria. Cost, reliability, operating and maintenance requirements are all important considerations when making the selection. It is difficult and beyond the scope of this *Manual* to develop a totally objective selection procedure. First costs are usually of more concern than operating costs in stormwater pump stations because the operating periods during the year are relatively short. Ordinarily, first costs are minimized by providing as much storage as possible, with two or three small pumps, electrically driven.

14.3.6 Submergence

Submergence is the depth of water above the pump inlet necessary to prevent cavitation and vortexing. It varies significantly with pump type and speed, atmospheric pressure and inlet bell diameter. This dimension is provided by the pump manufacturer and is determined by laboratory testing. A very important part of submergence is the required net positive suction head (NPSH) because it governs cavitation. The available NPSH should be calculated and compared to the manufacturer's requirement. Additional submergence may be required at higher elevations. As a general rule, radial flow pumps require the least submergence while axial flow pumps require the most. Required submergence criteria developed by the Hydraulic Institute are shown in Figure 14-3 as presented in Reference *(1)*.

One popular method of reducing the submergence requirement (and therefore the station depth) for axial and mixed flow pumps, when cavitation is not a concern, is to attach a suction umbrella. A suction umbrella is a dish-shaped steel plate attached to the pump inlet that improves the entrance conditions by reducing the intake velocities. For umbrella velocities of 2 ft/s or less, a submergence-to-pump-bell diameter ratio of 0.80 can be used.

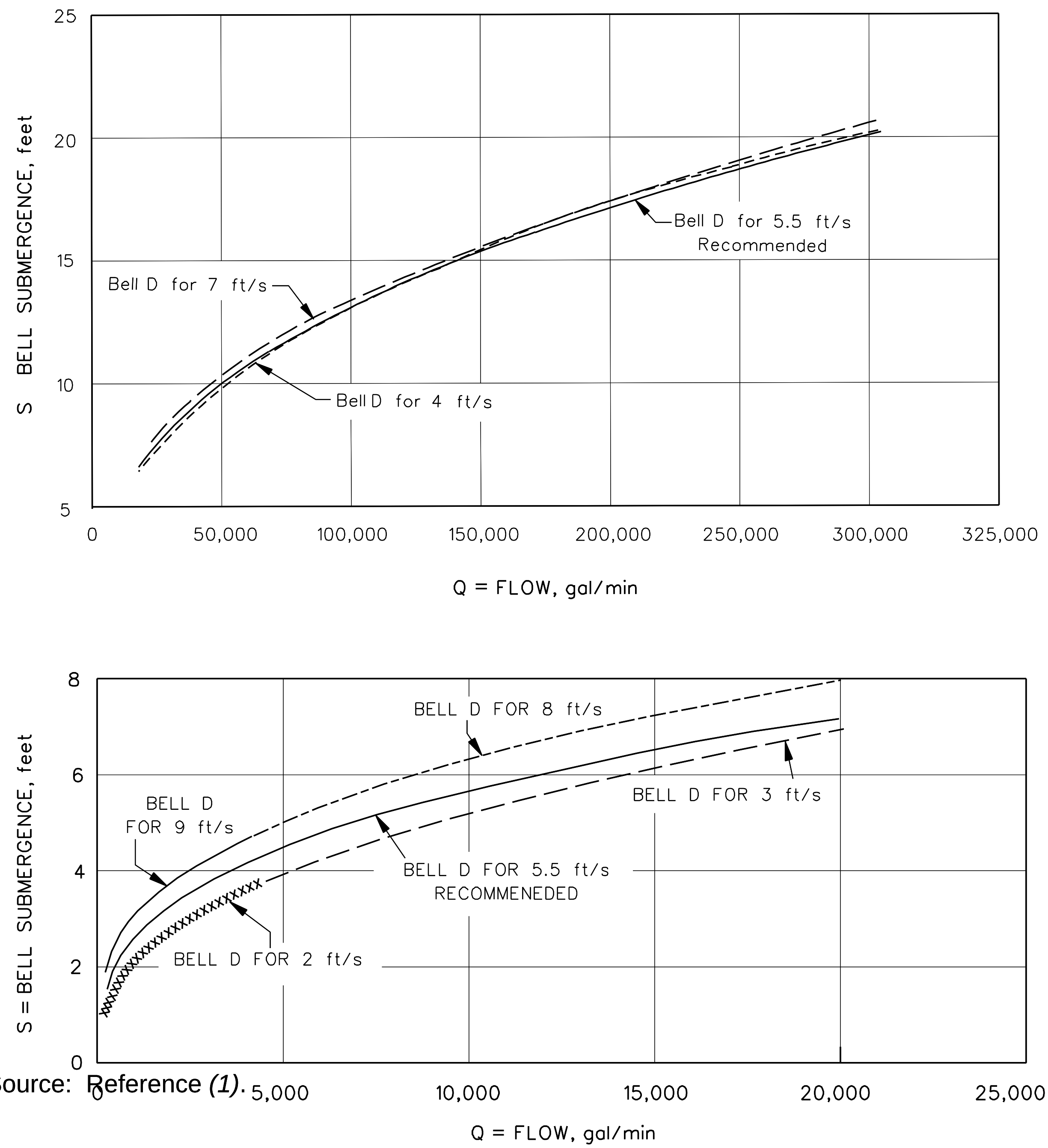
14.3.7 Water-Level Sensors

The water-level sensors activate the pumps and, therefore, are a vital component of the control system. There are a number of different types of sensors that can be used. Types include the float switch, electronic probes, ultrasonic devices, mercury switch and air pressure switch. The location or setting of these sensors control the starting and stopping of the pump motors. Their function is critical, because the pump motors or engines must not start more frequently than an allowable number of times per hour (i.e., the minimum cycle time) to avoid damage. To prolong the life of the motors, sufficient volume must be provided between the pump start and stop elevations to meet the minimum cycle time requirement. The on-off setting for the first pump is particularly important, because it defines the most frequently used cycle.

14.3.8 Power

Several types of power may be available for a pump station. Examples are electric motors and gasoline, diesel or natural gas engines. The designer should select the type of power that best meets the needs of the project based on an estimate of future energy considerations and overall station reliability. A comparative cost analysis of alternatives is helpful in making this decision. However, when readily available, electric power is usually the most economical and reliable choice today. The region maintenance engineer should provide input in the selection process.

The need for backup power is dependent upon the consequences of failure. The decision to provide it should be based on economics and safety. For electric motors, two independent electrical feeds from the electric utility with an automatic transfer switch may be the cost-effective choice when backup power is required. A standby generator is generally less cost effective because of its initial costs. Also, standby generators require considerable maintenance and testing to ensure operation in times of need.



Source: Reference *(1)*.

FIGURE 14-3 — Recommended Minimum Submergence to Minimize Free-Surface Vortices

For extensive, depressed freeway systems involving a number of electric motor-driven stations, a mobile generator may be the cost-effective choice for backup power. A trailer-mounted generator can be stored at any one of the pump stations. If a power outage occurs, maintenance forces can move the generator to the affected station to provide temporary power. If a mobile generator is used as the source of backup power, it may be necessary to add additional storage to compensate for the time lag that results in moving the generator from site to site. This lag will typically be 1.0 h to 1.5 h from the time the maintenance forces are notified.

14.3.9 Discharge System

The discharge piping should be as simple as possible. Pumping systems that lift the stormwater vertically and discharge it through individual lines to a gravity storm drain as quickly as possible are preferred. Individual pump discharge lines are the most cost-effective system for short outfall lengths. Damaging pump reversal could occur with very long forced mains. Check valves should be installed. The effect of stormwater returning to the sump after pumping stops should be considered. Individual lines may exit the pumping station either above or below grade. Frost depth shall be considered when deciding the depth of discharge piping. Frozen discharge pipe could exert additional back pressure on pumps.

It may be necessary to pump to a higher elevation using long discharge lines. This may dictate that the individual lines be combined into a force main via a manifold. For such cases, check valves must be provided on the individual lines to keep stormwater from running back into the wet well and restarting the pumps or prolonging their operation time. Check valves should preferably be located on horizontal layouts rather than vertical to prevent sedimentation on the downstream side after the valve closing. Gate valves should be provided in each pump discharge line to provide for continued operation during periods of repair. A cost analysis should be performed to determine what length and type of discharge piping justifies a manifold. The number of valves required shall be kept to a minimum to reduce cost, maintenance and head loss through the system.

14.3.10 Flap Gates and Valving

14.3.10.1 Flap Gates

The purpose of a flap gate is to restrict water from flowing back into the discharge pipe and to discourage entry into the outfall line. Flap gates are usually not watertight, so the elevation of the discharge pipe should be set above the normal water levels in the receiving channel. If flap gates are used, it may not be necessary to provide for check valves.

14.3.10.2 Check Valves

Check valves are watertight and are required to prevent backflow on forcemains that contain sufficient water to restart the pumps. They also effectively stop backflow from reversing the direction of pump and motor rotation. They must be used on manifolds to prevent return flow from perpetuating pump operation. Check valves should be “non-slam” to prevent water hammer. Types include swing, ball, dash pot and electric.

14.3.10.3 Gate Valves

Gate valves are simply a shut-off device used on force mains to allow for pump or valve removal. These valves should not be used to throttle flow. They should be either totally open or totally closed.

14.3.10.4 Air/Vacuum Valves

Air/vacuum valves are used to allow air to escape the discharge piping when pumping begins and to prevent vacuum damage to the discharge piping when pumping stops. They are especially important with large-diameter pipe. If the pump discharge is open to the atmosphere, an air-vacuum release valve is not necessary. Combination air release valves are used at high points in force mains to evacuate trapped air.

14.3.11 Trash Racks and Grit Chambers

Trash racks should be provided at the entrance to the wet well if large debris is anticipated. For stormwater pumping stations, simple steel bar screens are adequate. Usually, the bar screens are inclined with bar spacings approximately 1½ in. Constructing the screens in modules facilitates removal for maintenance. If the screen is relatively small, an emergency overflow should be provided to protect against clogging and subsequent surcharging of the collection system. As previously noted, screening large debris at surface inlets may be very effective in minimizing the need for trash racks.

If substantial amounts of sediment are anticipated, a chamber may be provided to catch solids that are expected to settle out. This will minimize wear on the pumps and limit deposits in the wet well. The grit chamber should be designed so that a convenient means of sediment removal is available.

14.3.12 Ventilation

Ventilation of dry and wet wells is necessary to ensure a safe working environment for maintenance personnel. The ventilation system can be activated by a light switch at the entrance to the station. Maintenance procedures should require personnel to wait several minutes after ventilation has started before entering the well. Some owners require the testing of the air in the wet well prior to allowing entry. Safety procedures for working in wet wells should be well established and carefully followed.

If mechanical ventilation is required to prevent buildup of potentially explosive gasses, the pump motors or any spark-producing equipment should be rated explosion proof or the fans run continuously.

Heating and dehumidifying requirements are variable. Their use is primarily dependent upon equipment and station type, environmental conditions and station use.

14.3.13 Roof Hatches and Monorails

It will be necessary to remove motors and pumps from the station for periodic maintenance and repair. Removable roof hatches located over the equipment is a cost-effective way of providing this capability. Mobile cranes can simply lift the equipment directly from the station onto maintenance trucks. Monorails are usually more cost-effective for larger stations.

14.3.14 Equipment Certification and Testing

Equipment certification and testing is a crucial element of pump station design. The purchaser has a right to witness equipment testing at the manufacturer's lab. However, this is not always practical. As an alternative, the manufacturer should provide certified test results to the owner. It is good practice to include in the contract specifications the requirement for acceptance testing by the owner, when possible, to ensure proper operation of the completed pump station. The testing should be done in the presence of the owner's representative. If the representative waves his right to observe the test, a written report should be provided to give assurance that the pump equipment meets all performance requirements. Any component that fails should be repaired and retested.

14.3.15 Monitoring

Pump stations are vulnerable to a wide range of operational problems from malfunction of the equipment to loss of power. Monitoring systems (e.g., on-site warning lights, remote alarms) can help minimize such failures and their consequences.

Telemetering is an option that should be considered for monitoring critical pump stations. Operating functions may be telemetered from the station to a central control unit. This allows the central control unit to initiate corrective actions immediately if a malfunction occurs. Such functions as power, pump operations, unauthorized entry, explosive fumes and high-water levels can be monitored effectively in this manner. Perhaps the best overall procedure to assure the proper functioning of a pump station is the implementation of a regular schedule of maintenance conducted by trained, experienced personnel.

14.3.16 Hazardous Spills

The possibility of hazardous spills is always present under highway conditions. In particular, this has reference to gasoline and the vulnerability of pump stations and pumping equipment to fire damage. There is a history of such incidents having occurred and also of spills of oils, corrosive chemicals, pesticides and the like having been flushed into stations, with undesirable results. The usual design practice has been to provide a closed conduit system leading directly from the highway to the pump station without any open forebay to intercept hazardous fluids or vent off volatile gases. With a closed system, there must be a gas-tight seal between the pump pit and the motor room in the pump station. Preferably, the pump station should be isolated from the main collection system and the effect of hazardous spills by a properly designed storage facility upstream of the station. This may be an open forebay or a closed box below the highway pavement or adjacent to it. The closed box must be ventilated by sufficient grating area at each end.

14.4 ADDITIONAL CONSIDERATIONS

14.4.1 Construction

The method of construction has a major impact on the cost of the pump station. For near continuous operation (e.g., pumping sewage), it has been estimated that construction represents more than 20% of the pump station costs over a 10-yr period. With a less frequently operating stormwater pump station, operating costs may be insignificant compared to construction costs. Therefore, the type of construction should be chosen carefully, between caisson construction, in which the station is usually circular, or open-pit construction. Soil conditions are the primary factor in selecting the most cost-effective alternative.

Feedback should be provided by the construction personnel on any problems encountered in the construction of the station so that the designers can improve future designs. Any changes should be documented by “as-built” drawings. Construction inspections of pump stations should be conducted by personnel who are knowledgeable and experienced with such equipment.

