## 12.5 Recommended Design Criteria

### 12.5.1 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.

### 12.5.2 Station Type And Depth

Since dry-pit stations are as much as 60% more expensive than wet-pit stations, wet-pit stations are recommended. Dry-pit stations are more appropriate for handling sewage because of the potential health hazards to maintenance personnel. The hazards associated with pumping stormwater usually do not warrant the added expense. Some advantages associated with dry-pit stations include ease of access for repair and maintenance and the protection of equipment from fire and explosion.

The station depth should be minimum. No more depth than that required for pump submergence and clearance below the inlet invert is necessary, unless foundation conditions dictate otherwise.

### 12.5.3 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 225 kW. 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 stations have a SBP receptacle, manual transfer switch and a portable engine/generator set, then the practical power limit of the pumps becomes 55 kW since this is the limit of the power generating capabilities of most portable generator units. Two pumps would be operated by one engine/generator set.

### 12.5.4 Discharge Head And System Curve

Since 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.

Once the head losses have been calculated for the range of discharges expected, the system curve (Q vs. TDH) can 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 manufacturer), it will yield the pump operating points (see Figure 12-4).

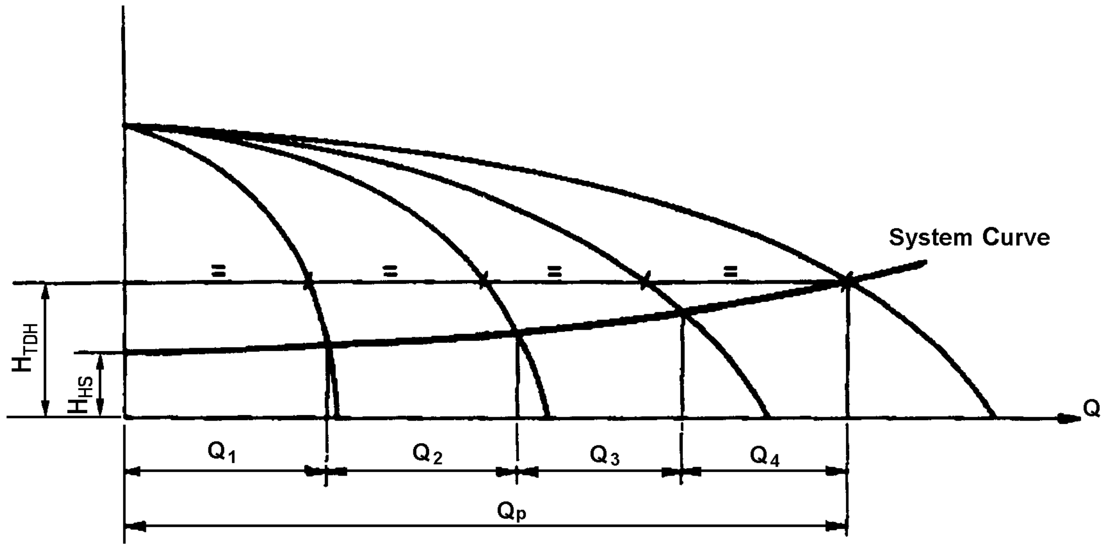



**Figure 12-4 Systems Curve**

When the pump is raising the water from the lowest level, the static head will be greatest and the discharge will be the least. When operating at the highest level, the static head will be the least and the discharge will be the greatest. The capabilities of pump must always be expressed in both quantity of discharge and the total dynamic head at a given level. The designer will typically specify these conditions for three points on the performance curve. One point will be near maximum TDH, the next will be the design point, and the third will be at about minimum TDH.

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

A pump is selected to operate with the best possible efficiency as its Design Point, corresponding to the Design Water Level of the station, and its performance is expressed as the required discharge at the resulting total dynamic head. The efficiency of a stormwater pump at its design point may be 75 or 80% or more, but this will depend on the type of pump.

