

Appendix 10-B

Pump Station Hydraulic Design Example

Revised September 2026

Table of Contents

10.B.1 INTRODUCTION.....	10-B-3
10.B.2 PUMP STATION HYDRAULIC DESIGN.....	10-B-3
REFERENCES.....	10-B-27

FIGURES

Figure 10-B-1. Estimating Required Storage.....	10-B-4
Figure 10-B-2. Wet Pit Dimensions, Plan and Section View, Wet-Pit Type Pumps	10-B-6
Figure 10-B-3. Minimum Pump Inlet Bell Submergence	10-B-8
Figure 10-B-5. Storage Volume in Ungula.....	10-B-10
Figure 10-B-6. Example Storage Pipe Sketch.....	10-B-12
Figure 10-B-7. Stage-Storage Curve.....	10-B-13
Figure 10-B-8. Estimated Required Storage, Example Problem	10-B-17
Figure 10-B-9. Stage-Discharge Curve, Example Problem	10-B-18
Figure 10-B-10. Development of Mass Inflow Curve, Example Problem.....	10-B-19
Figure 10-B-11. Mass Curve Routing Diagram, Example Problem	10-B-20
Figure 10-B-12. Pump Discharge, Example Problem.....	10-B-21
Figure 10-B-13. System Head Curves (for Single Pump).....	10-B-24
Figure 10-B-14. Manufacturer's Performance Curves	10-B-25

Tables

Table 10-B-1. Rectangular Sump Dimensions	10-B-7
Table 10-B-2. Stage-Storage Tabulation – 48-inch Pipe at 0.40%, 21-ft Dia. Wet Well	10-B-12
Table 10-B-3. Preliminary Estimated Allowable Cycle Time.....	10-B-14
Table 10-B-4. Pumping Conditions for Example Problem	10-B-17

If you require assistance accessing this information or require it in an alternative format, contact the Michigan Department of Transportation's (MDOT) Americans with disabilities Act (ADA) coordinator at Michigan.gov/MDOT-ADA.

10.B.1 Introduction

The following systematic procedure integrates the hydraulic design variables involved in wet pit design. It incorporates the recommended design criteria to determine the required number and capacity of pumps and the required wet well and storage dimensions.

Theoretically, an infinite number of pump station designs are possible for a given site. To initiate a design, constraints are identified and a trial design evaluated to meet these constraints. Routing the inflow hydrograph through the trial pump station determines the effectiveness of the initial design. Initial results are used to refine the initial and subsequent designs into a more effective final design through an iterative process.

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

- Inflow hydrograph
- Storage capacity of the wet well and outside storage
- Discharge rate of the pumping system

The inflow hydrograph is determined by the watershed's physical factors and regional climate factors. The discharge of the pump station is controlled by local regulations or capacity limitations of the receiving system. Therefore, the main objective in pump station design is to store enough inflow (volume of water under the inflow hydrograph) to allow the discharge to meet specified limits. Even if there are no physical limitations to pump station discharge, upstream storage can be added to allow for smaller and/or fewer pumps.

10.B.2 Pump Station Hydraulic Design

The procedure for pump station design is illustrated in the following 11 steps:

Step 1 – Inflow to Pump Station

Develop an inflow hydrograph to the pump station using a rainfall-runoff hydrologic model such as USDA WinTR55 or the EGLE Computing Flood Discharges in Small Ungauged Watersheds (dependent on the time of concentration). For more details, refer to the procedures presented in Chapter 3, Hydrology.

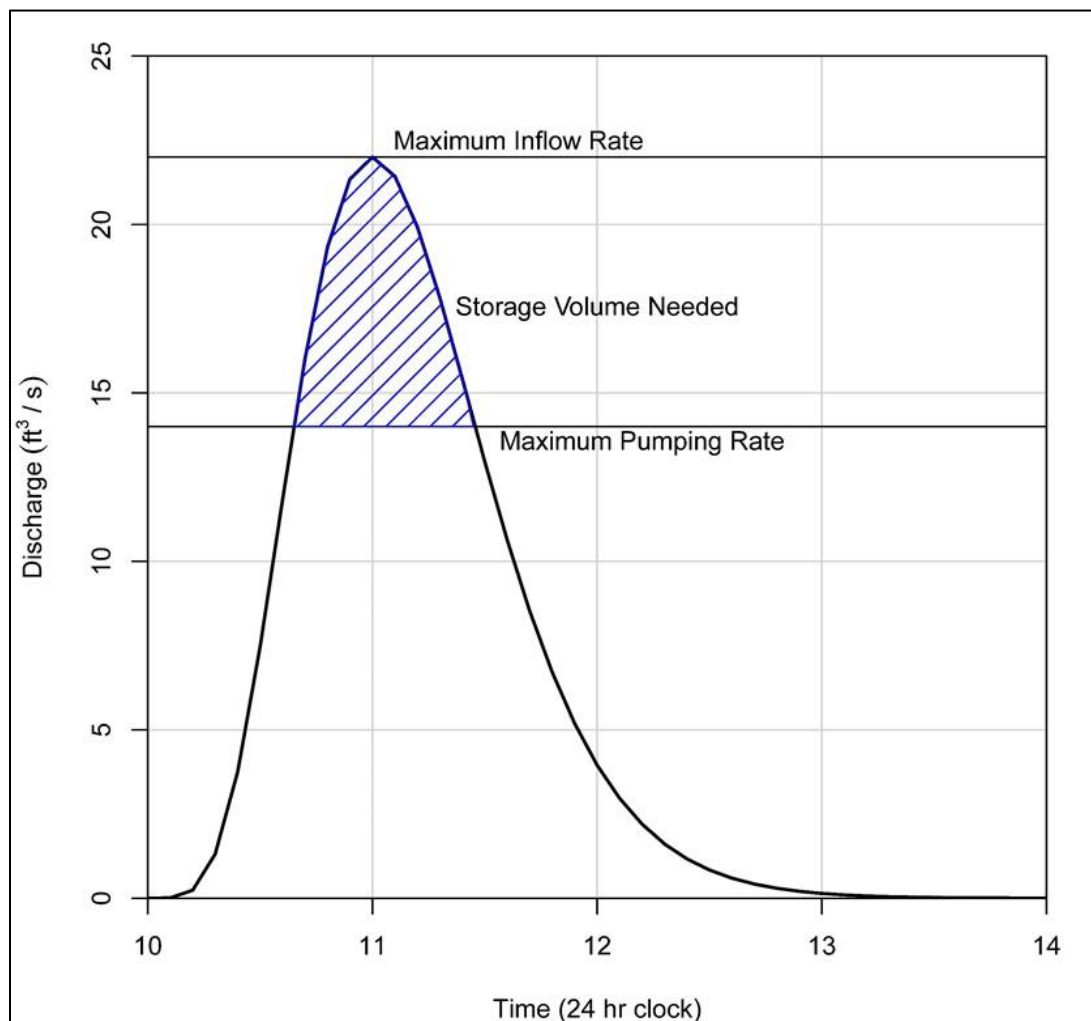
Step 2 – Estimate Pumping Rate, Volume of Storage, and Number of Pumps

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

Some approximation of all three parameters (pumping rates, storage, and pump on-off settings) is necessary to produce the first trial design. One approach is shown in **Figure 10-B-1**. In this approach, assign a maximum pumping rate and draw a horizontal line representing the maximum inflow rate across the top of the hydrograph. The shaded area above the maximum pumping rate represents an estimated volume of storage required above the last pump turn on point. Measure the shaded area to give an estimated starting size for the storage facility. Use the preliminary estimated storage facility to establish shape, size, depth, etc., to match the site and develop a stage-storage relationship.

The maximum pumping rate is established by stormwater management limitations, capacity of the receiving system, the desirable pump size, or available storage. A minimum of two pumps is required (MDOT prefers a minimum of three pumps). However, as many as five pumps may be needed for large drainage areas such as a continuously depressed highway. The pump station size and number of pumps are controlled by physical constraints, such as portable standby power., can define the pump station size and number of pumps.

Figure 10-B-1. Estimating Required Storage (FHWA HEC-22, 2024)



Step 3 – Set Allowable High-Water (AHW) Elevation

The AHW elevation in the pump station is set such that the water surface elevation (HGL) at the lowest inlet in the collection system provides a minimum 1 foot (2 feet desirable) of freeboard below the roadway grade for the design storm. The lower the elevation, the more conservative the design.