14.4.2 Maintenance

Because major storm events are infrequent, a comprehensive, preventive maintenance program should be developed for maintaining and testing the equipment so that it will function properly when needed. Instruments such as hour meters and number-of-starts meters should be used on each pump to help schedule maintenance. Input from maintenance forces should be a continuous process so that each new generation of stations will be an improvement.

14.4.3 Retrofitting Stations

Retrofitting existing stormwater pump stations may be required when changes to the highway increase runoff to them. The recommended approach to this problem is to first try to increase the capacity of the station without making major structural changes. This can be achieved by using a cycling sequence that requires less cycling volume or power units that allow a greater number of starts per hour (i.e., shorter cycling time). Submersible pumps have been used effectively in retrofitting stations because of the flexibility in design and construction afforded by their frequent cycling capability. Other common reasons for the need for retrofit include problems associated with excessive wear and tear and poor performance of the pumps such as:

- excessive cycling,
- cavitation causing excessive noise and vibration,
- poor distribution of flow to pumps, and
- excessive head losses in discharge system.

Refer to HEC 24 *(4)* for a detailed discussion on the cause of problems and appropriate retrofit measures and other correctional actions.

14.4.4 Safety

All elements of the pump station should be carefully reviewed for safety of operation and maintenance. Ladders, stairwells and other access points should facilitate use by maintenance personnel. Adequate space should be provided for the operation and maintenance of all equipment. Particular attention should be given to guarding moving components such as drive shafts and providing proper and reliable lighting. It may also be prudent to provide air-testing equipment in the station so maintenance personnel can be assured of clean air before entering.

Pump stations may be classified as a confined space, in which case access requirements and any safety equipment are all defined by code. Pump stations should be designed to be secure from entry by unauthorized personnel, and as few windows as possible should be provided.

14.5 RECOMMENDED DESIGN CRITERIA

14.5.1 Objective

The following recommendations are being made with the objective of minimizing the construction, operation and maintenance costs of highway stormwater pump stations while remaining consistent with the practical limitations of all aspects.

14.5.2 Station Type and Depth

Because dry-pit stations are as much as 60% more expensive than wet-pit stations, wet-pit stations are recommended. Dry-pit stations may be considered where ease of access for repair and maintenance is a primary concern.

The station depth should be no greater than that required for pump submergence and clearance below the inlet invert, unless foundation conditions dictate otherwise.

14.5.3 Power

Electric is usually most desirable if available. Recommend constant-speed, 3-phase induction motors (National Electrical Manufacturers Association, Design B). Motor voltages between 440 and 575 are very economical for pumping applications. Consequently, the recommended maximum size is 300 hp. This size is also a good upper limit for ease of maintenance.

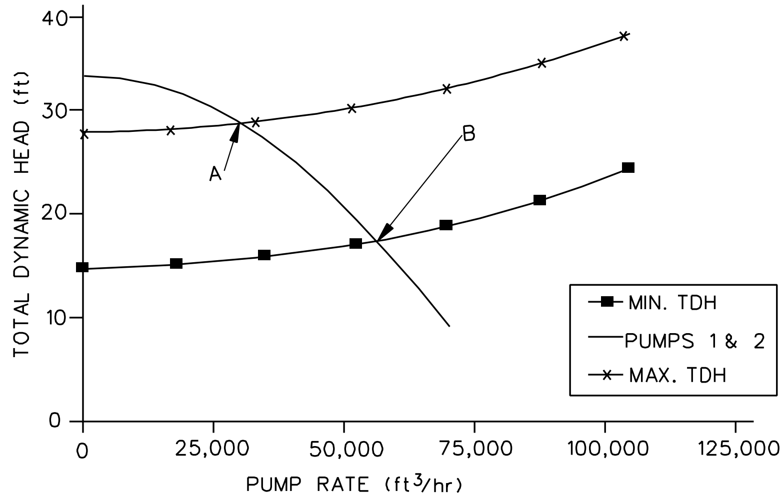
Consideration should be given to whether the pump station is to have standby power (SBP). If the Department prefers that stations have a SBP receptacle, a manual transfer switch and a portable engine/generator set, then the practical power limit of the pumps becomes (75 hp), because this is the limit of the power-generating capabilities of most portable generator units. Two pumps would be operated by one engine/generator set.

14.5.4 Total Dynamic Head and System Curve

Because stormwater pumps are extremely sensitive to changes in head, the head demand on the pumps should be calculated as accurately as possible. All valve and bend losses should be considered in the computations. In selecting the size of discharge piping, consideration should be given to the manufactured pump outlet size vs. the head loss produced by smaller piping. This approach should identify a reasonable compromise in balancing cost.

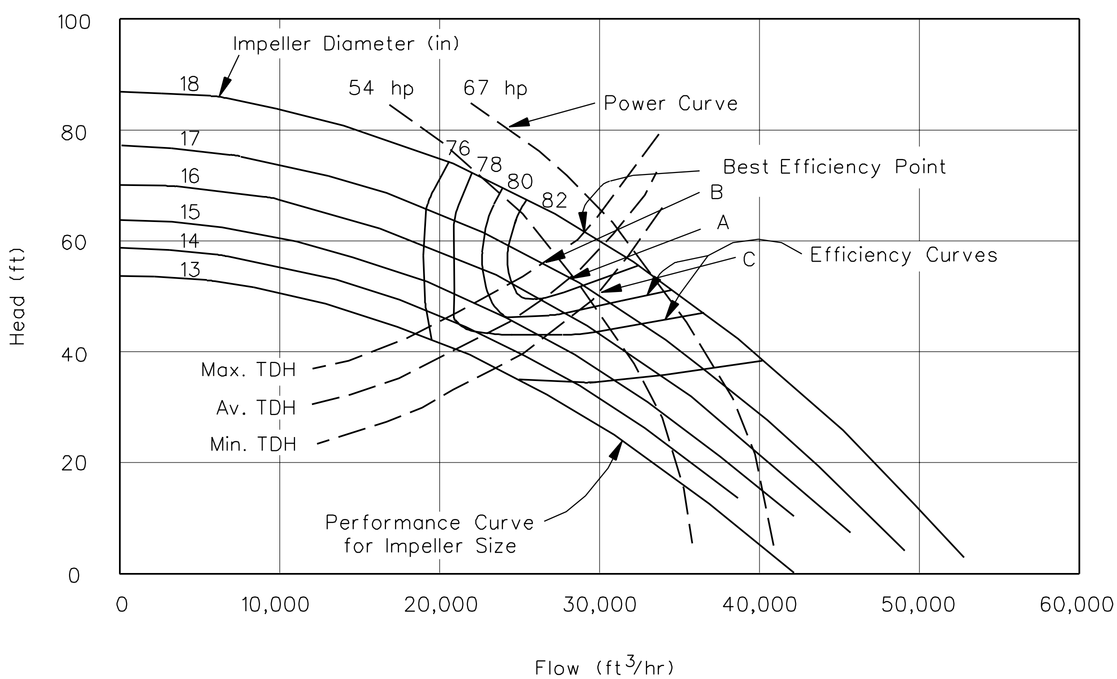
The combination of static head, velocity head and various head losses in the discharge system due to friction is called total dynamic head (TDH). It is usual to minimize these various head losses by the selection of correctly sized discharge lines and other components.

The total dynamic head varies with the discharge and is calculated for the range of discharges expected; the system head curve (Q vs. TDH) can then be plotted. This curve defines the energy required to pump any flow through the discharge system. It is especially critical for the analysis of a discharge system with a forcemain. When overlaid with pump performance curves (provided by the manufacturer), it will yield the pump operating points (see Figure 14-4). Pumps for a given station are selected to all operate together to deliver the design Q at a total dynamic head computed to correspond with the design water level.



Source: Reference *(4)*.

FIGURE 14-4 — System Curve for Two Pumps with Separate Discharge Lines with an Operating Range from A to B



Source: Reference *(4)*.

Note: Point A represents the design point. Points B and C represent pump rates at maximum and minimum TDH.

FIGURE 14-5 — Pump Performance

14.5.5 Main Pumps

14.5.5.1 Number And Capacity

The designer will determine the number of pumps needed by following a systematic process defined in Section 14.6. However, two to three pumps have been judged to be the recommended minimum. If the total discharge to be pumped is small and the area draining to the station has little chance of increasing substantially, the use of a two-pump station is preferred. Consideration may be given to oversizing the pumps to compensate, in part, for a pump failure. The two-pump system could have pumps designed to pump 66% to 100% of the required discharge, and the three-pump system could be designed so that each pump will pump 50% of the design flow. The resulting damage caused by the loss of one pump could be used as a basis for deciding the size and number of pumps.

It is recommended that economic limitations on power unit size and practical limitations governing operation and maintenance be used to determine the upper limit of pump size. The minimum number of pumps used may increase due to these limitations.

It is also recommended that equal-size pumps be used. Identical size and type enables all pumps to be freely alternated into service. This equalizes wear and reduces needed cycling storage. It also simplifies scheduling maintenance and allows pump parts to be interchangeable. Hour feet and start feet) should be provided to aid in scheduling needed maintenance.

14.5.5.2 Final Selection

For the typical highway application, any of the three pump types described in Section 14.3.5 will usually suffice. If not, manufacturers' information will likely dictate the type required. However, knowing the operating RPMs, a computation for specific speed can be made to check the appropriateness of the pump type (see Figure 14-6 to determine the ranges where specific impeller types should be used). Suction-specific speed may be defined as that speed in revolutions per minute at which a given impeller would operate if reduced proportionally in size so as to deliver a capacity of 0.06 gal/s against a TDH of 3 ft. This is an index number descriptive of the suction characteristics of a given pump. Higher, numerical values are associated with better NPSH capabilities. This number should be checked for dry-pit applications and systems with suction lifts. Once the pump type and capacity have been determined, the final selection of the pump can be made.

14.5.6 Standby/Spare Pumps

Considering the short duration of high inflows, the low frequency of the design storm, the odds of a malfunction and the typical consequences of a malfunction, spare or standby pumps are not warranted in stormwater applications. If the consequences of a malfunction are particularly critical, it is more appropriate to add another main pump and reduce their size accordingly.

14.5.7 Sump Pumps

These are usually small, submersible pumps necessary only in the dry well of dry-pit stations to protect equipment from seepage water damage. Because of their size, they are prone to sediment locking in wet-pit stations and, therefore, are not recommended for those installations. If it is necessary to evacuate the wet well, a portable pump can be used.

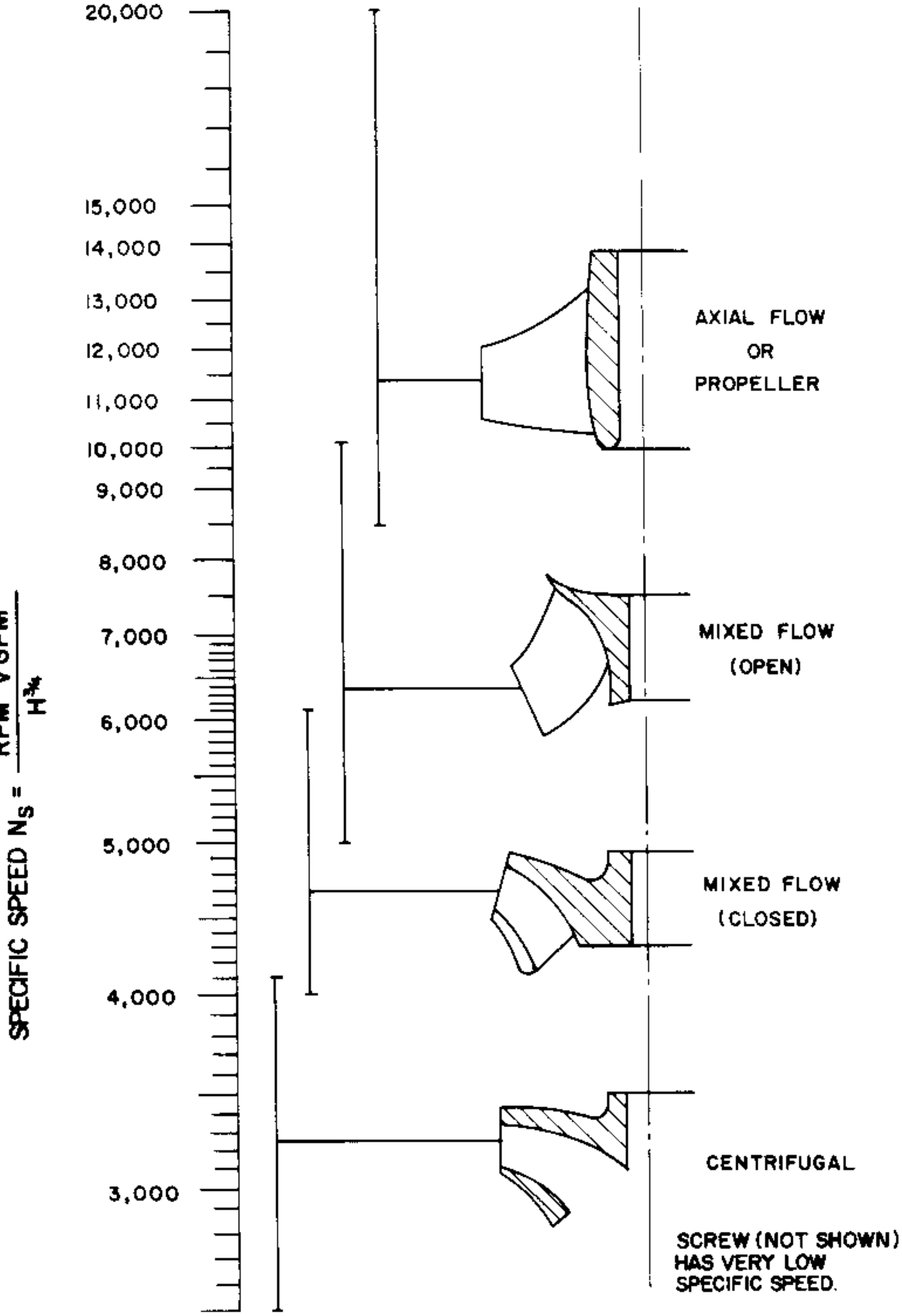


FIGURE 14-6— Specific Pump Speed vs. Impeller Types

Source: Reference *(7)*.