When the static lift is greatest (low water in sump), the energy required (kilowatt) maybe the greatest even though the quantity of water raised is less. This is because the pump efficiency may also be much less. The pump selection should be made so that maximum efficiency is at the design point.

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. Because pumps must operate over a range of water levels, the quantity delivered will vary significantly between the low level of the range and the high level. Typically, the designer will be required to specify at least three points on the performance curve. These will typically be the conditions for the TDH at the highest head, the design head and the lowest head expected over the full operating range of the pump. The manufacturer always plots a curve of total dynamic head versus pump capacity for each pump. When running, the pump will respond to the total dynamic head prevailing and the quantity of discharge will be in accordance with the curve. The designer must study the pump performance curves for various pumps in order to develop an understanding of the pumping conditions (head, discharge, efficiency, kilowatt, etc.) throughout the full range of head that the pump will operate under. The system specified must operate properly under the full range of specified head.

The total dynamic head (TDH) should be determined for a sufficient number of points to draw the system head curve that will be discussed later in this chapter. The TDH is computed as follows:

$$\mathbf{TDH = H_s + H_f + H_v + H_l} \qquad \mathbf{(12.1)}$$

where: TDH = total dynamic head, m (ft)
$H_s$ = max. static head (at lowest pump-off elevation), m (ft)
$H_f$ = friction head, m (ft) (i.e., friction loss)
$H_v$ = velocity head, m (ft) ($V^2/2g$)
$H_l$ = losses through fittings, valves, etc., m (ft)

Adjustments may have to be made to these curves to account for losses within the pumping unit provided by the manufacturer.

## 12.5.5 Main Pumps

Number And Capacity

The designer will determine the number of pumps needed by following a systematic process defined in section 12.6. 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 over sizing the pumps to compensate, in part, for a pump failure. The two pump system could have pumps designed to pump 66 - 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 numbers of the pumps.

It is recommended that economic limitations on power unit size as well as 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 meters and start meters should be provided to aid in scheduling needed maintenance.

Final Selection

For the typical highway application, any of the three pump types described earlier will usually suffice. If not, manufacturers' information will likely dictate the type required. However, knowing the operating RPMs, a computation can be made to check the appropriateness of the pump type (see Figure 12-5 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 0.21 L/s (.055 gal/s) against TDH of 1 m (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.