Step 4 – Determine Pump Pit Dimensions

Determine the minimum required plan dimensions for the pump station from manufacturer's literature or from dimensioning guides such as those provided by FHWA HEC-24 (2001) (refer to **Figure 10-B-2** and **Table 10-B-1**). The pit dimensions are determined by the selected number and spacing of pumps on the floor plan. Consider clearances needed for pump intake system design. Also consider clearances around electrical panels and other associated equipment that will be housed in the pump station building.

Figure 10-B-2. Wet Pit Dimensions, Plan and Section View, Wet-Pit Type Pumps (ANSI/HI 9.8, 2024)

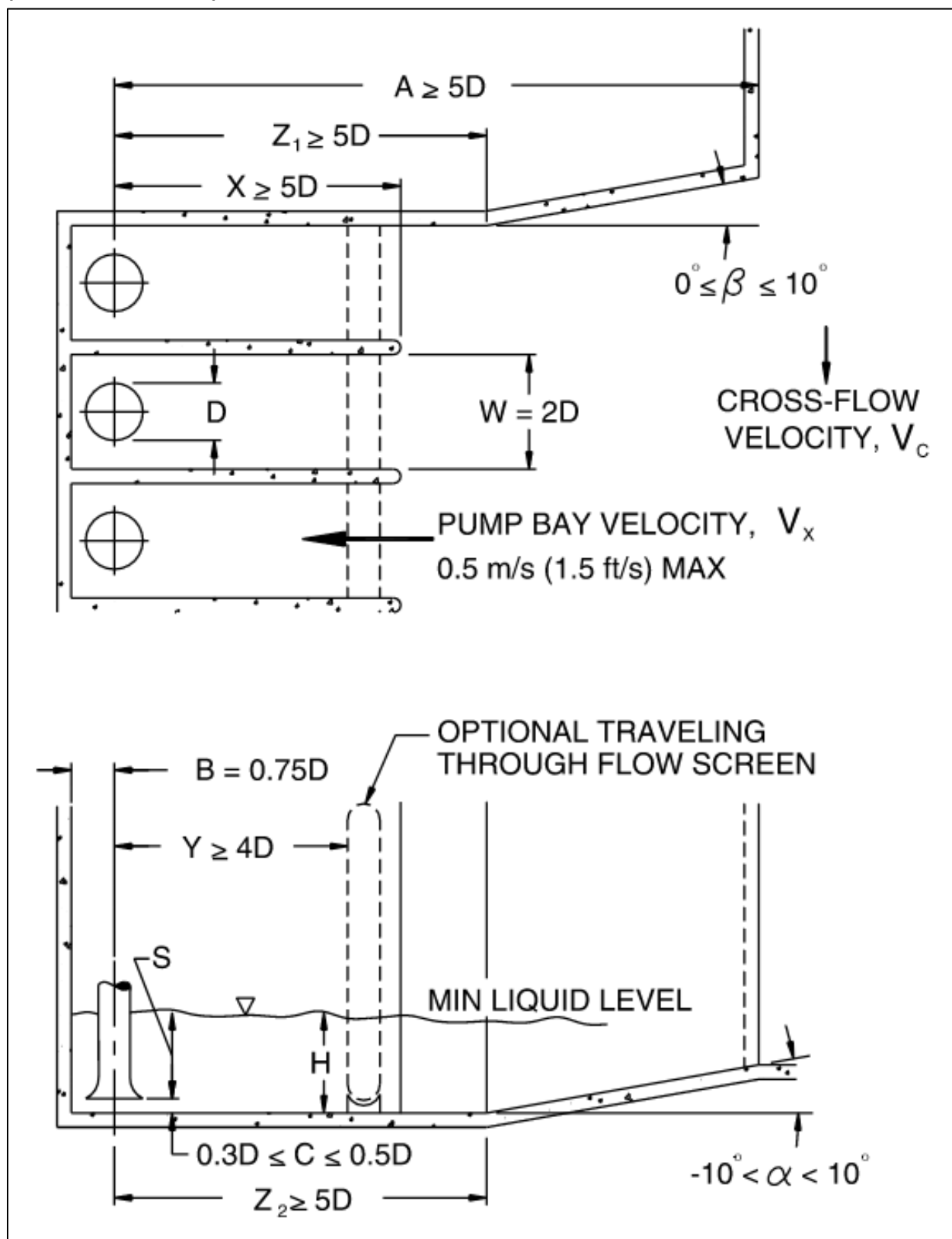
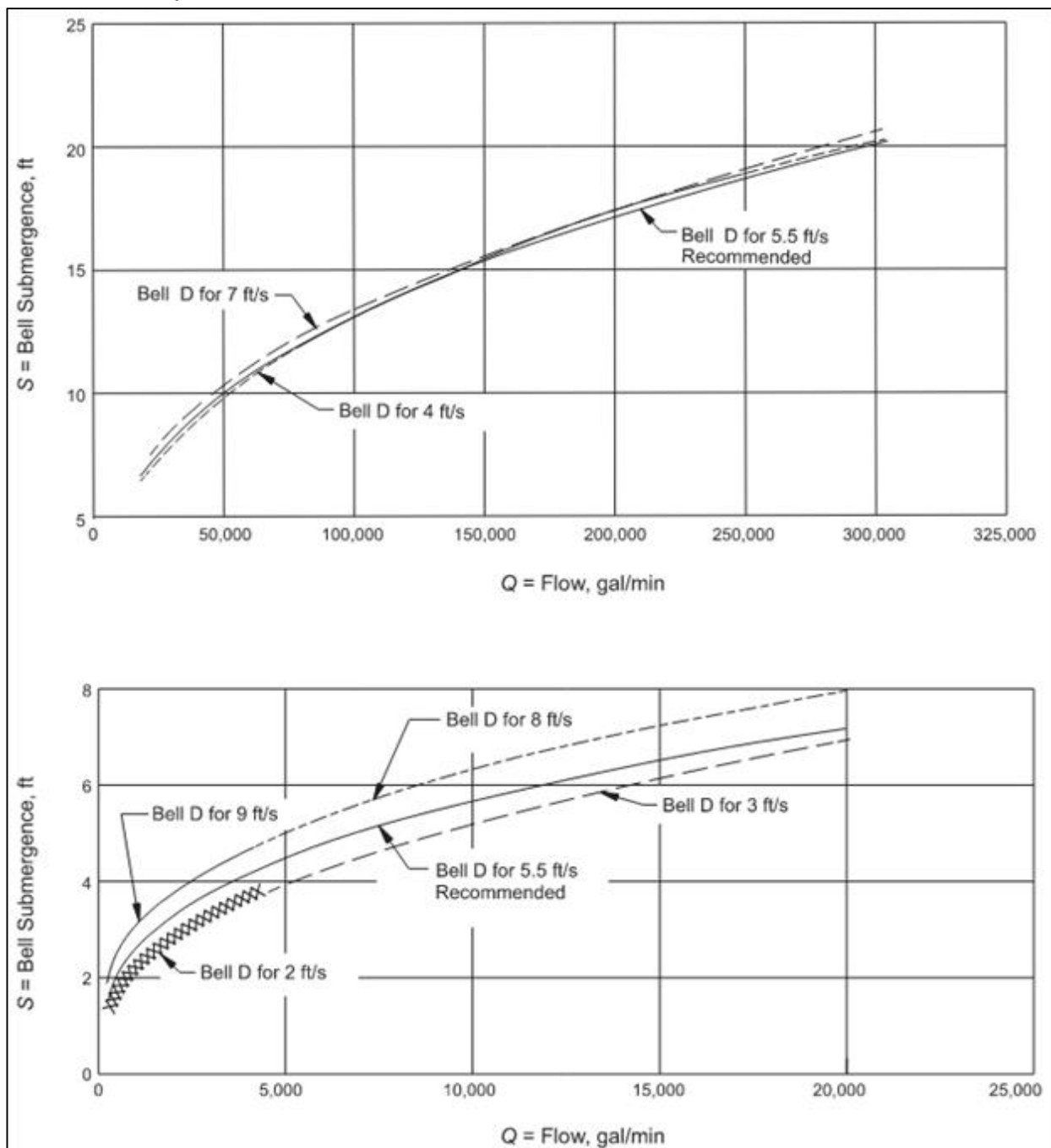


Table 10-B-1. Rectangular Sump Dimensions (FHWA HEC-24, 2001)

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	Based on flow and intake velocity
H_{min}	Minimum water depth in sump	$S + C$
h	Minimum height of constricted bay section	Greater of H or 2.5D
S	Minimum pump inlet bell submergence	Refer to Figure 10-B-3
W	Pump inlet bay width	$\geq 2D$
w	Minimum width of constricted bay section	2D
X	Pump inlet bay length	$\geq 5D$
Y	Distance from centerline of pump inlet bell/volute to screen	$\geq 4D$
Z_1	Distance from centerline of pump inlet bell/volute to diverging walls	$\geq 5D$
Z_2	Distance from centerline of pump inlet bell/volute to sloping floor	$\geq 5D$
α	Angle of floor slope	0 - 10 degrees
β	Angle of wall convergence	0 - 10 degrees
φ	Angle of convergence from constricted area to pump bay walls	10 degrees max.