14.5.8

Storage

The total storage capacity that can or should be provided is an important initial consideration in pump station design. Using the hydrograph and pump-system curves, various levels of pump capacity can be tried, and the corresponding required total storage can be determined. The basic principle is that the volume of water as represented by the shaded area of the hydrograph in Figure 14-7 is beyond the capacity of the pumps and must be stored. If a larger part or most of the design storm is allowed to collect in a storage facility, a smaller set of pumps can be utilized, with anticipated cost/benefits. If the discharge rate is to be limited, ample storage is essential.

Because most highway-related pump stations are associated with either short underpasses or long depressed sections, it is not reasonable to consider aboveground storage. Water that originates outside of the depressed areas should not be allowed to enter the depressed areas because of the need to pump all of this water. The simplest form of storage for these depressed situations is either the enlargement of the collection system or the construction of an underground storage facility. These can typically be constructed under the roadway area and will not require additional right-of-way.

14.5.9 Wet Well Design

The primary variables for sizing the wet well are:

- number of pumps,
- pump bell diameter,
- pump bay width,
- minimum distance to trash rack, and
- minimum distance to inlet invert.

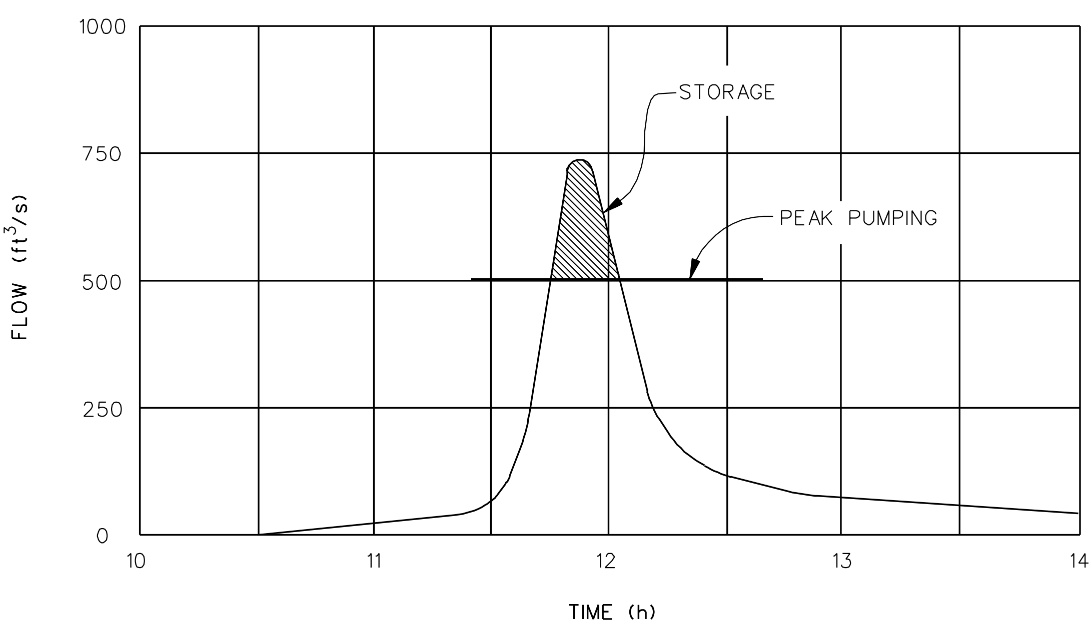


FIGURE 14-7 — Estimating Required Storage

The criteria to be considered in wet well design includes the floor clearance, the minimum distance between an inlet bell and the wall, the minimum clearance between adjacent inlet bells, the width of partition walls between pumps and the submergence required for the pump bell diameter. The specific criteria for both circular and rectangular wet wells can be found in Reference *(1)*, from the pump supplier and in HEC 24 *(4)*.

The wet well size and shape are important factors for both their contribution to available storage and for providing room for proper sizing and layout of pumps. However, the final number and size of pumps is normally not known until the final design phase. Therefore, it becomes necessary to estimate wet well dimensions based on a trial number and size of pumps. It may then be necessary during the design process to increase dimensions to provide additional storage or to accommodate additional pumps. The determination of final sump size and clearance checks are performed after trial pump selection and sizing procedures in Section 14.6.2. The volume of the wet well is often only a small portion of the total available storage that is used by the system to reduce the required pump capacity and to control the cycling interval of the pumps.

14.5.9.1 Cycling Sequence and Volumes

Cycling is the starting and stopping of pumps, the frequency of which must be limited to prevent damage and possible malfunction. The wet well must be designed to provide sufficient volume for safe cycling, or sufficient volume must be provided outside the wet well. However, to keep sediment in suspension, the wet well should not be oversized. The volume required to satisfy the minimum cycle time is dependent upon the characteristics of the power unit, the number and capacity of pumps, the sequential order in which the pumps operate and whether or not the pumps are alternated during operation.

There are two basic cycling sequences. One will be referred to here as the “common off elevation.” In this sequence, the pumps start at successively higher elevations as required; however, they all stop at the same off elevation. This is advantageous when large amounts of sediment are anticipated. The other sequence uses a “successive start/stop” arrangement in which the start elevation for one pump is also the stop elevation for the subsequent pump; i.e., the start elevation for Pump 1 is the stop elevation for Pump 2, the start elevation for Pump 2 is the stop elevation for Pump 3 (see Figure 14-8). There are countless variations between these two sequences.

There are also different alternation techniques that reduce the cycling volume requirement and equalize wear on the pumps. They range from simply alternating the first pump to start, to continuously alternating all pumps during operation, a technique referred to as “cyclical running alternation.” Using this technique, each pump is stopped in the same order in which it starts; i.e., the first pump to start will be the first pump to stop (see Figure 14-9).

Alternating the first pump to start is sufficient for stormwater pump stations where more than one pump on will be rare and of short duration. This alternation technique coupled with the successive start/stop cycling sequence requires the smallest total cycling volume possible (see Figure 14-10).

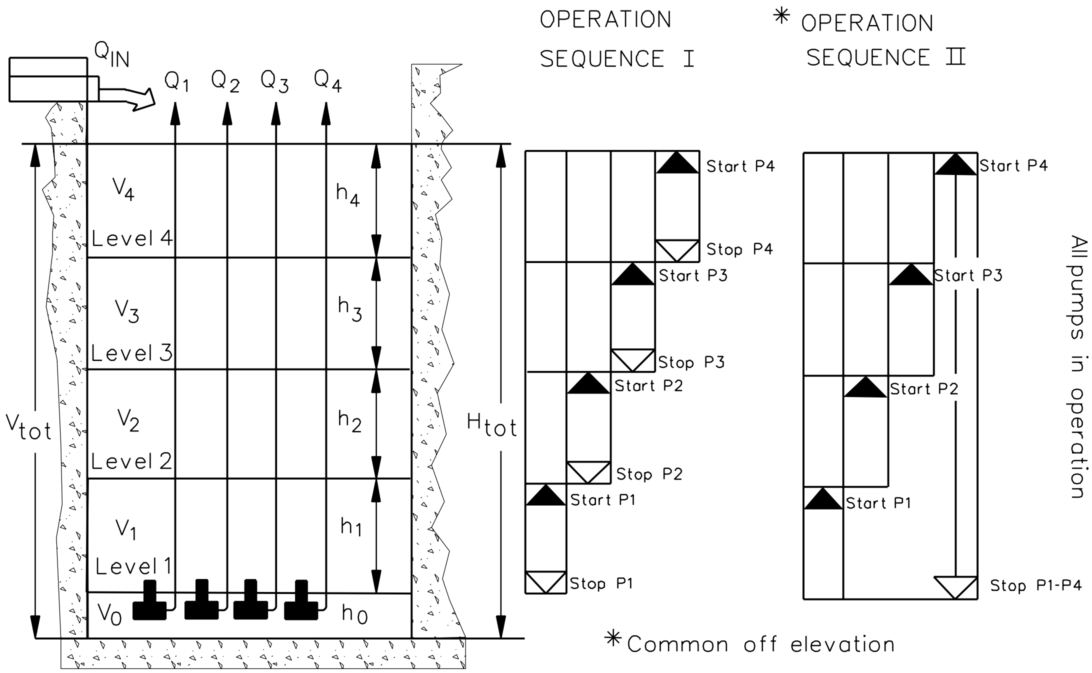


FIGURE 14-8 — Pump Sequences (after *(5)*)

Notes to Figure 14-8:

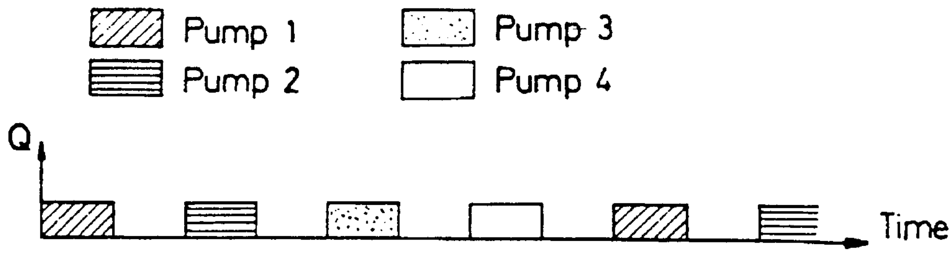
1. Decreasing sump volume by pump alternation. Pumps starting in sequence and stopping in reverse order.

2. By designing the control system for pump alternation, the sump volumes can be reduced and distribute the pump operating time more evenly between the four pumps.

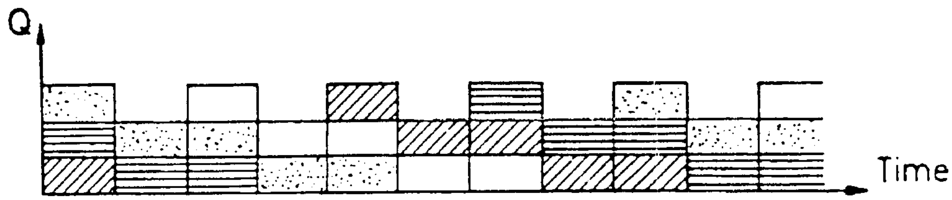
3. This system works for any number of pumps in a station.

4. For example, when four pumps are installed in the same station and if the inflow is less than the capacity of one pump, pump number one would, without alternation, do all the work.

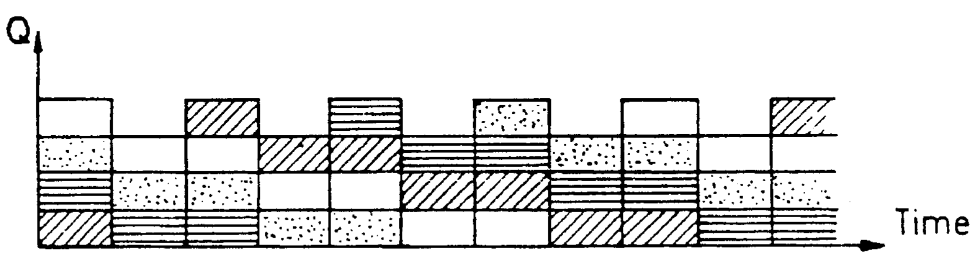
5. With alternation, pump number one starts and draws down. Next start would call pump number two. This means that, with four pumps of the same size and operating in an alternating sequence, each pump is called on to pump down the sump volume, V1, every fourth time. The cycle time of each pump will be four times longer than the cycle time of filling and emptying of V1.



The pumps will, however, run only 1/8 of their cycle time (provided Q pump = 2 x Qin).

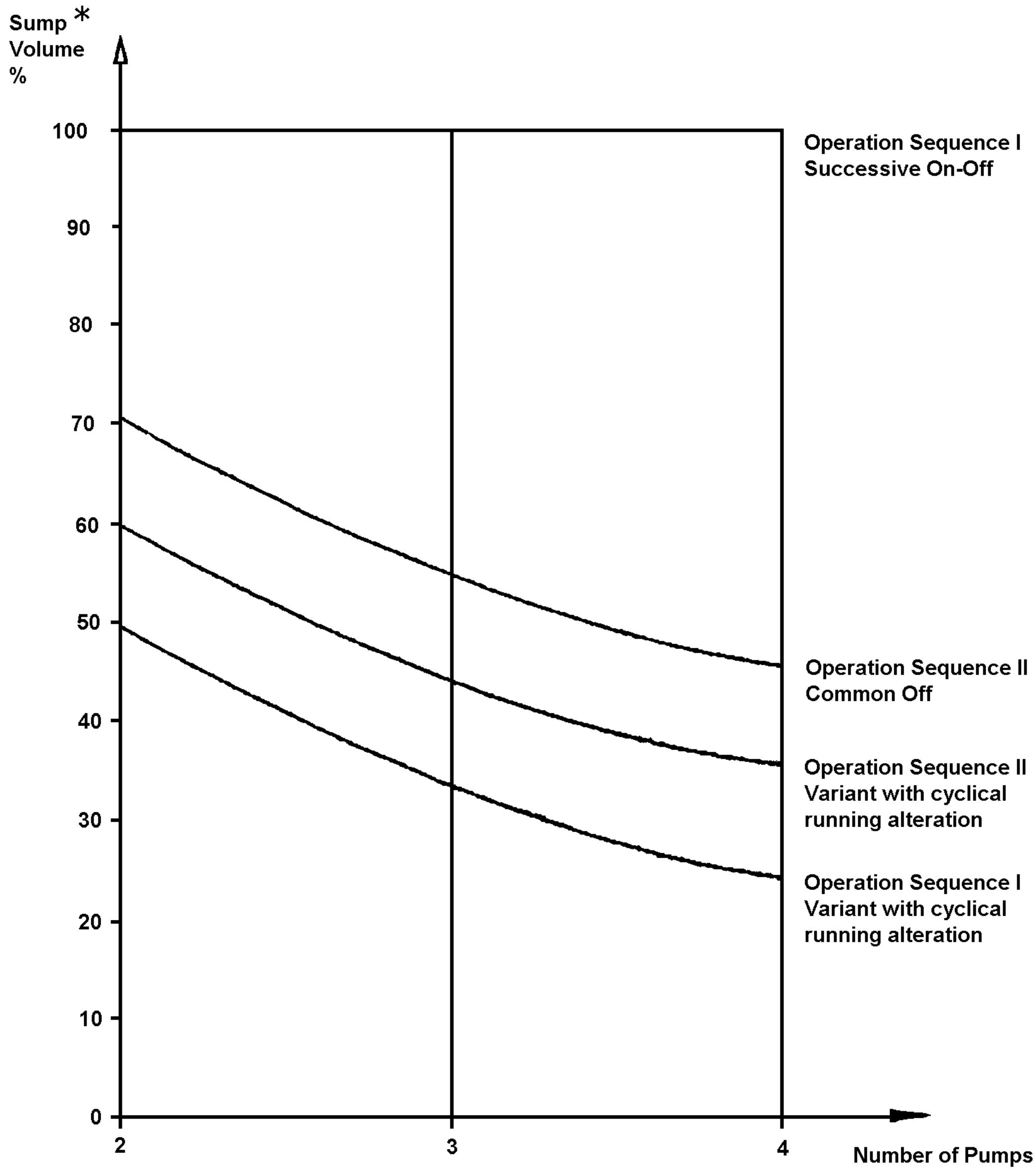


If Qin is greater than the capacity of two pumps but less than three pumps, the pumps are operating 5/8 of their cycle time.