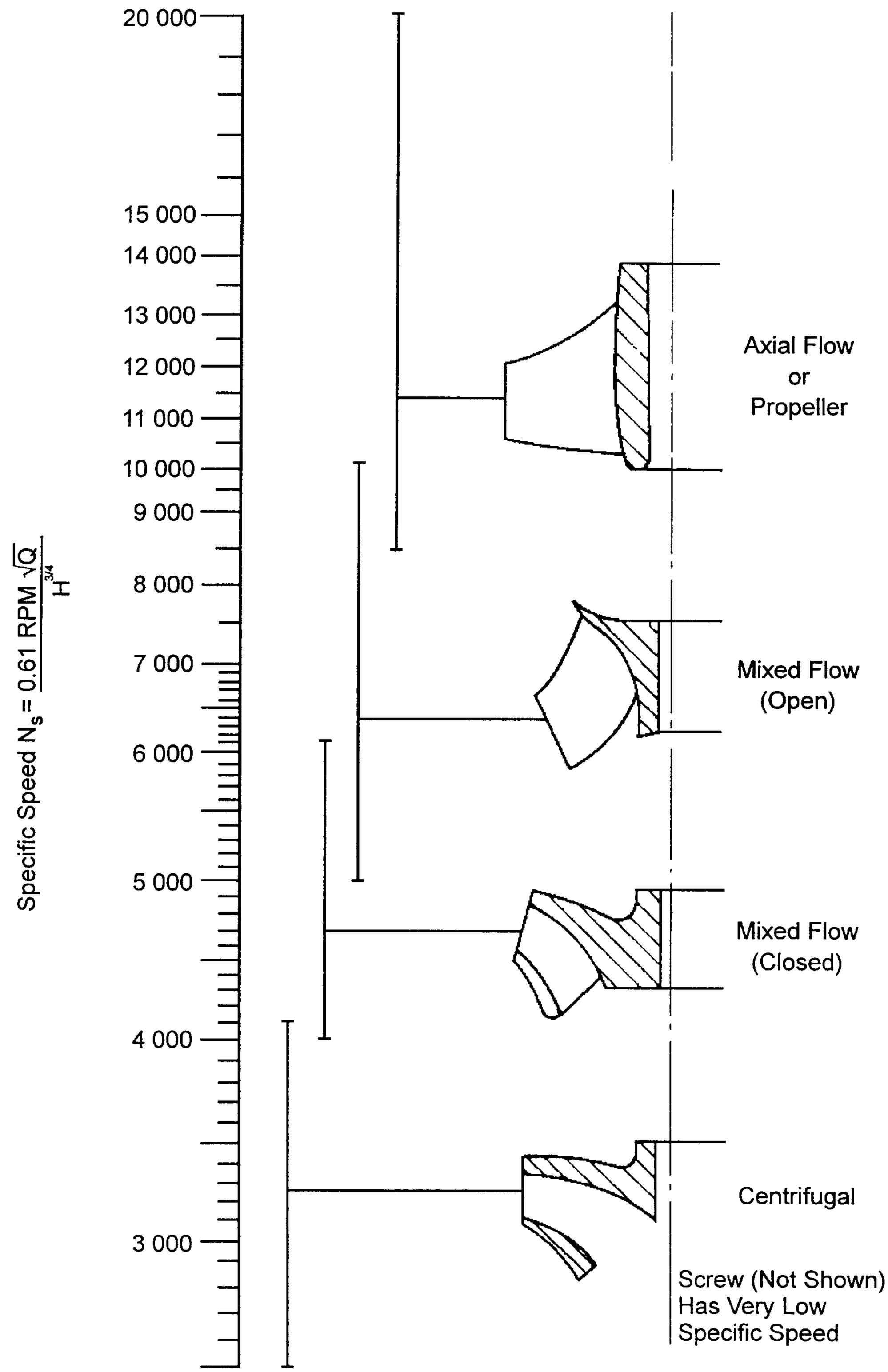



**Figure 12-5 Specific Pump Speed vs. Impeller Types**

### 12.5.6 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.

### 12.5.7 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 12-6 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 much smaller pump station can be utilized, with anticipated cost benefits. If the discharge rate is to be limited, ample storage is essential.

Since most highway related pump stations are associated with either short underpasses or long depressed sections, it is not reasonable to consider above ground 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. The pump stations can remove the stored water by either the dry pit or the wet pit approach as chosen by the designer.