Figure 10-B-3. Minimum Pump Inlet Bell Submergence (AASHTO Drainage Manual, 2014)

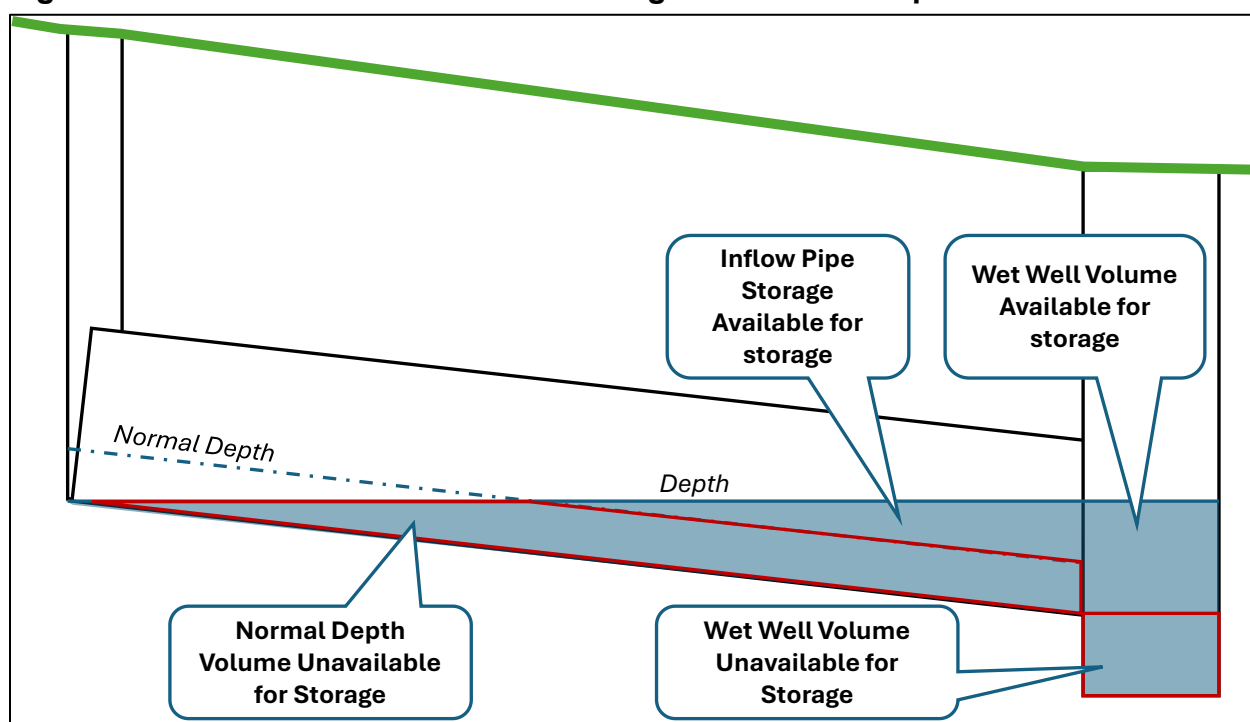


Step 5 – Stage-Storage Relationship

Routing procedures require the development of a stage-storage relationship by calculating the available volume of water for storage at uniform depth intervals.

Using the estimate of the required storage volume and the trial pumping rate, initially set the wet-pit configuration and elevations of the storage chamber. Knowing this geometry, calculate the volume of water stored for its respective depth. In addition to the wet-pit, storage is also provided by the inflow pipes and exterior storage if the elevation of water in the wet-pit is above the normal flow depth in the inflow pipe. The volume between normal depth and the invert in the inflow pipe is unavailable for cycling storage. Available and unavailable storage is shown in **Figure 10-B-4**.

Figure 10-B-4. Volume Available for Storage in an Inflow Pipe and Wet-Pit



The total storage volume is the volume in the upstream pipe and the volume in the wet well. If the upstream storage pipe is circular, calculate the volume using the formula for the volume of a cylinder. Equations for other pipe volumes are available in FHWA HEC-24 (2001). The volume of the wet well is simply the end area times the depth above the lowest pump off level.

The volume of the ungula (refer to **Figure 10-B-5**) is calculated with **Equation 10-B-1**. The downstream cross section of the ungula is called the base. If the base is greater than a semicircle, use a plus (+) sign. If the base is less than a semicircle use a minus (-) sign.

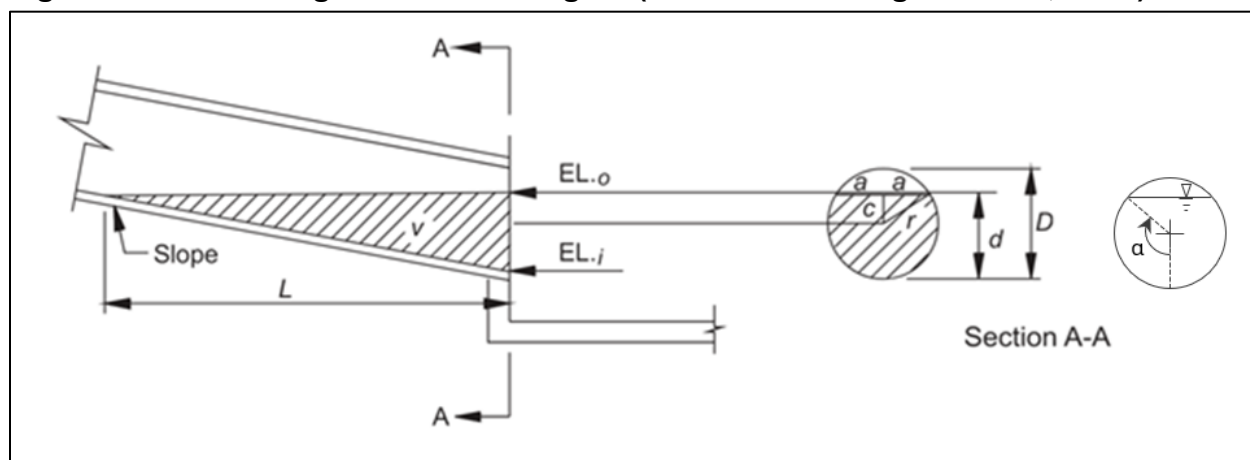
$$V_u = \frac{L(0.67a^3 \pm cA)}{(r \pm c)} \quad (10-B-1)$$

Where:

- V_u = volume of ungula of pipe, cft
- D = Pipe Diameter, ft
- d = Depth of water, ft
- L = $d/(\text{slope of pipe})$, length of ungula, ft
- α = $\cos^{-1} \left(1 - 2 \left(\frac{d}{D} \right) \right)$, angle marked on **Figure 10-B-5**, radians
- B = $D (\sin(\alpha))$, free water surface width, ft
- a = $B/2$, ft
- c = vertical distance between water surface and center of pipe, ft
- A = $(D^2/4) [(\alpha - (\sin(\alpha) \cos(\alpha)))]$, cross-sectional area of the base, sft
- r = radius of pipe, ft

For water depths above the upstream invert, artificially extend the pipe and add and subtract ungula calculated volumes to determine the correct pipe volume. The normal depth volume is subtracted from the calculated pipe volume to determine the storage volume available.

Figure 10-B-5. Storage Volume in Ungula (AASHTO Drainage Manual, 2014)



Example: Step 5 – Stage-Storage Relationship

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. There is no available storage below this elevation. The maximum inflow rate is 22 cfs.

Determine normal depth based on maximum inflow rate and pipe diameter. There is no storage available in the pipe below normal depth. Using FHWA Hydraulic Toolbox, normal depth is 1.45 feet.

In this example, the stage-storage relationship is developed in 0.5 increments. From 0.0 ft to 1.0 ft, the only storage available is in the wet well. At 1.5 ft, storage also becomes available in the pipe.