If the inflow is greater than the capacity of three pumps but less than the capacity of four pumps the pumps will operate 7/8 of their cycle time.

FIGURE 14-9 — Schematic of Pump Sequences at Different Inflow Rates; Pumps with Cyclical Running Alternation — A Variant of Operation Sequence I in Figure 14-8 (after *(5)*)



*100% corresponds to the volume, which is received from the formula $V_t = (Q_p T_{min})/4$

FIGURE 14-10 — A Comparison of The Pump Volume With and Without Cyclical Running Alternation (after *(5)*)

14.5.9.2 Lowest Pump “Off” Elevation

The Hydraulic Institute recommends that the lowest pump “off” elevation be no lower than the invert elevation, unless plan dimension constraints dictate that the station floor be lowered to obtain the necessary cycling volume (refer to HEC 24 *(4)*). This recommendation is based on the fact that it is usually less expensive to expand a station’s plan dimensions than to increase its depth. This elevation represents the static pumping head to be used for pumping selection.

14.5.9.3 Pump “On” Elevations

These should be set at the elevations that satisfy the individual pump cycling volumes (V_x). Starting the pumps as soon as possible by incrementing these volumes successively above the lowest pump-off elevation will maximize what storage is available within the wet well and the collection system. The depth required for each volume is computed as follows:

$$H_x = V_x / \text{plan area} \tag{14.1}$$

14.5.9.4 Allowable High-Water Elevation

The allowable high-water (AHW) elevation in the station should be set such that the water surface elevation at the lowest inlet in the collection system provides 1 ft to 2 ft of freeboard below the roadway grate.

14.5.9.5 Clearances

Pump-to-pump, pump-to-backwall and pump-to-sidewall clearances should be the minimum possible to minimize the potential for sedimentation problems. Consult the manufacturer or a dimensioning guide. The pump inlet-to-floor clearance plus the pump submergence requirement constitutes the distance from the lowest pump “off” elevation to the wet well floor. The final elevation may have to be adjusted if the type of pump to be installed is different than anticipated. (Refer to HEC 24 *(4)* for additional information).

14.5.9.6 Intake System Design

The primary function of the intake structure is to supply an even distribution of flow to the pumps. An uneven distribution may cause strong local currents resulting in reduced pump efficiency and undesirable operational characteristics. The ideal approach is a straight channel coming directly into the pump or suction pipe. Turns and obstructions are detrimental, because they may cause eddy currents and tend to initiate deep-cored vortices. The inflow should be perpendicular to a line of pumps, and water should not flow past one pump to get to another. Unusual circumstances will require a unique design of the intake structure to provide proper flow to the pumps.

14.5.10 Stormwater Pump Station Storage

The development of the wet well design as discussed in Section 14.5.9 has general application when it is anticipated that most of the peak flow will be pumped. In that case, pump run time and cycling sequences are of great importance. For many of the highway storm drain situations, it has been the practice to store substantial parts of the flow to minimize pumping requirements and outflow piping. The demands on the pumping system are different and, thus, additional considerations should be made.

The designer should recognize that a balance should be reached between pump rate and storage volume. This will require a trial-and-error procedure used in conjunction with an economic analysis. Pump stations are very costly, and alternatives to minimize total costs need to be considered.

The principles discussed for minimum run time, pump cycling, etc., in the design of wet wells should also be considered for larger storage volume development. However, it will be noted that differences exist as the volume of storage becomes larger. Typically, the concern for meeting minimum run times and cycling time will be reduced because the volume of storage is sufficient to prevent these conditions from controlling the pump operation. The start and stop elevations will be of different magnitudes because of the volume represented by each increment of storage depth.

The approach used for the design of the pump station will be that associated with the development of an inflow mass curve. In this process, the designer will need to have an inflow hydrograph and a developed stage-storage relationship. Trial pumping systems will be imposed on the inflow mass curve to develop a mass curve routing diagram. The inflow hydrograph is a fixed design component while the storage and pumping discharge rates are variable. The designer may assign a pumping discharge rate based on downstream capacity considerations, limits imposed by local jurisdictions, etc. It is becoming a common requirement that post-development discharges not exceed predevelopment discharges. This requirement can most often be met with a design that includes storage. With the inflow mass curve and an assigned pumping rate, the required storage can be determined by various trials of the routing procedure.

14.6 DESIGN PROCEDURE

14.6.1 Introduction

The following is a systematic procedure that integrates the hydraulic design variables involved in pump design. It incorporates the above recommended design criteria and yields the required number and capacity of pumps and the wet well and storage dimensions. The final dimensions can be adjusted as required to accommodate non-hydraulic considerations such as maintenance. Though the recommended station is a wet-pit, this procedure can be adapted for use in designing dry-pit stations.

Theoretically, an infinite number of designs are possible for a given site. Therefore, to initiate design, constraints must be evaluated and a trial design formulated to meet these constraints. Then, by routing the inflow hydrograph through the trial pump station, its adequacy can be evaluated.

The hydraulic analysis of a pump station involves the interrelationship of three components:

- the inflow hydrograph,
- the storage capacity of the wet well and the outside storage, and
- the discharge rate of the pumping system.

The inflow hydrograph is determined by the physical factors of the watershed and regional climatological factors. The discharge of the pump station is often controlled by local regulations or physical factors. Therefore, the main objective in pump station design is to store enough inflow (volume of water under the inflow hydrograph) to allow the station discharge to meet specified limits. Even if there are no physical limitations to pump station discharge, storage should always be considered because storage permits the use of smaller and/or fewer pumps.

14.6.2 Pump Station Design

The procedure for pump station design is illustrated in the following Steps:

Step 1 Inflow to Pump Station

Develop inflow hydrograph to the pump station using the procedures presented in the Hydrology Chapter.

Step 2 Estimate Pumping Rate, Volume of Storage and Number of Pumps

Because of the complex relationship among the variables of pumping rates, storage and pump on-off settings, a trial-and-error approach is usually necessary for estimating the pumping rates and storage required for a balanced design. A wide range of combinations will produce an adequate design. The goal is to develop an economic balance between volume and pumping capacity.

Some approximation of all three parameters is necessary to produce the first trial design. One approach is shown in Figure 14-7. In this approach, the peak pumping rate is assigned, and a horizontal line representing the peak rate is drawn across the top of the hydrograph. The shaded area above the peak pumping rate represents an estimated volume of storage required above the last pump turn-on point. This area is measured to give an estimated starting size for the storage facility. Once an estimated storage volume is determined, a storage facility can be estimated. The shape, size and depth can be established to match the site, and a stage-storage relationship can be developed.

The total pumping rate may be set by stormwater management limitations, capacity of the receiving system, the desirable pump size or available storage. Two pumps would be the minimum number of pumps required. However, as many as five pumps may be needed for a continuously depressed highway situation. Size and thus numbers of pumps may be controlled by physical constraints such as portable standby power as discussed in Section 14.5.3.

Step 3 Design High-Water Level

The highest permissible water level must be set as 1 ft to 2 ft below the finished pavement surface at the lowest pavement inlet. The lower the elevation, the more conservative the design.

At the design inflow, some head loss will occur through the pipes and appurtenances leading to the pump station. Therefore, a hydraulic gradient will be established, and the maximum permissible water elevation at the station will be the elevation of the hydraulic gradient. This gradient will be very flat for most wet well designs with exterior storage because of the unrestricted flow into the wet well.

Step 4 Determine Pump Pit Dimensions

Determine the minimum required plan dimensions for the pump station from the manufacturer's literature or from dimensioning guides such as those provided by the Hydraulic Institute; see Table 14-2 and Figure 14-11. The dimensions are usually determined by locating the selected number of pumps on a floor plan, considering the guidance in Section 14.5.9 for clearances and intake system design. Remember the need for clearances around electrical panels and other associated equipment that will be housed in the pump station building.

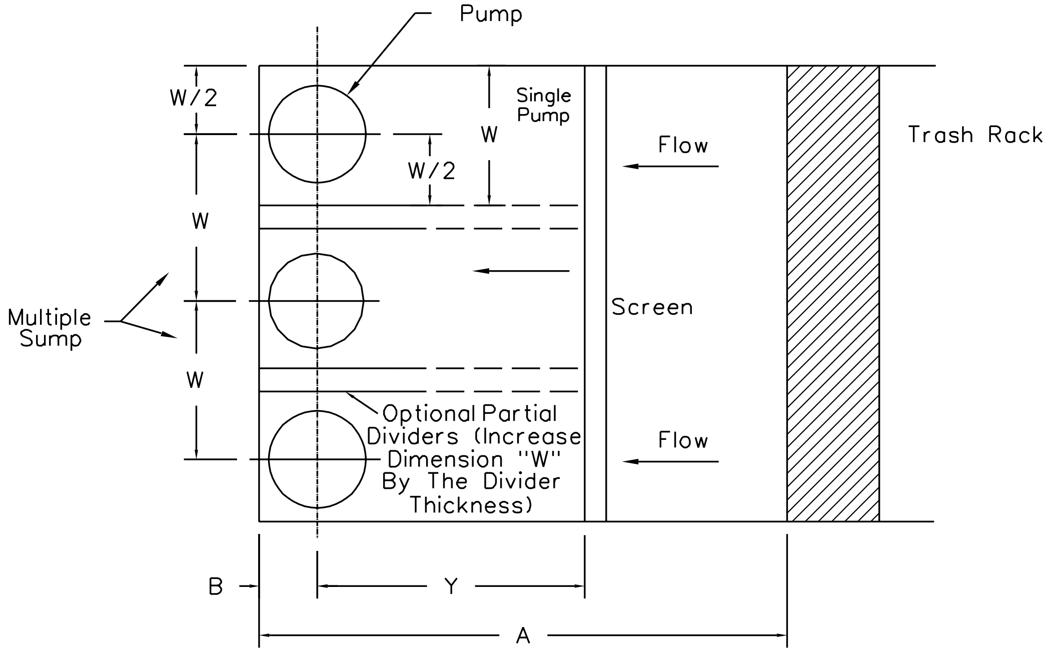
Step 5 Stage-Storage Relationship

Routing procedures require that a stage-storage relationship be developed. This is accomplished by calculating the available volume of water for storage at uniform vertical intervals.

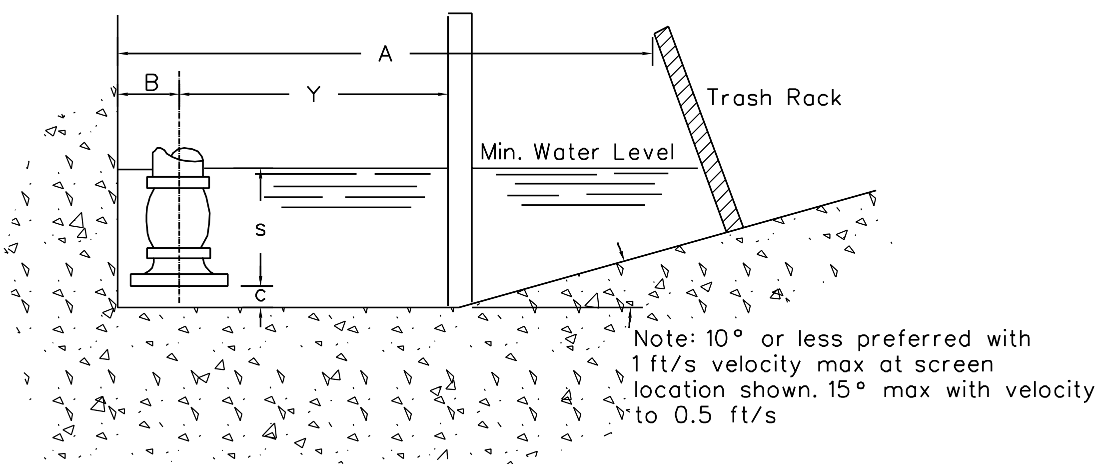
TABLE 14-2 — Rectangular Sump Dimensions

Dimension	Description	Recommended Value
A	Distance from centerline of pump inlet bell/volute to sump entrance	5D
A	Length of constricted bay section at pump	2.5D
B	Clearance from back wall to centerline of pump inlet bell/volute	0.75D
C	Clearance between pump inlet bell and sump floor	0.3D – 0.5D
D	Outside diameter of pump inlet bell/volute	$D = 1.128\sqrt{Q/V}$
H_{min}	Minimum water depth in sump	S + C
H	Minimum height of constructed bay section	Greater of H or 2.5D
S	Minimum pump inlet bell submergence	See Figure 14-3
W	Pump inlet bay width	≥2D
W	Minimum width of constricted bay section	2D
X	Pump inlet bay length	≥5D
Y	Distance from centerline of pump inlet bell/volute to screen	≥4D
Z_1	Distance from centerline of pump inlet bell/volute to diverging walls	≥5D
Z_2	Distance from centerline of pump inlet bell/volute to sloping floor	≥5D
α	Angle of floor slope	0 - 10°
β	Angle of wall convergence	0 - 10°
ϕ	Angle of convergence from constricted area to pump bay walls	10° max.