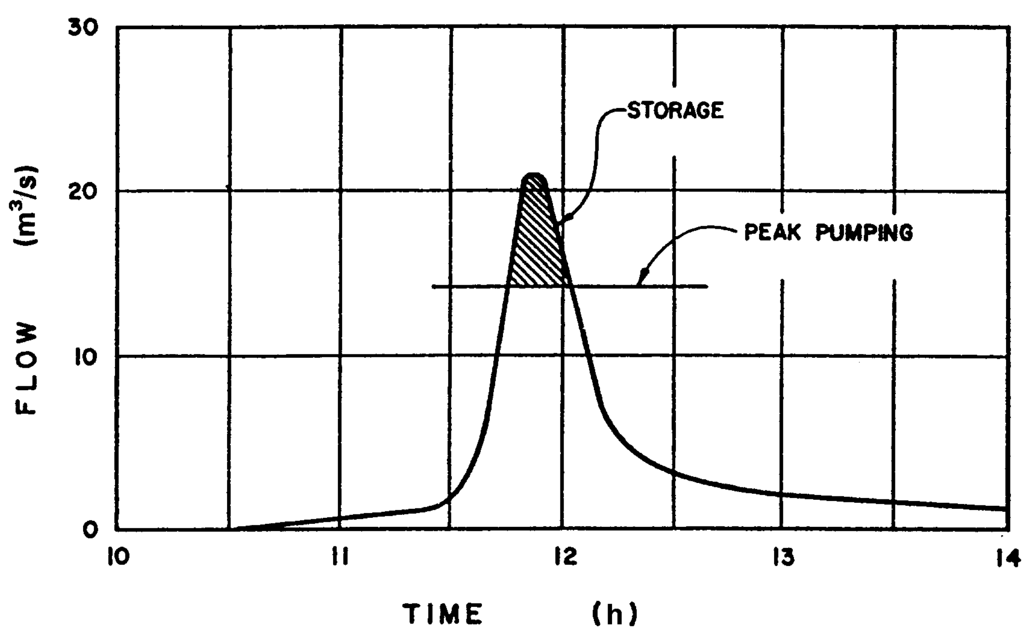



**Figure 12-6 Estimating Required Storage**

### 12.5.8 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.

### 12.5.9 Wet Well Design

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, it 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, etc. (see Figure 12-7). There are countless variations between these two sequences.

There are also different alternation techniques which 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, etc. (See Figure 12-8).

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 12-9). This total volume is computed as follows:

$$\mathbf{V_t = Q_p\, t_c/4N} \qquad \textbf{(12.2)}$$

where: $V_t$ = total cycling storage volume, $m^3$ ($ft^3$)
$Q_p$ = total capacity of all pumps, $m^3/s$ ($ft^3/s$)
$t_c$ = minimum allowable cycle time, s (= 3600/max. starts per hour)
N = total number of equal-size pumps

The volume required for each pump will vary, depending upon the characteristics of the discharge system. It should be noted here that with these volumes, the minimum allowable cycle time will only be experienced when the proportionate inflow to each pump is exactly one-half the capacity of that pump. All other inflows will produce a cycle time longer than the minimum.

Lowest Pump "Off" Elevation

It is recommended that the lowest pump "off" elevation be located at or within 0.3 m (1 ft) below the inlet invert elevation unless plan dimension constraints dictate that the station floor be lowered to obtain the necessary cycling volume. 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.

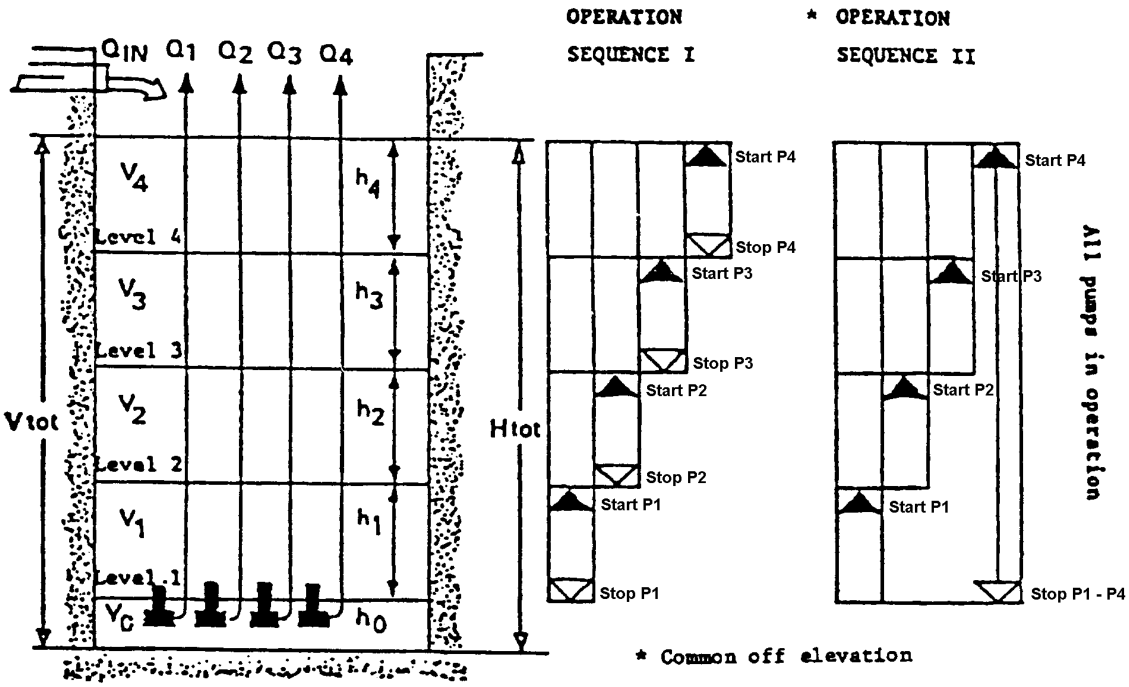
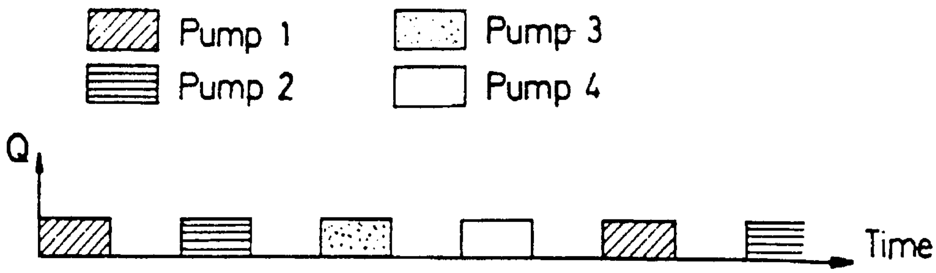