Calculate the volume of the wet well at 1.5 ft:

$$V_1 = (\pi) (10.5)^2 (1.5)$$

$$V_1 = 520 \text{ sft}$$

Next, calculate the volume of the inlet pipe at a depth of 1.5 ft using **Equation 10-B-1**.

$$V_2 = \frac{L(0.67a^3 \pm cA)}{(r \pm c)}$$

$$L = 1.5 / 0.004 = 375 \text{ ft}$$

$$R = 24 \text{ in.} \sim 2 \text{ ft}$$

$$d = 1.5 \text{ ft}$$

$$D = 4 \text{ ft}$$

$$c = d - r = 1.5 - 2 = -0.5 \text{ ft}$$

$$a = (r^2 - c^2)^{(1/2)} = (4 - 0.25)^{(1/2)} = 1.94 \text{ ft}$$

$$\alpha = \cos^{-1} \left(1 - 2 \left(\frac{d}{D} \right) \right) = \cos^{-1} \left(1 - 2 \left(\frac{1.5}{4} \right) \right) = 1.32 \text{ radians}$$

$$A = (D^2/4) [\alpha - (\sin(\alpha) \cos(\alpha))] = (4^2/4) [(1.32 - (\sin(1.32) \cos(1.32)))]$$

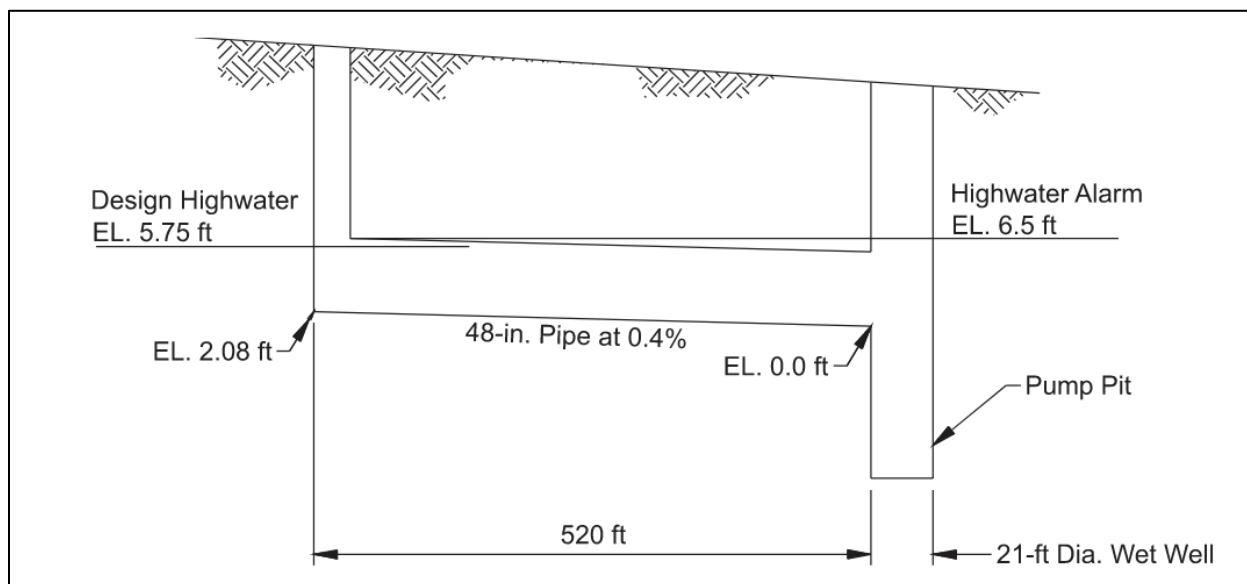
$$= 4.30 \text{ sf}$$

$$V_2 = \frac{375[(0.67 \times 1.94^3) - (0.5 \times 4.30)]}{(2 - 0.5)}$$

$$V_2 = 678.33 \text{ cft}$$

The ungula volume and wet well volumes are added and the normal depth volume is subtracted.

The procedure is repeated for additional increments of depth for the layout shown in **Figure 10-B-6** and the results are tabulated as shown in **Table 10-B-2**. Use the tabulated results to plot a stage-storage curve as shown in **Figure 10-B-7**.

Figure 10-B-6. Example Storage Pipe Sketch (AASHTO Drainage Manual, 2014)**Table 10-B-2. Stage-Storage Tabulation – 48-inch Pipe at 0.40%, 21-ft Dia. Wet Well**

Elevation (feet)	Pipe (ft ³)	Wet Well (ft ³)	Total (ft ³)
0.0	0	0	0
0.5	0	173	173
1.0	0	346	346
1.5	1	520	520
2.0	151	693	1,467
2.5	547	866	2,005
3.0	1,183	1,039	2,634
3.5	2,033	1,212	3,270
4.0	2,934	1,385	4,313
4.5	3,635	1,559	5,188
5.0	4,094	1,732	5,820
5.5	4,334	1,905	6,233
6.0	4,402	2,078	6,474
6.5	4,403	2,251	6,648
7.0	4,403	2,425	6,821

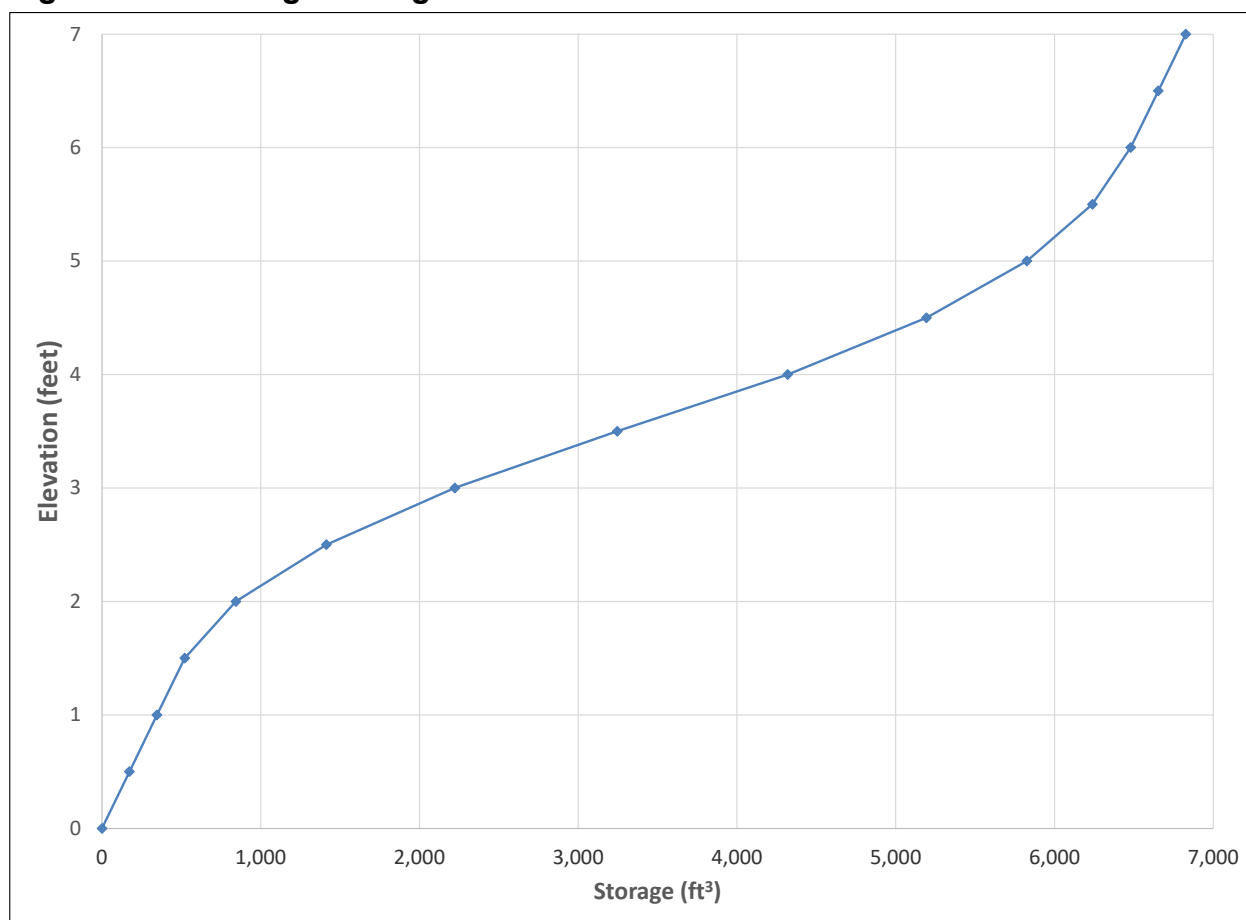
Figure 10-B-7. Stage-Storage Curve

Table 10-B-2, Figure 10-B-6, and Figure 10-B-7 give examples of the calculation and plotting of the storage in a circular pipe and a circular wet-pit. A similar procedure is followed for other storage geometric configurations. Volume in a storage chamber is calculated below various elevations by formulas, depending on the shape of the chamber. A stage-storage curve is plotted and the storage at a specified elevation is obtained from the plot. Select increments of stage that develop a smooth curve without inflection points.