Source: HEC 24 *(4)*.



1. Plan View



2. Elevation View

FIGURE 14-11 — Sump Dimensions, Plan and Elevation View, Wet-Pit Type Pumps

Having roughly estimated the volume of storage required and the trial pumping rate by the approximate methods described in the preceding Sections, the configuration and elevations of the storage chamber can be initially set. Knowing this geometry, the volume of water stored can be calculated for its respective depth. In addition to the wet-pit, storage will also be provided by the inflow pipes and exterior storage if the elevation of water in the wet-pit is above the inflow invert. The calculation of storage in a circular pipe and a circular wet well as shown in Figures 14-12, 14-13 and 14-14 is illustrated in the following Example.

Example: It is necessary to develop a stage-storage curve for a pump station with a 21-ft diameter wet well and a 48-in inlet pipe on a 0.4% slope. The outlet invert elevation of the pipe is set at an elevation of 0.0 ft, which is also the stop elevation of the pumps. As such, there is no available storage to be calculated below this elevation.

The first increment of depth in this Example is set at 0.5 ft. The volume of the wet well at this depth is calculated first and is simply the end area, πr^2, times the depth:

V_1 = $(\pi)(r^2)(D)$
V_1 = wet well volume, ft^3
r = radius of wet well, ft
D = depth of water, ft

$V_1 = (\pi)(10.5^2)(0.5) = 173\ ft^3$

The volume of the inlet pipe at a depth of 0.5 ft is then calculated using the ungula of a cone formula shown in Figure 14-12:

V_2 = $L((0.67)(a^3) \pm (c)(B))/(r \pm c)$
L = 0.5/0.004 = 125 ft
r = 24 in = 2 ft
c = depth of water minus r = 0.5 – 2 = -1.5 ft
a = $(r^2 - c^2)^{1/2} = (2^2 - 1.5^2)^{1/2} = 1.32$ ft

The base area, B, of the ungula can be determined with the aid of the King and Brater table shown in Figure 14-12:

$B = (C_a)(d^2)$

Enter table with value of D/d:

D/d = 0.5/4 = 0.125

C_a is then read from the Table:

C_a = 0.0567; this is determined by interpolating between 0.0534 and 0.06.

Then, $B = (0.0567)(4^2) = 0.91\ ft^2$

The Equation for ungula volume can then be solved with the above variables:

$V_2 = 125((0.67)(1.32^3) - (1.5)(0.91))/(2 - 1.5) = 44\ ft^3$

Ungula Volume: $V_3 = L(0.67a^3 \pm cB) / (r \pm c)$

If base is greater than a semicircle, use + sign. If base is less than a semicircle, use – sign.

where:
- L = length of ungula, ft
- r = radius of base, ft
- B = area of base, ft^2
- c = vertical distance between water surface and center of base, ft
- a = ½ width of water surface at base, ft

May use King & Brater table, for determining area, B, of the cross section of a circular conduit flowing part full.

Let (Depth of water)/(Diameter of channel) = D/d and C_a = the tabulated value.

Then, $B = C_a d^2$

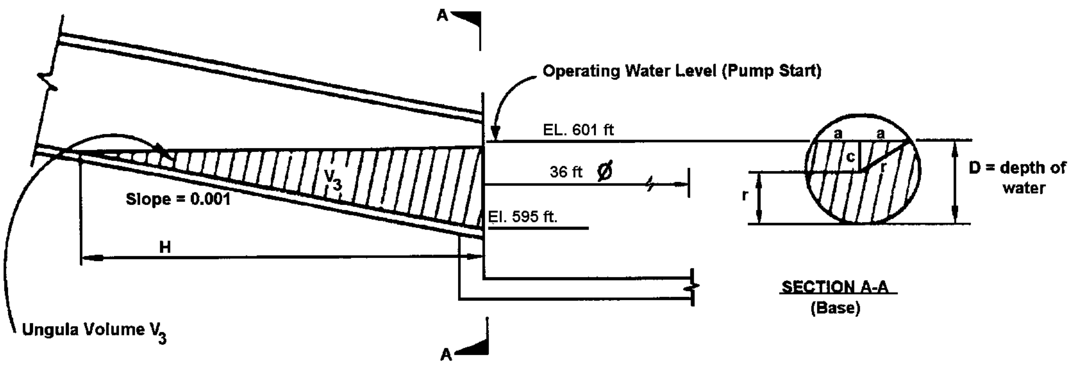


FIGURE 14-12(a) — Storage Volume in Ungula

D/d	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0.0	0.0000	0.0013	0.0037	0.0069	0.0105	0.0417	0.0192	0.0242	0.0294	0.0350
0.1	0.0409	0.0470	0.0534	0.0600	0.0668	0.0739	0.0811	0.0885	0.0961	0.1039
0.2	0.1118	0.1199	0.1281	0.1365	0.1449	0.1535	0.1623	0.1711	0.1800	0.1890
0.3	0.1982	0.2074	0.2167	0.2260	0.2355	0.2450	0.2546	0.2642	0.2739	0.2836
0.4	0.2934	0.3032	0.3130	0.3229	0.3328	0.3428	0.3527	0.3627	0.3727	0.3827
0.5	0.393	0.403	0.413	0.423	0.433	0.443	0.453	0.462	0.472	0.482
0.6	0.492	0.502	0.512	0.521	0.531	0.540	0.550	0.559	0.569	0.578
0.7	0.587	0.596	0.605	0.614	0.623	0.632	0.640	0.649	0.657	0.666
0.8	0.674	0.681	0.689	0.697	0.704	0.712	0.719	0.725	0.732	0.738
0.9	0.745	0.750	0.756	0.761	0.766	0.771	0.775	0.779	0.782	0.784

Source: King and Brater Table *(6)*.

FIGURE 14-12(b) — Values of C_a for Determining Area of Cross Section of a Circular Conduit Flowing Part Full

Stage-Storage Tabulation
48-in Pipe at 0.40%, 21-ft Diameter Wet Well

Elevation (ft)	Pipe (ft^3)	Wet Well (ft^3)	Total (ft^3)
0	0	0	0
0.5	44	173	217
1.0	240	346	586
1.5	641	520	1161
2.0	1271	693	1964
2.5	2115	866	2981
3.0	3056	1039	4095
3.5	4006	1212	5218
4.0	4883	1385	6268
4.5	5567	1559	7126
5.0	6018	1732	7750
5.5	6257	1905	8162
6.0	6329	2078	8407
6.5	6330	2251	8581
7.0	6330	2425	8755

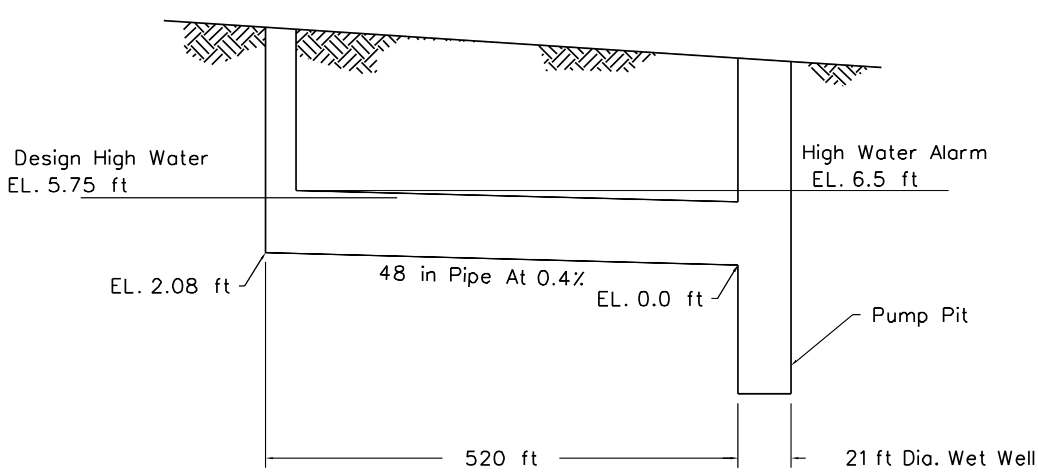


FIGURE 14-13 — Storage Pipe Sketch and Tabulation of Stage-Storage Relationship

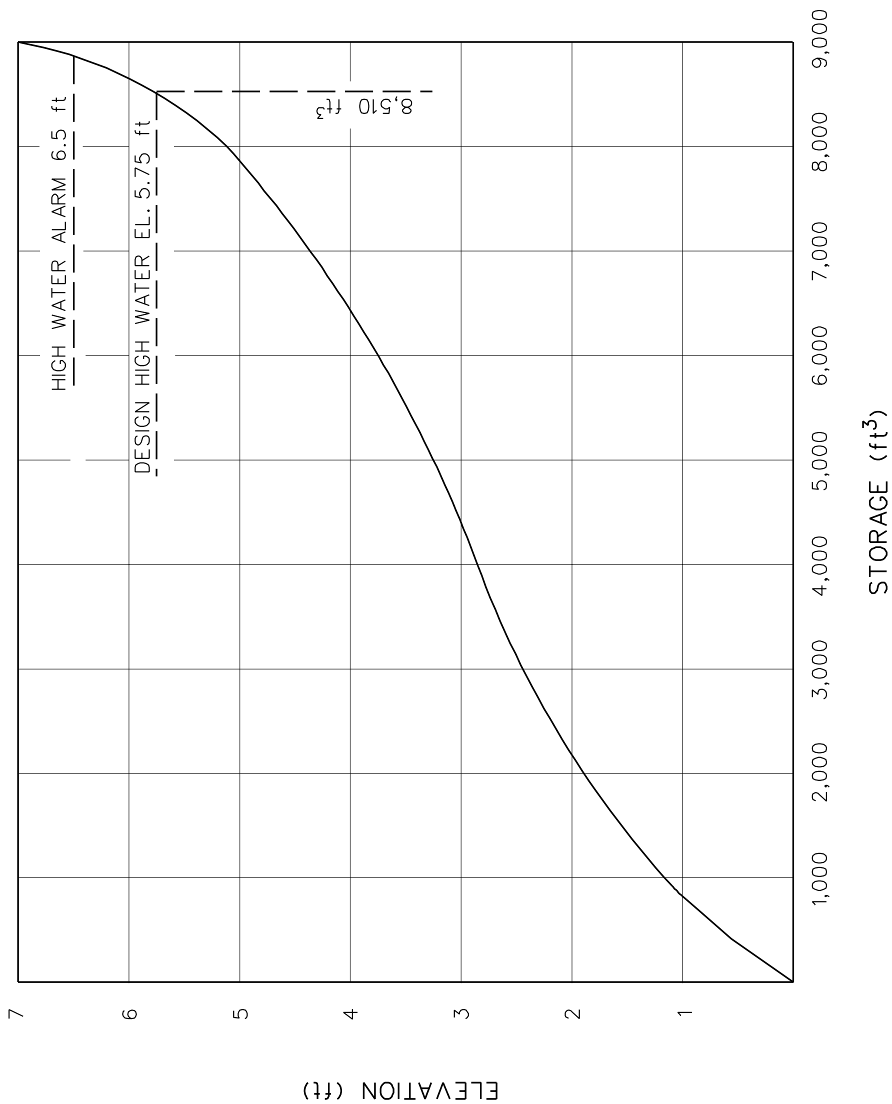


FIGURE 14-14 — Stage-Storage Curve

The total volume of available storage for D = 0.5 ft is the summation of the volume of the wet well and the ungula volume in the inlet pipe:

$$V_t = V_1 + V_2 = 173 + 44 = 217 \text{ ft}^3$$

The procedure is then repeated for additional increments of depth, and the results can be tabulated as shown in Figure 14-13. A storage-verses-elevation curve can then be plotted (Figure 14-14), and storage below any elevation can readily be obtained.

Step 6 Pump Cycling and Usable Storage

One of the basic parameters addressed initially was that the proper number of pumps must be selected to deliver the design Q. Also, the correct elevations must be chosen to turn each pump on and off. Otherwise, rapid cycling (frequent starting and stopping of pumps) may occur causing undue wear and possible damage to the pumps.

Before discussing pump cycling calculations, operation of a pump station will be described. Initially, the water level in the storage basin will rise at a rate depending on the rate of the inflow and physical geometry of the storage basin. When the water level reaches the stage designated as the first pump start elevation, the pump will be activated and discharge water from storage at its designated pumping rate. If this rate exceeds the rate of inflow, the water level will drop until it reaches the first pump-stop elevation. With the pump stopped, the basin begins to refill and the cycle is repeated. This scenario illustrates that the cycling time will be lengthened by increasing the amount of storage between pump on and off elevations. This volume of storage between the first pump on and off elevations minus the volume occupied by the flow being conveyed below normal depth in the inflow pipe is termed usable volume. In theory, the minimum cycle time allowable to reduce wear on the pumps will occur when the inflow to the usable storage volume is one-half the pump capacity. Assuming this condition, cycling time can be related to usable volume.