Step 6 – Pump Cycling and Usable Storage

One of the basic pump station parameters addressed initially is selecting the proper number of pumps to deliver the design flow. Also, the correct elevations are 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 and motors.

Before discussing pump cycling calculations, it is important to understand how a pump station operates. Initially, the water level in the wet well rises at a rate depending on the rate of the inflow and physical geometry of the wet well. When the water level reaches the stage designated as the first pump start elevation, the pump is activated and water

is pumped at its design pumping rate. If this rate exceeds the rate of inflow, the water level drops until it reaches the first pump stop elevation. With the pump stopped, the wet well refills and the cycle is repeated. This scenario illustrates that increasing the volume of storage between pump on and off elevations, known as usable volume, lengthens the cycling time. In theory, the minimum cycle time allowable to reduce wear on the pumps occurs when the inflow to the usable storage volume is half of the pump capacity. Assuming this condition, cycling time is related to usable volume.

The pump manufacturer designates the minimum allowable cycling time based on electric motor size. A larger motor requires a longer minimum cycling time because they draw a larger starting current that generates more heat. The longer cycling time allows the motor to cool down before restarting. Consult the pump manufacturer for allowable cycling time during the final design phase of project development. Initial estimates for minimum cycle time are provided in **Table 10-B-3**.

For a given pump with a capacity Q , the cycling frequency is at the maximum (least time between starts) when the inflow, Q_i , is half of the pump capacity. Use **Equation 10-B-2** as follows:

t = cycle time (time between starts)

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

$$t = \frac{V_t}{(Q - Q_i)} + \frac{V_t}{Q_i} \quad (10-B-2)$$

Where:

Q_i = $Q/2$, pump inflow, cfs

t_c = $(4V_t)/Q$, minimum cycle time, sec

t_c = $(4V_t)/60Q = V_t/15Q$, minimum cycle time, min

V_t = minimum required volume for pump cycling, cft

Q = individual pump rate, cfs

Table 10-B-3. Preliminary Estimated Allowable Cycle Time (AASHTO Drainage Manual, 2014)

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

After selecting trial wet-pit dimensions, the pumping range, Δh , is calculated. The pumping range represents the vertical height between pump-start and pump-stop elevations. 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 is a distance, Δh , above H .

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

$$\Delta h = \frac{V}{\text{wet pit area}} \quad (10-B-3)$$

If the total storage volume includes storage outside the wet well, the initial pump-start elevations are selected from the stage-storage curve. The first pump turned on typically can empty the storage facility, therefore its turn-off elevation is the bottom of the storage basin. The minimum required usable storage for each pump is calculated by using **Equation 10-B-3**. The elevation associated with this volume in the stage-storage curve is the lowest turn-on elevation that is allowed for the starting point of the first pump. Plot the pump performance on the mass inflow curve to determine the second and subsequent pump-start elevations.

The distance between pump starts are in the range of 1 to 3 ft for stations with a small wet well area, and 0.25 to 0.5 ft for larger total storage situations. Sensitivity of controls and precision of level measurements require a minimum distance between starts greater than theoretically required.

Step 7 – Mass Curve Routing

The procedures described in Steps 1 through 6 provide the necessary dimensions, cycle times, appurtenances, etc., to design the pump station. Next, the pump station performance is evaluated by routing the design inflow hydrograph through the pump station using a mass curve routing. This step is repeated after pump selection to verify system performance.

- As the stormwater flows into the storage basin, it accumulates until the first pump-start elevation is reached. The first pump is activated and, if the inflow rate is greater than the pump rate of the first pump, the stormwater continues to accumulate until the second pump-start elevation is reached. As the inflow rate decreases, the pumps 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 reaches 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 pump discharge rate. 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 pump discharge curve 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, draw a line 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 is completely empty and a pumping cycle is completed. As the storm recedes, the pumps cycle to discharge the remaining runoff.

The pump's performance curve is sensitive to changes in head and the static head fluctuates as the water level in the storage basin fluctuates. The pump discharge rate represents an average pumping rate.

Example: Step 7 – Mass Curve Routing

Determine the storage required to reduce the peak flow of 22 cfs to 14 cfs as shown in **Figure 10-B-8**. Use the following given information:

- Storage pipe shown in **Figure 10-B-6**
- Stage-storage curve in **Figure 10-B-7**
- Stage-discharge curve in **Figure 10-B-9**
- Pumping conditions in **Table 10-B-4**

Figure 10-B-8. Estimated Required Storage, Example Problem (AASHTO Drainage Manual, 2014)

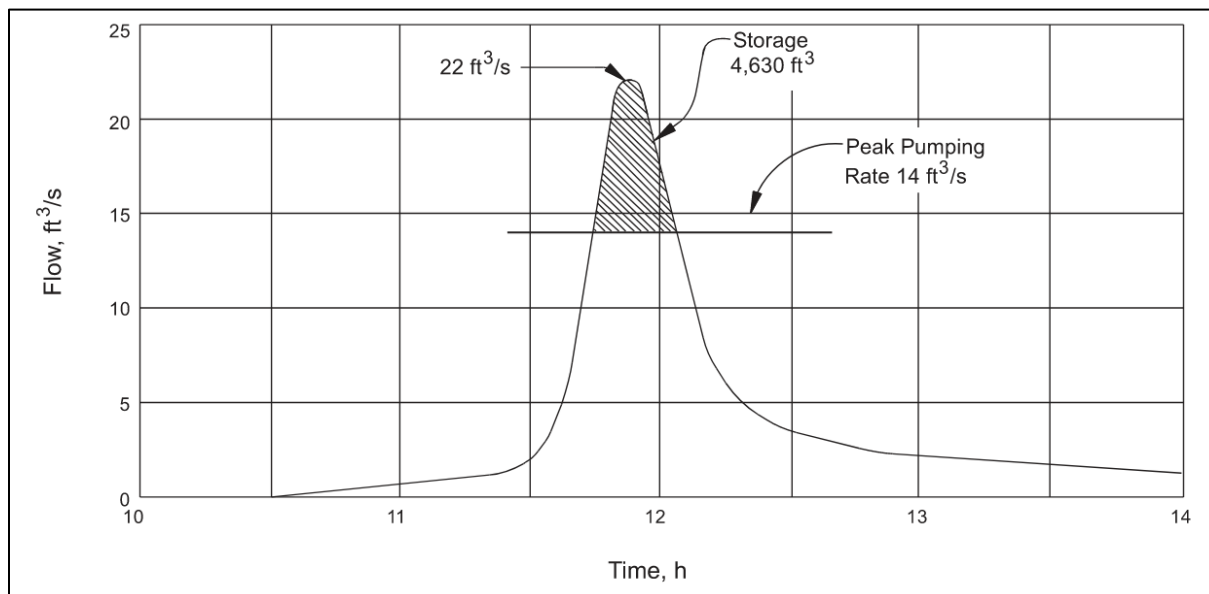
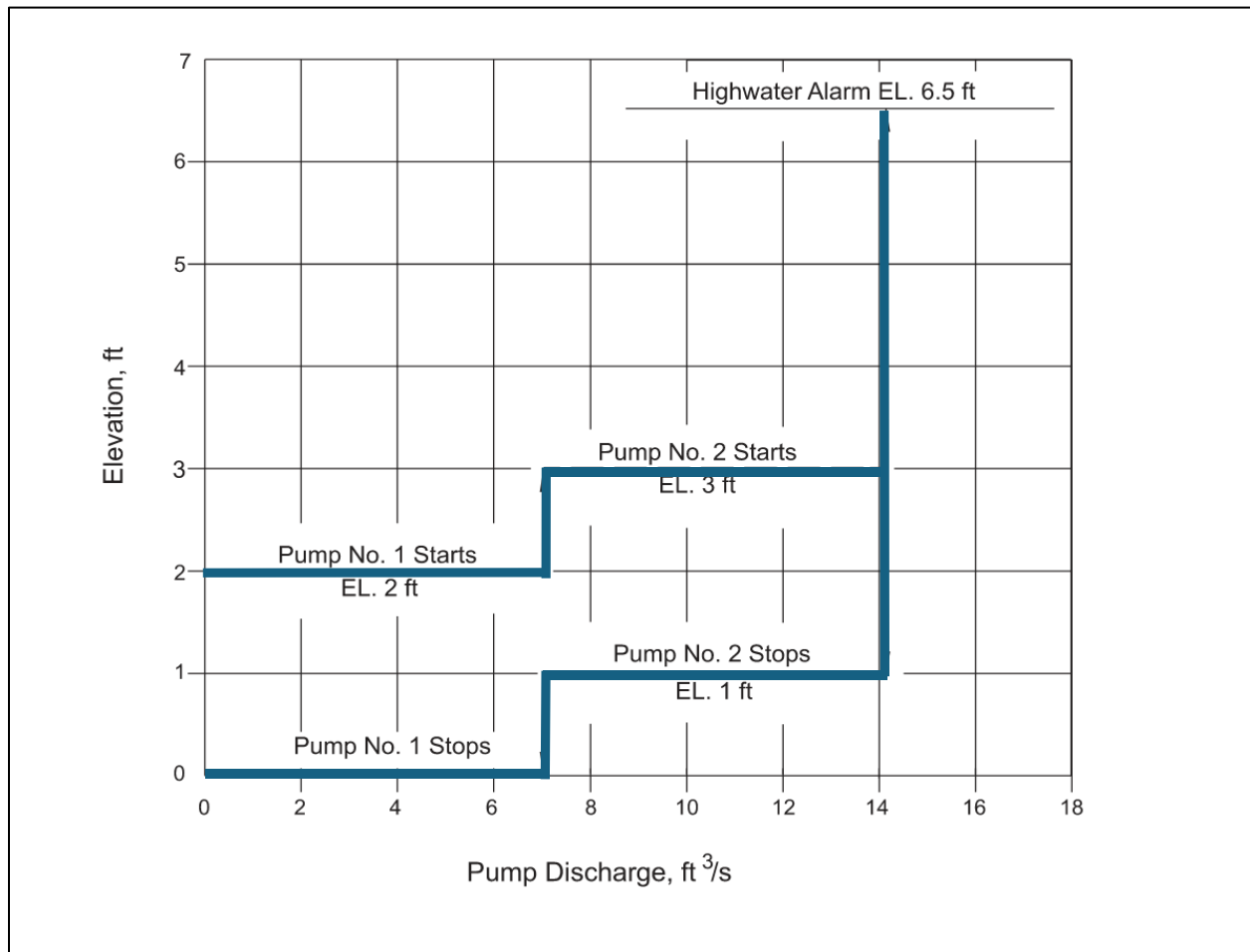