For a given pump operating alone with a capacity Q_p, cycling will be a maximum (least time between starts) when the inflow Q_i to the usable storage is one-half the pump capacity. The proof is as follows:

t = time between starts
t = (time to empty) + (time to fill usable storage volume), V_t

$$t = V_t/(Q_p - Q_i) + V_t/Q_i \qquad (14.2)$$

where: $Q_i = Q_p/2$, ft^3/s
$t = 4V_t/Q_p$, s

or t in minutes: $t = 4V_t/60Q_p = V_t/15Q_p$

where:
t = minimum cycle time, min
V_t = minimum required volume for pump cycling, ft^3
Q_p = individual pump rate, ft^3/s

The time in minutes for all pumps is computed as follows:

$t_{min} = V_{min} / 15Q_p$

where: V_{min} = minimum required cycle volume, ft^3

Generally, the minimum allowable cycling time, t, is designated by the pump manufacturer based on electric motor size. In general, the larger the motor, the larger the starting current required, the larger the damaging heating effect and the greater the cycling time required. The pump manufacturer should always be consulted for allowable cycling time during the final design phase of project development. However, the following limits may be used for estimating allowable cycle time during preliminary design:

Motor kW	Cycling Time (t), min
0 – 11	5.0
15 – 22	6.5
26 – 45	8.0
48 – 75	10.0
112 – 149	13.0

Knowing the pumping rate and minimum cycling time, the minimum necessary allowable storage, V, to achieve this time can be calculated by:

$$V = 15Q_pt \qquad (14.3)$$

Having selected the trial wet-pit dimensions, the pumping range, Δh, can then be calculated. The pumping range represents the vertical height between pump-start and pump-stop elevations. Usually, the first pump-stop elevation is controlled by the minimum recommended bell submergence criteria specified by the pump manufacturer or the minimum water level, H, specified in the design. The first pump-start elevation will be a distance, Δh, above H.

Where the only storage provided is in the wet pit, the pumping range can be calculated by dividing the allowable storage volume by the wet pit area:

$$\Delta h = V/\text{wet pit area} \qquad (14.4)$$

Where larger volumes of storage are available, the initial pump-start elevations can be selected from the stage-storage curve. Because the first pump turned on should typically have the ability to empty the storage facility, its turn-off elevation would be the bottom of the storage basin. The minimum allowable storage would be calculated by the equation $V=15Q_pt$. The elevation associated with this volume in the stage-storage curve would be the lowest turn-on elevation that should be allowed for the starting point of the first pump. The second and subsequent pump-start elevations will be determined by plotting the pump performance on the mass inflow curve.

This distance between pump starts may be in the range of 1 ft to 3 ft for stations with a small amount of storage and 0.25 ft to 0.5 ft for larger storage situations.

Step 7 Mass Curve Routing

The procedures described thus far will provide all necessary dimensions, cycle times, appurtenances, etc., to design the pump station. A flood event can be simulated by routing the design inflow hydrograph through the pump station using a mass curve routing. In this way, the performance of the pump station can be observed at each hydrograph time increment and the pump station design evaluated. Then, if necessary, the design can be “fine-tuned.” It may be necessary to recalculate this Step after pump selection to verify system performance.

As the stormwater flows into the storage basin, it will accumulate until the first pump-start elevation is reached. The first pump is activated and, if the inflow rate is greater than the pump rate, the stormwater will continue to accumulate until the second pump-start elevation is reached. As the inflow rate decreases, the pumps will shut off at their respective pump-stop elevations.

These conditions are modeled in the mass curve diagram by establishing the point at which the cumulative flow curve has reached the storage volume associated with the first pump-start elevation. This storage volume is represented by the vertical distance between the cumulative flow curve and the base line. A vertical storage line is drawn at this point because it establishes the time at which the pump first starts.

The pump discharge line is drawn from the intersection of the vertical storage line and the base line upwards toward the right; the slope of this line is equal to the discharge rate of the pump. The pump discharge curve represents the cumulative discharge from the storage basin, while the vertical distance between the inflow mass curve and the pump discharge curve represents the amount of stormwater stored in the basin.

If the rate of inflow is greater than the pump capacity, the inflow mass curve and the pump discharge curve will continue to diverge until the volume of water in storage is equal to the storage associated with the second pump-start elevation. At this point, the second pump starts, and the slope of the pump discharge line is increased to equal the combined pumping rates.

The procedure continues until peak storage conditions are reached. At some point on the inflow mass curve, the inflow rate will decrease and the slope of the inflow mass curve will flatten. To determine the maximum amount of storage required, a line is drawn parallel to the pump discharge curve and tangent to the inflow mass curve. The vertical distance between the lines represents the maximum amount of storage required.

The routing procedure continues until the pump discharge curve intersects the inflow mass curve. At this point, the storage basin has been completely emptied and a pumping cycle has been completed. As the storm recedes, the pumps will cycle to discharge the remaining runoff.

In developing the pump discharge curve, the designer should remember that the pump's performance curve is sensitive to changes in head and that the static head will fluctuate as the water level in the storage basin fluctuates. The designer should also recognize that the pump discharge rate represents an average pumping rate.

Example: Determine the required storage to reduce the peak flow of 22 ft^3/s to 14 ft^3/s as shown in Figure 14-15. Using the assumed storage pipe shown in Figure 14-16, the stage-storage curve in Figure 14-17, the stage-discharge curve in Figure 14-18 and the inflow hydrograph in Figure 14-15, the storage can be determined.

The inflow mass curve is developed in Figure 14-19. Because 14 ft^3/s was to be pumped, it was assumed that two, 7-ft^3/s pumps would be used. The pumping conditions are as follows:

	Pump-Start Elevation	Pump-Stop Elevation
Pump No. 1 (7 ft^3/s)	2 ft (2,025 ft^3)	0.0 ft (0 ft^3)
Pump No. 2 (7 ft^3/s)	3 ft (4,225 ft^3)	1 ft (600 ft^3)

The numbers in parentheses are the storage volumes associated with the respective elevations.

Figure 14-20 shows the plotting of the pump discharge curve on the inflow mass diagram. Note that the first pump is turned on at approximately Hour 11.4 when a storage volume of 2,010 ft^3 has accumulated. At approximately Hour 11.5, Pump No. 1 has emptied the storage basin, and the pump turns off. At approximately Hour 11.7, the storage volume has again reached 2,010 ft^3, and a pump is turned on. If an alternating start plan had been developed, this would be the second pump that would turn on at this point. If an alternating start plan had not been designed, the first pump would again be started. At approximately Hour 11.8, the volume in storage has increased to 4,235 ft^3, which is associated with a turn-on elevation of 3 ft. Both pumps operate until approximately Hour 12.4 when the volume in the storage basin has been essentially pumped out. The pumps will continue to start and stop until the hydrograph has receded and the inflow stops.

The shaded area between the curves (see Figure 14-21) represents stormwater that is going into storage. Pump cycling at the end of the storm has been omitted to simplify the illustration. When the stored volume remaining is equal to the volume (600 ft^3) associated with the Pump No. 2 stop elevation (1 ft), Pump No. 2 shuts off; Pump No. 1 shuts off when the storage pipe is emptied at the Pump No. 1 stop elevation (0.0 m).

The maximum vertical distance between the pump discharge curve and the inflow mass hydrograph is 8,475 ft^3. This represents the maximum storage required for the reduction of the 22-ft^3/s peak to 14 ft^3/s for the defined conditions; i.e., the storage volume as defined, the start elevations defined and the pump rates defined. The design is adequate because the available storage at the high-water alarm is 8,800 ft^3. It should be noted that a reduction of the starting elevations would have reduced the required storage volume. The designer must make these adjustments on a trial basis until a satisfactory operating condition is developed. Other pumping rates could have also been plotted on the inflow mass curve to determine their performance.

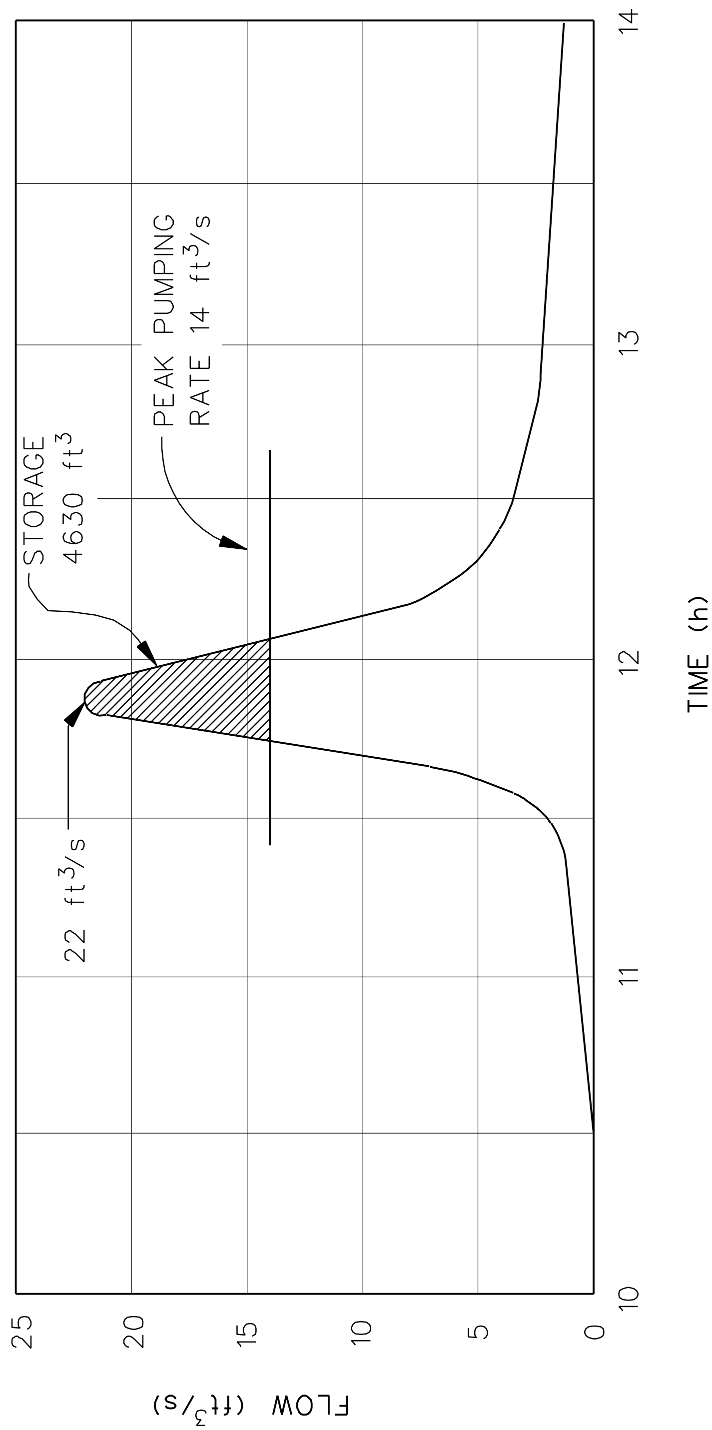


FIGURE 14-15 — Estimating Required Storage

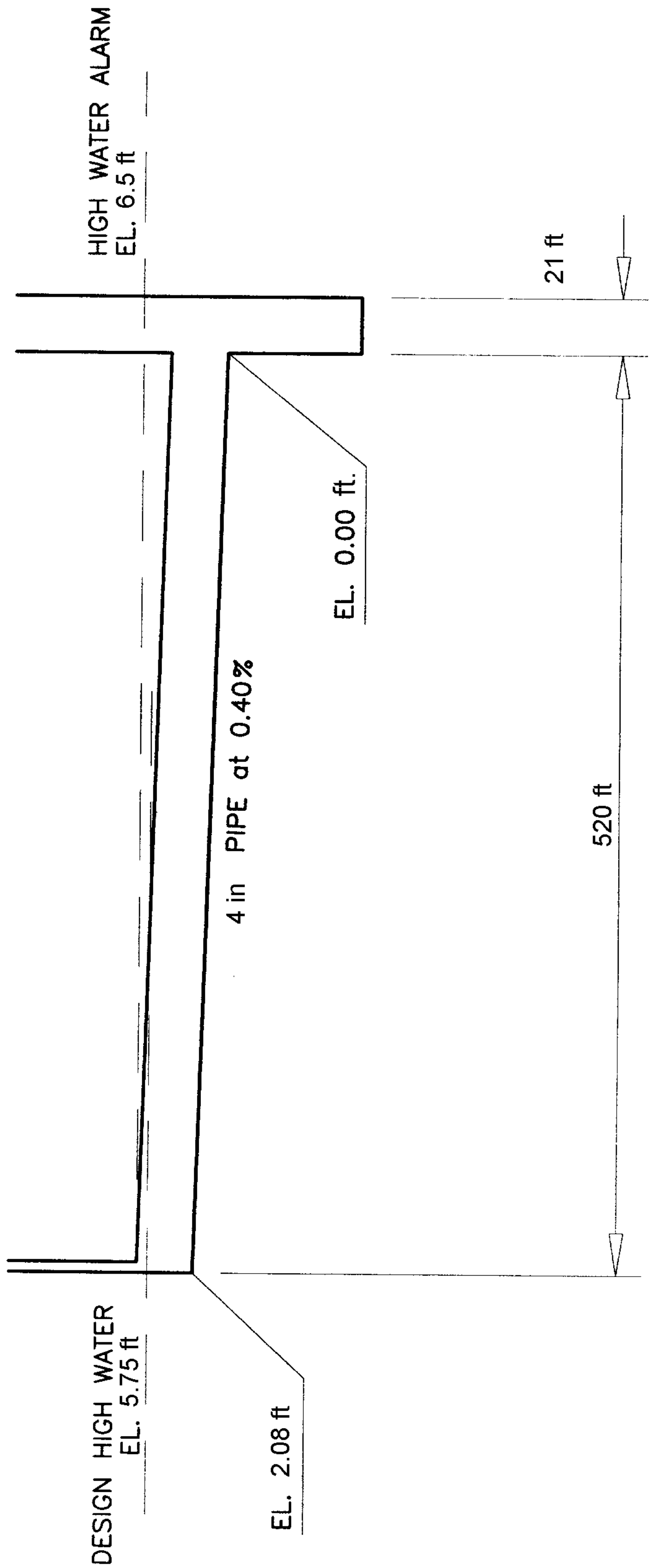


FIGURE 14-16 — Storage Pipe Sketch

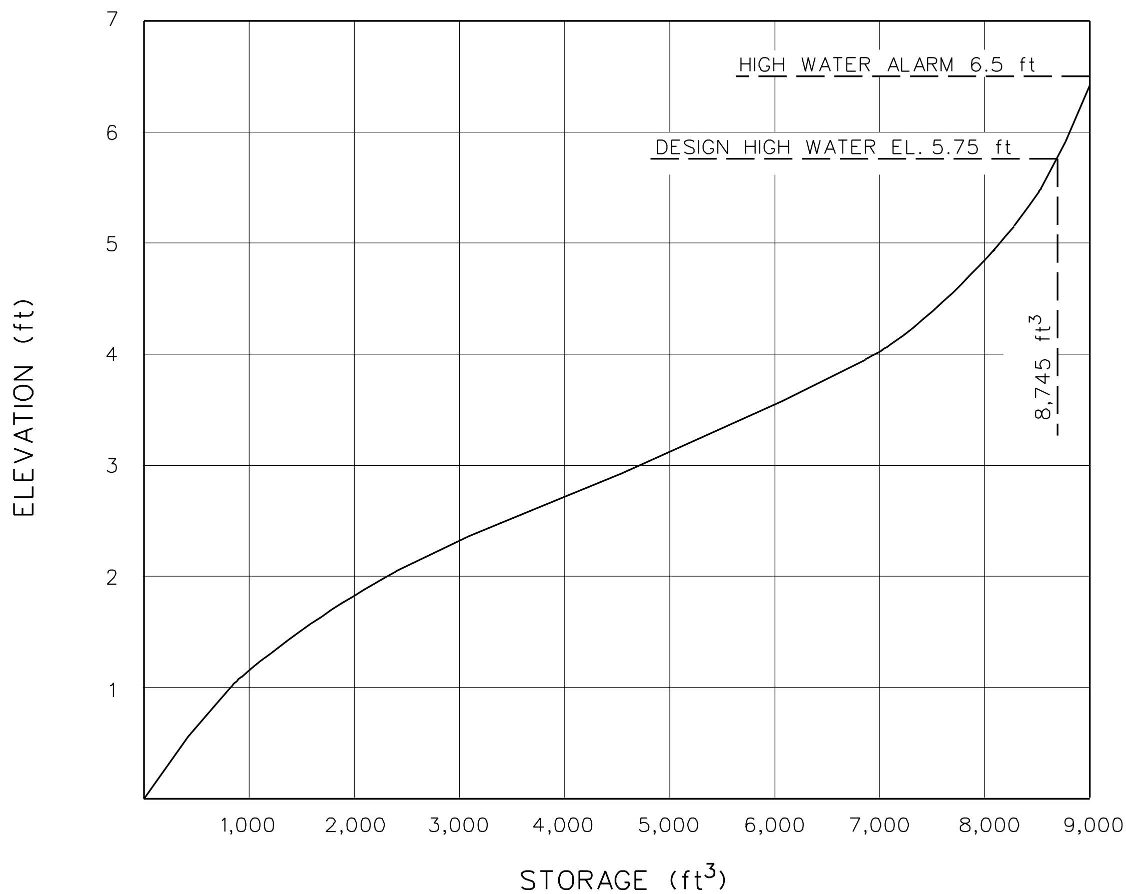

HIGH WATER ALARM 6.5 ft
DESIGN HIGH WATER EL. 5.75 ft
8,745 ft3
ELEVATION (ft)
STORAGE (ft3)
1
2
3
4
5
6
7
1,000
2,000
3,000
4,000
5,000
6,000
7,000
8,000
9,000

FIGURE 14-17 — Stage-Storage Curve

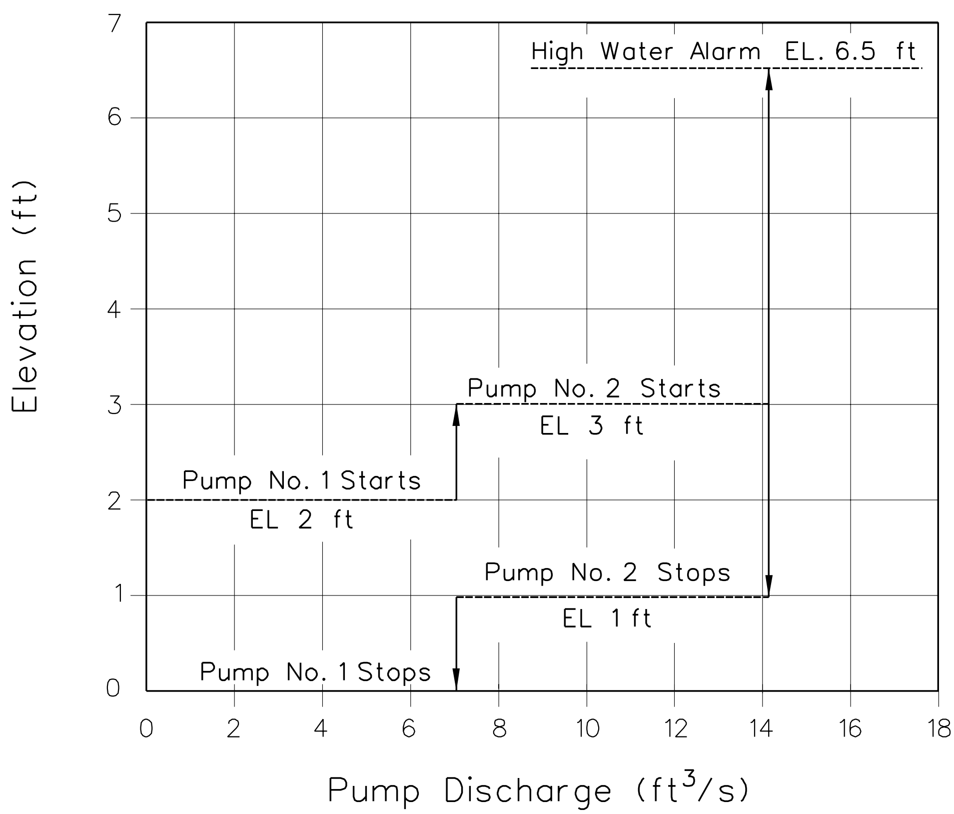


FIGURE 14-18 — Stage-Discharge Curve

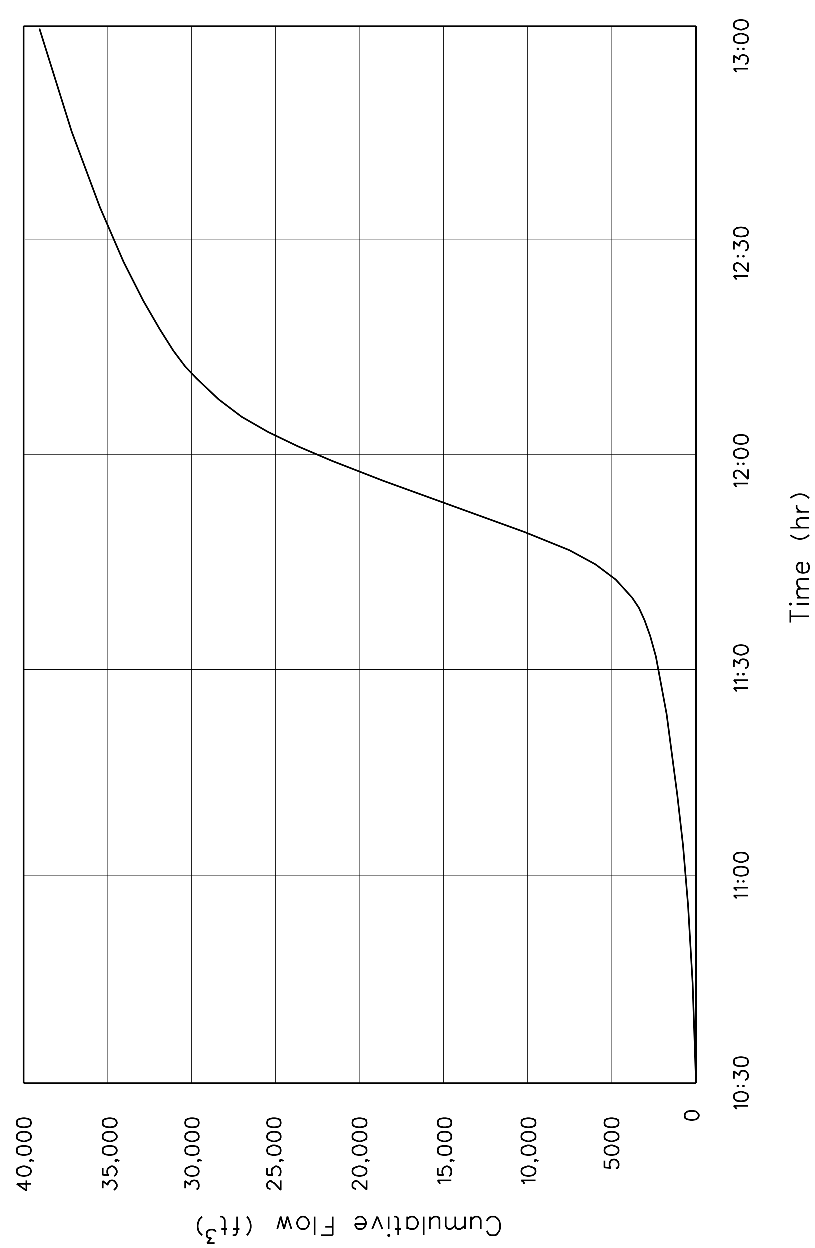


FIGURE 14-19 — Development of Inflow Mass Curve

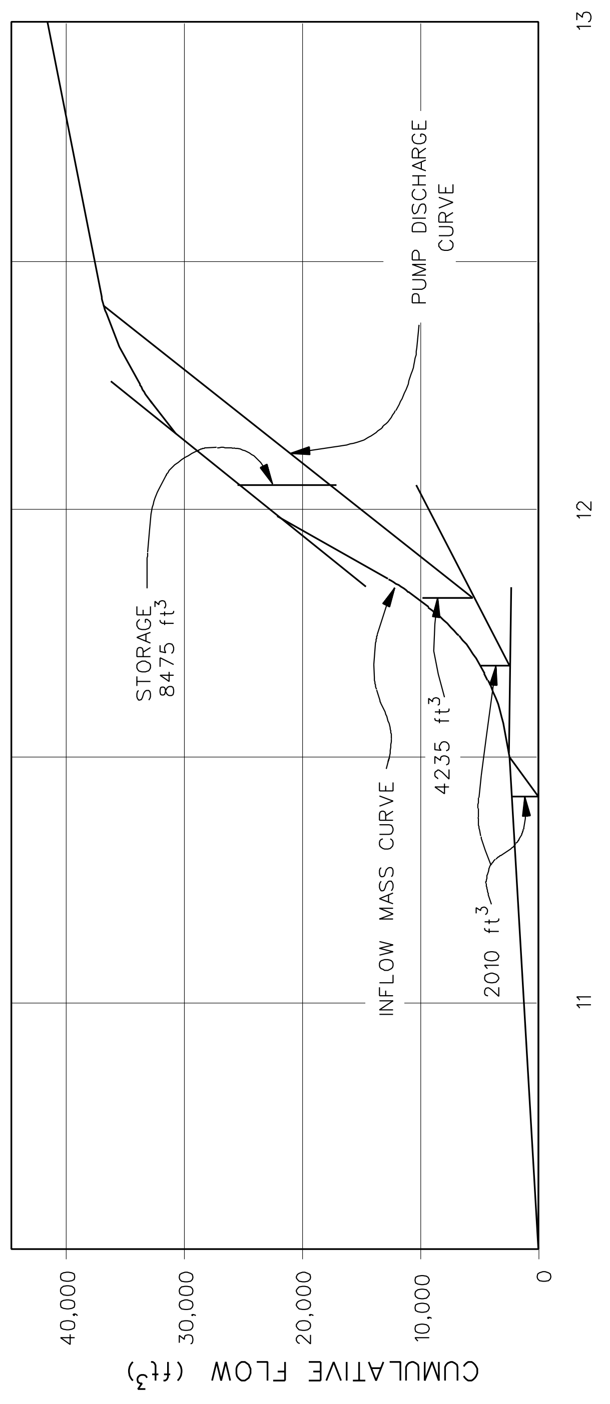


FIGURE 14-20 — Mass Curve Routing Diagram

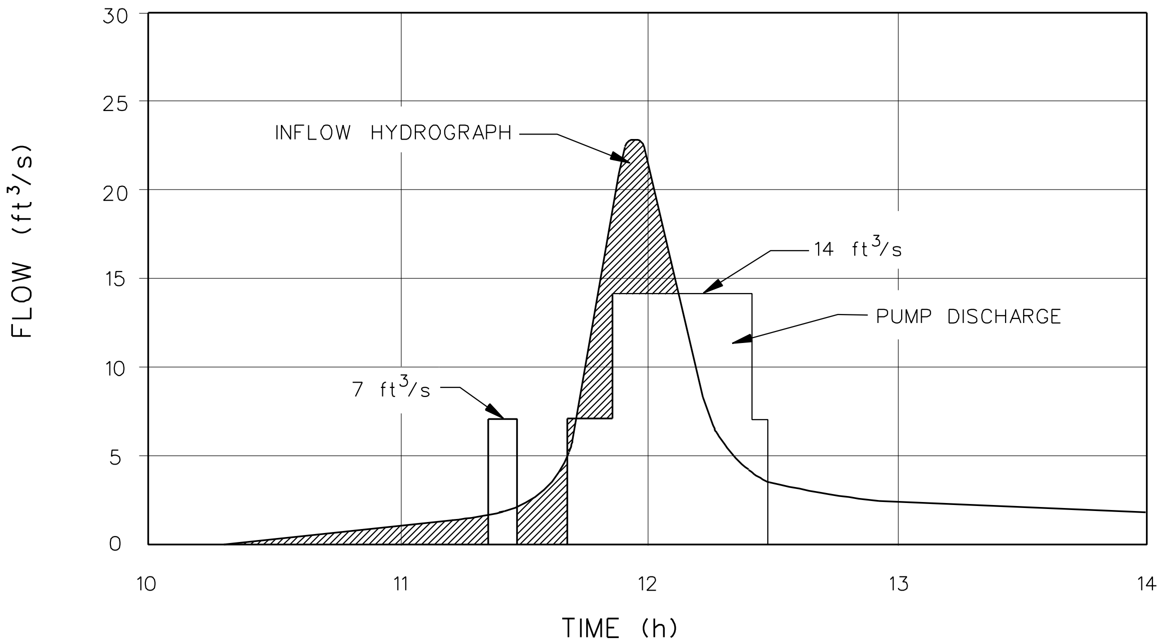


FIGURE 14-21 — Pump Discharge

Once the mass inflow curve has been developed, it is a relatively easy process to try different pumping rates and different starting elevations until a satisfactory design is developed. It should be noted that the number of starts per hour can be determined by looking at the plots on the inflow mass curve. The system should be designed so that the allowable number of starts per hour for the selected pump size is not exceeded.