Table 10-B-4. Pumping Conditions for Example Problem (AASHTO Drainage Manual, 2014)

Pump No.	Pump-Start Elevation	Pump-Stop Elevation
Pump No. 1 (7ft/s)	2 ft (2,024 cft)	0 ft (0 cft)
Pump No. 2 (7ft/s)	3 ft (4,223 cft)	1 ft (596 cft)

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

Figure 10-B-9. Stage-Discharge Curve, Example Problem (AASHTO Drainage Manual, 2014)



The mass inflow curve is developed in **Figure 10-B-10**. Assume that two pumps have individual flow rates of 7 cfs since the peak flow was set at 14 cfs for the station.

Figure 10-B-10. Development of Mass Inflow Curve, Example Problem (AASHTO Drainage Manual, 2014)

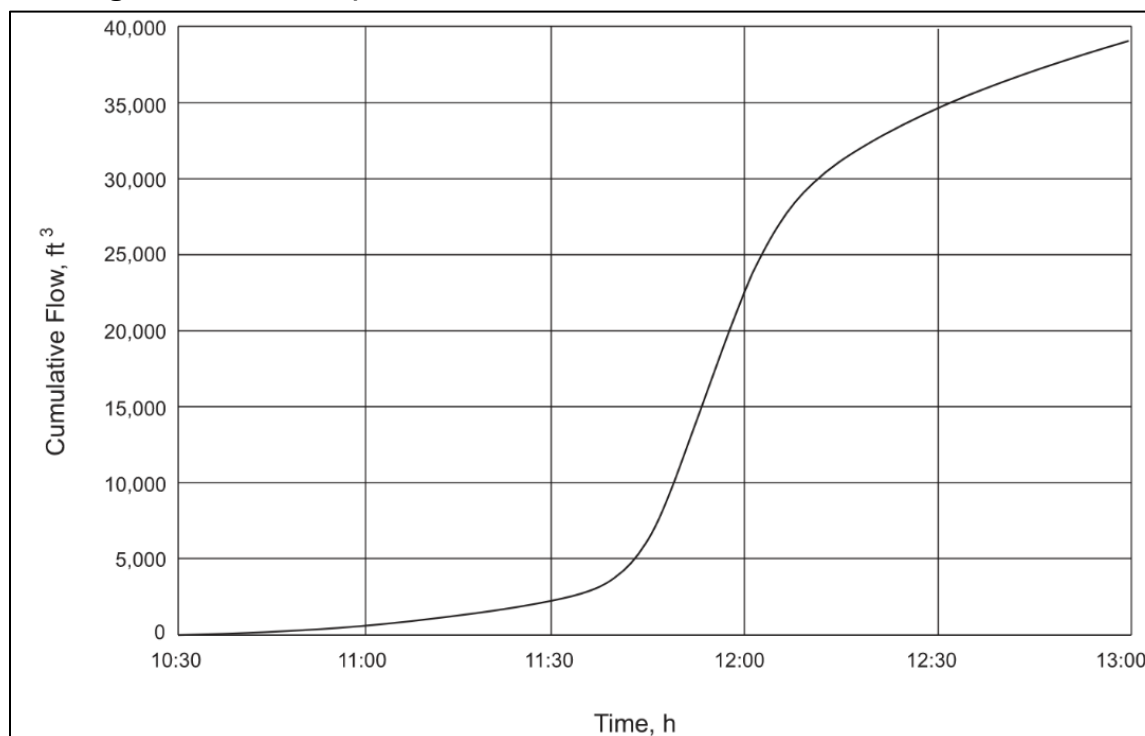


Figure 10-B-11 shows the plotting of the pump discharge curve on the inflow mass diagram.

- At approximately hour 11.4, the first pump is turned on when a storage volume of 2,024 cft 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 2,024 cft, 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 4,223 cft, which is associated with a turn-on elevation of 3 ft (refer to **Figure 10-B-9**). 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 (refer to **Figure 10-B-12**) represents stormwater that goes into storage. Pump cycling at the end of the storm has been omitted to simplify the illustration. When the remaining stored volume is equal to the 596 cft volume associated with the Pump No. 2 stop elevation of 1 ft, Pump No. 2 shuts off; Pump No. 1 shuts off when the storage pipe empties at the Pump No. 1 stop elevation of 0.0 ft.

The maximum difference in volume between the pump discharge curve and the inflow mass hydrograph is 8,475 cft. This represents the maximum storage required for the reduction of the 22 cfs peak to 14 cfs for the defined conditions (storage volume, start elevations, and pump rates). The design is adequate because the available storage at the highwater alarm is 8,778 cft. Note that a reduction of the starting elevations reduces the required storage volume. Adjustments are made on a trial basis until a satisfactory operating condition is developed.

Other pumping rates can be plotted on the inflow mass curve to determine their performance. Try different pumping rates and starting elevations until a satisfactory design is developed. The number of starts per hour is determined by using the inflow mass curve plots. The system is designed so that the allowable number of starts per hour for the selected pump size is not exceeded.

The pump discharge curve is superimposed on the design inflow hydrograph as shown in **Figure 10-B-12** to aid the designer in visualizing what is happening during the peak design period in this process.