To aid the designer in visualizing what is happening during the peak design period in this process, the pump discharge curve can be superimposed on the design inflow hydrograph as shown in Figure 14-21.

Step 8 Trial Pumps and Pump Station Piping

The designer must select a specific pump to establish the size of the discharge piping that will be needed. This is done by using information either previously developed or established. Though the designer will not typically specify the manufacturer or a specific pump, a study of various manufacturers' literature is necessary to establish reasonable relationships between total dynamic head, discharge, efficiency and energy requirements. This study will also give the designer a good indication of discharge piping needed, because pumps that produce the desired results will have a specific discharge pipe size.

The discharge line is typically either steel or cast-iron pipe and must be sized to be at least the size of the pump discharge diameter. The need for a larger pipe size required to sustain a maximum velocity of 10 ft/s should be checked by the following equation:

$$d = 1.128(Q/V)^{1/2} \qquad (14.5)$$

where: d = pipe diameter, ft
Q = discharge in pipe, ft^3/s
V = velocity in pipe, ft/s

The length of the discharge pipe must also be determined at this stage based on the station layout. The pump location with respect to the outfall chamber should be set to provide as short a discharge line as possible. Other components of the discharge line that will affect head losses, such as valves, elbows, manifolds, flap gates and other fixtures, should also be identified for their effect on total dynamic head. The discharge line layout should be set to limit the amount of backflow when the pumps shut off and to prevent backwater from the outfall from entering the discharge line.

Step 9 Total Dynamic Head

Total Dynamic Head is the sum of the static head, velocity head and various head losses in the pump discharge system due to friction. Knowing the range of water levels in the storage pit and having a trial pump pit design with discharge pipe lengths and diameters and appurtenances (e.g., elbows, valves) designated, total dynamic head for the discharge system can be calculated. To summarize, the Total Dynamic Head (TDH) is equal to:

$$TDH = H_s + H_f + H_v + H_p \qquad (14.6)$$

where: H_s = static head or height through which the water must be raised, ft
H_f = loss due to friction in the pipe, ft
H_v = velocity head, ft
H_i = loss due to friction in water passing through the pump valves, fittings and other items, ft

The friction loss in the discharge line is normally computed by using the Hazen-Williams Equation as follows:

$$H_l = [(C_u)(V^{1.85})(L)]/[(C^{1.85})(D^{1.165})] \qquad (14.7)$$

where: H_l = friction loss, ft
C_u = unit conversion factor, 3.022
L = length of pipe, ft
V = discharge velocity, ft/s
C = friction factor
D = pipe diameter, ft

The Hazen-Williams Equation should only be used for turbulent flow and is most applicable to water at a temperature of approximately 60°F. The friction factor, C, varies with pipe material and is typically in the range of 60 to 160. A design value of 100 is typical for smooth steel pipe and smooth concrete pipe.

Energy losses through appurtenances such as valves and elbows are determined through the use of a dimensionless loss factor, K, applied to the velocity head as follows:

$$H_l = KV^2/2g \qquad (14.8)$$

where:
- H_l = friction loss through appurtenance, ft
- K = loss factor based on standard data or manufacturer's specified data
- V = velocity through appurtenance, ft/s
- g = acceleration due to gravity, ft/s^2

HEC 24 *(4)* should be consulted for additional information on estimating friction losses in pipe appurtenances.

Step 10 Pump Design Point

Using methods described in Step 8, the Total Dynamic Head of the outlet system can be calculated for a specific static head and various discharges. These TDHs are then plotted vs. discharge. This plot is called a system head curve. A system head curve is a graphical representation of total dynamic head plotted against discharge, Q, for the entire pumping and discharge system. The required design point of a pump can be established after the pump curve is superimposed to give a visual representation of both system and pump (see Figure 14-22). As usually drawn, the system head curve starts from a low point on the Y-ordinate representing the static head at zero discharge. It then rises to the right as the discharge and the friction losses increase. A design point can be selected on the system head curve, and a pump can be selected to match that point. The usual pump curve is the reverse of the system head curve so that the point of intersection is clearly identifiable. One, two or more pump curves can be plotted over the system head curves and conditions examined. If a change of discharge line size is contemplated, a new system head curve for the changed size (and changed head loss) is easily constructed.

Each pump considered will have a unique performance curve that has been developed by the manufacturer, Figure 14-23. More precisely, a family of curves is shown for each pump, because any pump can be fitted with various size impellers. These performance curves are the basis for the pump curve plotted in the system head curves discussed above. The designer must have specific information on the pumps available to be able to specify pumps needed for the pump station. A study of pump performance curves should be completed for each design.

Any point on an individual performance curve identifies the performance of a pump for a specific Total Dynamic Head (TDH) that exists in the system. It also identifies the power required and the efficiency of operation of the pump. It can be seen that for either an increase or decrease in TDH, the efficiency is reduced as the performance moves away from the mid-point of the performance curve. It should also be noted that, as the TDH increases, the power requirement also increases. The designer must make certain that the motor specified is adequate over the full range of TDHs that will exist. It is desirable that the design point be as close to the mid-

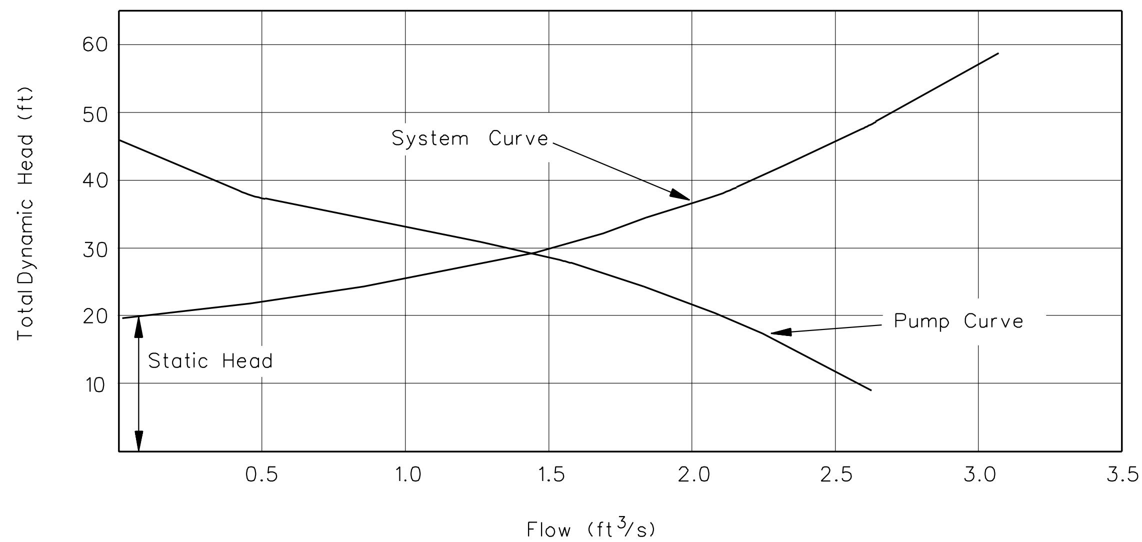
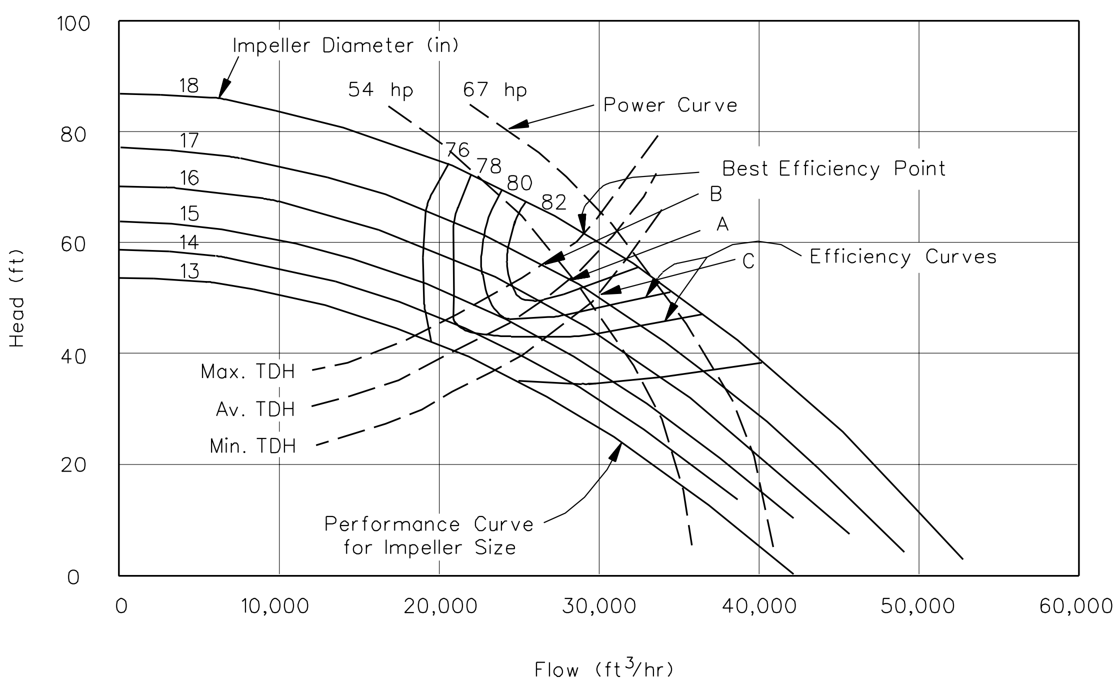


FIGURE 14-22 — System Head Curves (For Single Pump)



Source: Reference *(4)*.

Note: Point A represents the design point. Points B and C represent pump rates at maximum and minimum TDH.

FIGURE 14-23 — Manufacturer's Performance Curves

point as possible, or else to the left of the mid-point rather than to the right of or above it. The range of the pump performance should not extend into the areas where substantially reduced efficiencies exist.

It is necessary that the designer correlate the design point discussed above with an elevation at approximately the mid-point of the pumping range. By doing this, the pump will work both above and below the TDH for the design point and will, thus, operate in the best efficiency range.

Step 11 Power Requirements

To select the proper size of pump motor, compute the energy required to raise the water from its lowest level in the pump pit to its point of discharge. This is best described by analyzing pump efficiency. Pump efficiency is defined as the ratio of pump energy output to the energy input applied to the pump. The energy input to the pump is the same as the driver's output and is called brake power.

$$e = Q\gamma H/550P \tag{14.8}$$

where:
- e = efficiency = pump output/brake power
- Q = pump capacity, ft^3/s
- γ = specific weight of liquid ($62.4\ lb/ft^3 \times 32.2\ ft/s^2$ for cold water)
- H = head, ft
- P = brake power, hp

Efficiency can be segregated into partial efficiencies — hydraulic and mechanical. The efficiency as described above, however, is a gross efficiency used for the comparison of centrifugal pumps. The designer should study pump performance curves from several manufacturers to determine appropriate efficiency ranges. A minimum acceptable efficiency should be specified by the designer for each performance point specified.

To compute the energy required to drive a pump, use the pump efficiency from the manufacturer's performance curves at the established design point. The above equation can then be solved for brake power.

14.7 PHILOSOPHY

Typical pump station design procedures available in the literature do not represent most highway stormwater pump station situations. Many stormwater management plans limit the post-development discharge to that which existed prior to the development. To meet this requirement, it is often necessary to provide storage in the system. Traditional pump design procedures have not considered this storage volume and are thus oriented toward only wet well volumes. These designs are required to pump higher rates with limited storage volumes and, thus, start-stop and cycling relationships are very critical and can consume considerable design effort.

The mass inflow curve procedure discussed in this document is commonly used when significant storage is provided outside of the wet well. The plotting of the performance curve on the mass inflow diagram gives the designer a good graphical tool for determining storage requirements. The procedure also makes it easy to visualize pump start/stop and run times. If a pump failure should occur, the designer can also evaluate the storage requirement and thus the flooding or inundation that could occur.

14.8 REFERENCES

(1) *American National Standard for Pump Intake Design*, ANSI/HI 9.8-1998, Hydraulic Institute, *Engineering Data Book*, 1998.

(2) *American National Standard for Vertical Pumps*, Hydraulic Institute, ANSI/HI 2.1-2.5, 1994.

(3) *Centrifugal Pump Design and Application*, Hydraulic Institute, ANSI/HI 1.1-1.5, 1994.

(4) Federal Highway Administration, *Highway Stormwater Pump Station Design*, Hydraulic Engineering Circular No. 24, NHI-01-007, 2001.

(5) Flygt, "Cost Savings In Pumping Stations."

(6) King, H.W. and Brater, E.F., *Handbook of Hydraulics*, McGraw-Hill, Seventh Edition, 1996.

(7) Los Angeles County Flood Control District, *Design Manual*, 1971.