Figure 10-B-11. Mass Curve Routing Diagram, Example Problem (AASHTO Drainage Manual, 2014)

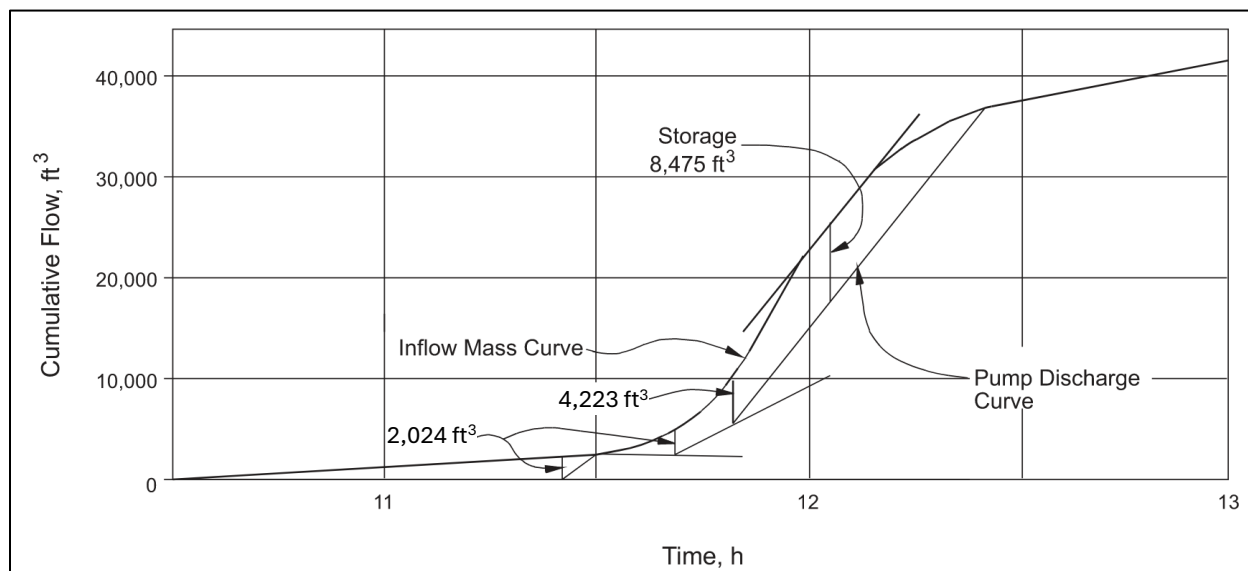
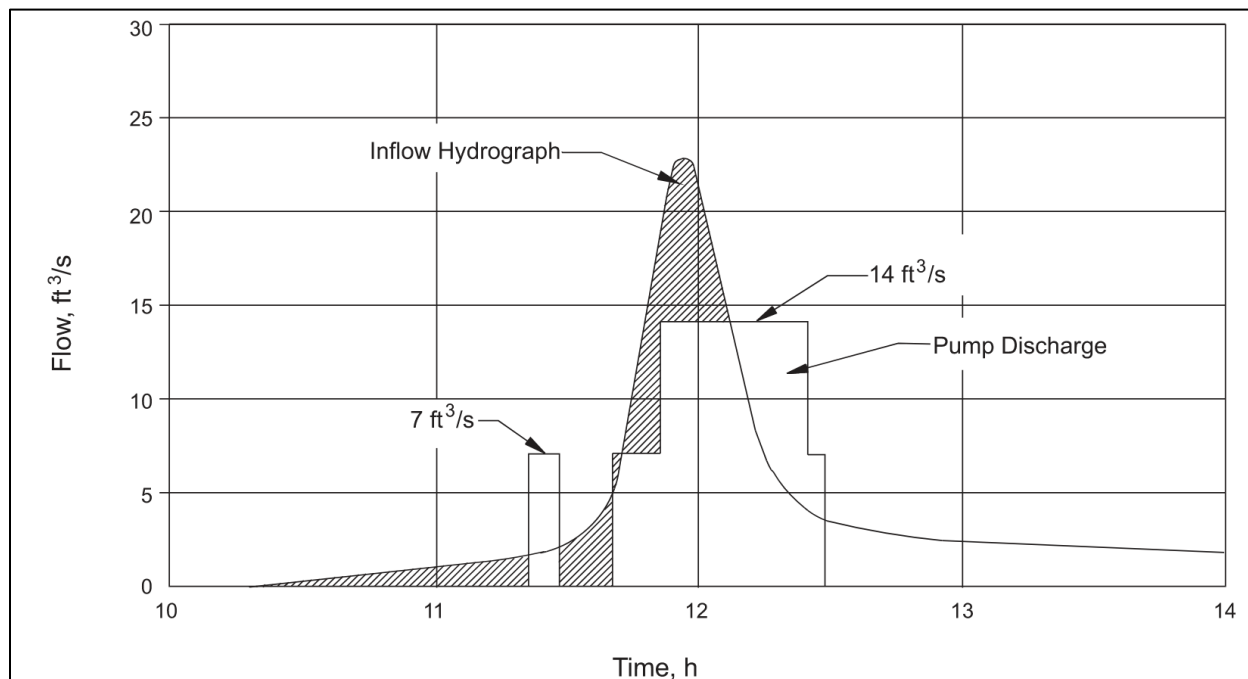


Figure 10-B-12. Pump Discharge, Example Problem (AASHTO Drainage Manual, 2014)



Step 8 – Trial Pumps and Pump Station Piping

A specific pump is selected to establish the size of the discharge piping by using information previously developed or established. During design a specific manufacturer is not selected. Instead, a specification is developed to describe the relationships between total dynamic head, discharge, efficiency, and energy requirements. The specification also indicates the size of discharge piping needed because pumps that produce the desired results will have a specific discharge pipe size.

The discharge line is steel or cast-iron pipe and is at least the size of the pump discharge diameter. The pipe is sized to maintain a maximum velocity of 10 ft/s and is checked using **Equation 10-B-4**:

$$D = 1.128 \left(\frac{Q}{v} \right)^{1/2} \quad (10-B-4)$$

Where:

- D = pipe diameter, ft
- Q = discharge in pipe, cfs
- v = velocity in pipe, ft/s

Determine the length of the discharge pipe based on the station layout. The pump location with respect to the outfall chamber is set to provide the shortest discharge line

possible. Other components of the discharge line that affect the head losses (e.g., valves, elbows, manifolds, flap gates, other fixtures) are identified for their effect on total dynamic head. The discharge line is designed 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 (TDH) is the sum of the static head, velocity head, and various head losses in the pump discharge system due to minor losses and 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 and valves) designated, calculate total dynamic head for the discharge system using **Equation 10-B-5**:

$$\text{TDH} = H_s + \Sigma H_f + H_v + H_p \quad (10-B-5)$$

Where:

- H_s = static head or height which the water must be raised, ft
- H_f = loss due to friction in the pipe, ft
- H_v = velocity head, ft
- H_p = Minor losses through appurtenances (e.g., valves and elbows), ft

The friction loss in the discharge line is computed by using the Hazen-Williams Equation in **Equation 10-B-6**:

$$H_f = \frac{C_u \times v^{1.85} \times L}{C^{1.85} \times D^{1.165}} \quad (10-B-6)$$

Where:

- H_f = loss due to friction in the pipe, ft
- C_u = unit conversion factor, 3.022
- L = length of pipe, ft
- v = velocity in the pipe, ft/s
- C = friction factor
- D = pipe diameter, ft

The Hazen-Williams Equation is 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.

Minor losses through appurtenances (e.g., valves and elbows) are determined by using a dimensionless loss factor, K , applied to the velocity head using **Equation 10-B-7**:

$$H_p = \frac{Kv^2}{2g} \quad (10-B-7)$$

Where:

- H_p = friction loss through appurtenance, ft
- K = Minor loss factor (based on standard data or manufacturer's specified data)
- v = velocity through appurtenance, ft/s
- g = acceleration due to gravity, ft/s²

Consult FHWA HEC-24 (2001) for additional information on estimating minor losses in pipe appurtenances.

Step 10 – Pump Design Point

Using methods described in Step 9, the TDH of the outlet system is calculated for a specific static head and dynamic head for a range of discharge rates. The TDH is plotted against discharge, Q , for the entire pumping and discharge system. This graphical representation is called a system head curve. The required design point of a pump is established after the pump curve is superimposed on the system head curve (refer to **Figure 10-B-13**). The system head curve starts from a low point on the Y-coordinate representing the static head at zero discharge. It rises to the right as the discharge and the friction losses increase. A design flow is selected on the system head curve and a pump is selected to match that point. The pump curve is superimposed on the system head curve and the point of intersection is clearly identifiable. Several pump curves can be plotted over the system head curves. If the discharge line size is changed, a new system head curve for the updated size (and changed head loss) is developed.

Each pump has a unique performance curve developed by the manufacturer (refer to **Figure 10-B-14**). A family of curves is developed for each pump because any pump can be fitted with various size impellers. The performance curves are the basis for the pump curve plotted with the system head curves previously discussed. Specific information on the pumps available are used to specify the pumps needed for the pump station. A study of pump performance curves is completed for each design.

Any point on an individual performance curve identifies the performance of a pump for a specific TDH that exists in the system. It also identifies the power required and the efficiency of operation of the pump. For either an increase or decrease in TDH, the pump efficiency is reduced as the performance moves away from the mid-point of the performance curve. As the TDH increases, the power requirement also increases. Confirm the motor specified is adequate over the full range of TDH and flow rates that

will exist. It is desirable that the design point be as close to the mid-point as possible or else to the left of the mid-point. The range of the pump operation should not extend into the areas where substantially reduced efficiencies exist.

Correlate the design point with an elevation at approximately the mid-point of the pumping range. With this design point, the pump works above and below the TDH for the design point, operating in the best efficiency range.

Figure 10-B-13. System Head Curves (for Single Pump) (AASHTO Drainage Manual, 2014)

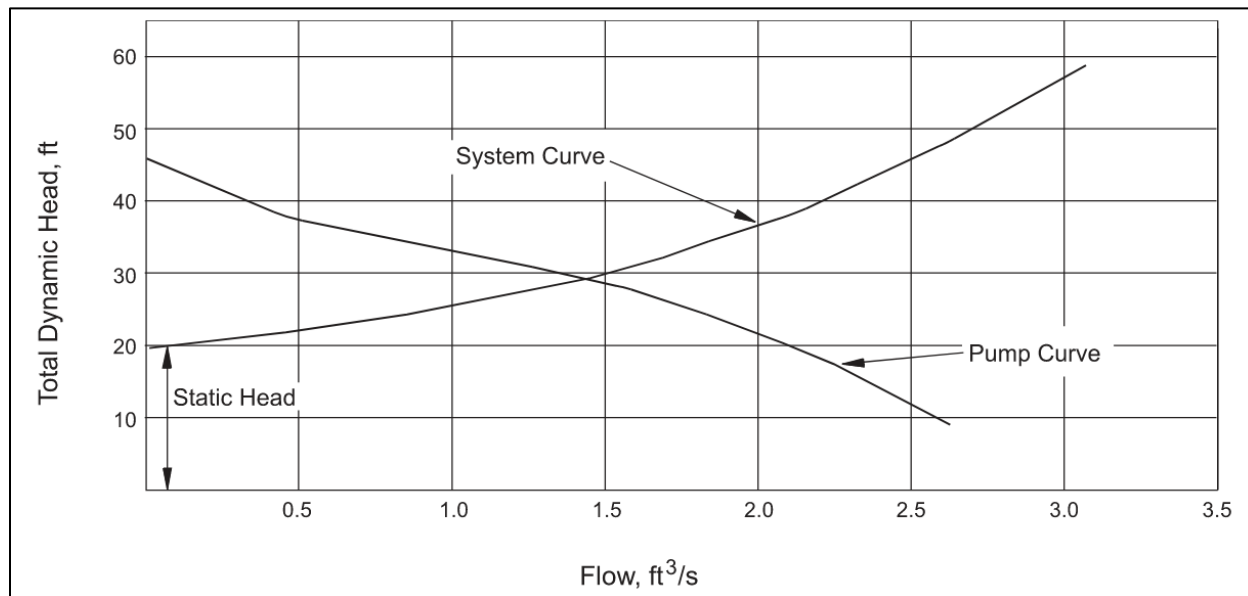
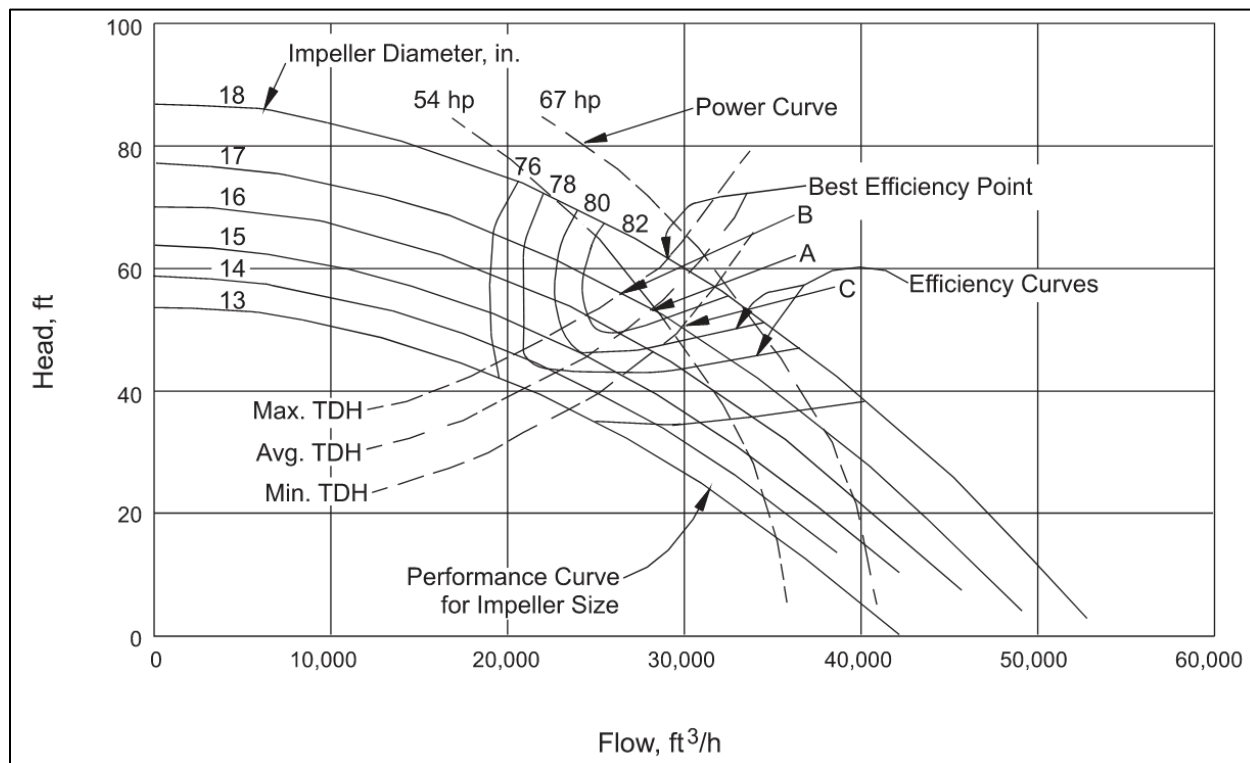


Figure 10-B-14. Manufacturer's Performance Curves (AASHTO Drainage Manual, 2014)



Step 11 – Power Requirements

To select the proper size of pump motor, compute the energy required to raise the water from the lowest level in the pump pit to the 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. Use **Equation 10-B-8** to solve for brake power.

$$e = \frac{(Q \gamma H)}{(550 \times P)} \quad (10-B-8)$$

Where:

- e = efficiency = pump output/brake power
- Q = pump capacity, cfs
- γ = specific weight of liquid, 62.4 lb/cft for cold water
- H = head, ft
- P = brake power, hp

Efficiency is segregated into partial efficiencies – hydraulic and mechanical. However, the efficiency as described in this section is a gross efficiency used for the comparison of centrifugal pumps. Pump performance curves are studied from several

manufacturers to determine appropriate efficiency ranges. A minimum acceptable efficiency is specified by the designer for each performance point specified.

Use the pump efficiency from the manufacturer's performance curves at the established design point to compute the energy required to drive a pump.

References

References in bold type are recommended reading for the engineer's library. MDOT does not claim responsibility for the information at these links or their maintenance.

AASHTO Technical Committee on Hydraulics and Hydrology. (2014). *AASHTO Drainage Manual*. American Association of State Highway and Transportation Officials.

American National Standards, Inc. (ANSI). (2024). *Rotodynamic Pumps for Pump Intake Design*. ANSI/HI 9.8. Hydraulic Institute.

Federal Highway Administration. (2024, February). *Urban Drainage Design Manual*. Hydraulic Engineering Circular No. 22 (HEC-22). (4th Ed). FHWA-HIF-24-006. U.S. Department of Transportation.

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif24006.pdf>

Federal Highway Administration. (2001, February). *Highway Stormwater Pump Station Design*. Hydraulic Engineering Circular No. 24 (HEC-24). (1st Ed). FWHA-NHI-01-007. U.S. Department of Transportation.

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/hec/hec24.pdf>

If you require assistance accessing this information or require it in an alternative format, contact the Michigan Department of Transportation's (MDOT) Americans with disabilities Act (ADA) coordinator at Michigan.gov/MDOT-ADA.