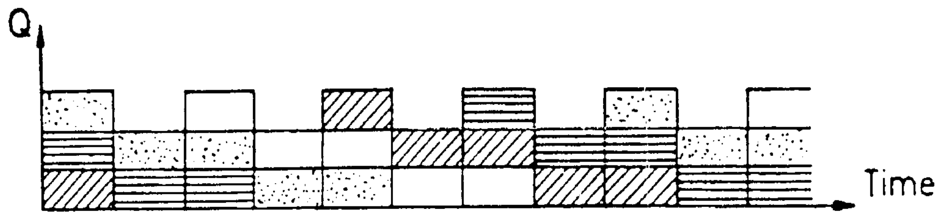

**Figure 12-7 Pump Sequences**

Notes to Figure 12-7.

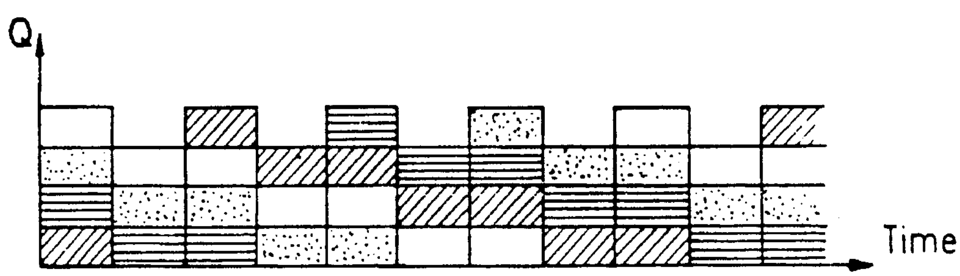
- Decreasing sump volume by pump alternation. Pumps starting in sequence and stopping in reverse order.
- By designing the control system for pump alternation, the sump volumes can be reduced as well as distribute the pump operating time more evenly between the four pumps.
- This system works for any number of pumps in a station.
- For example when four pumps are installed in the same station. If the inflow is less than the capacity of one pump, pump number one would, without alternation, do all the work.
- With alternation pump number one starts and draws down. Next start would call pump number two, etc. This means that with four pumps of the same size and operating in an alternating sequence each pump is called on to pump down 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).

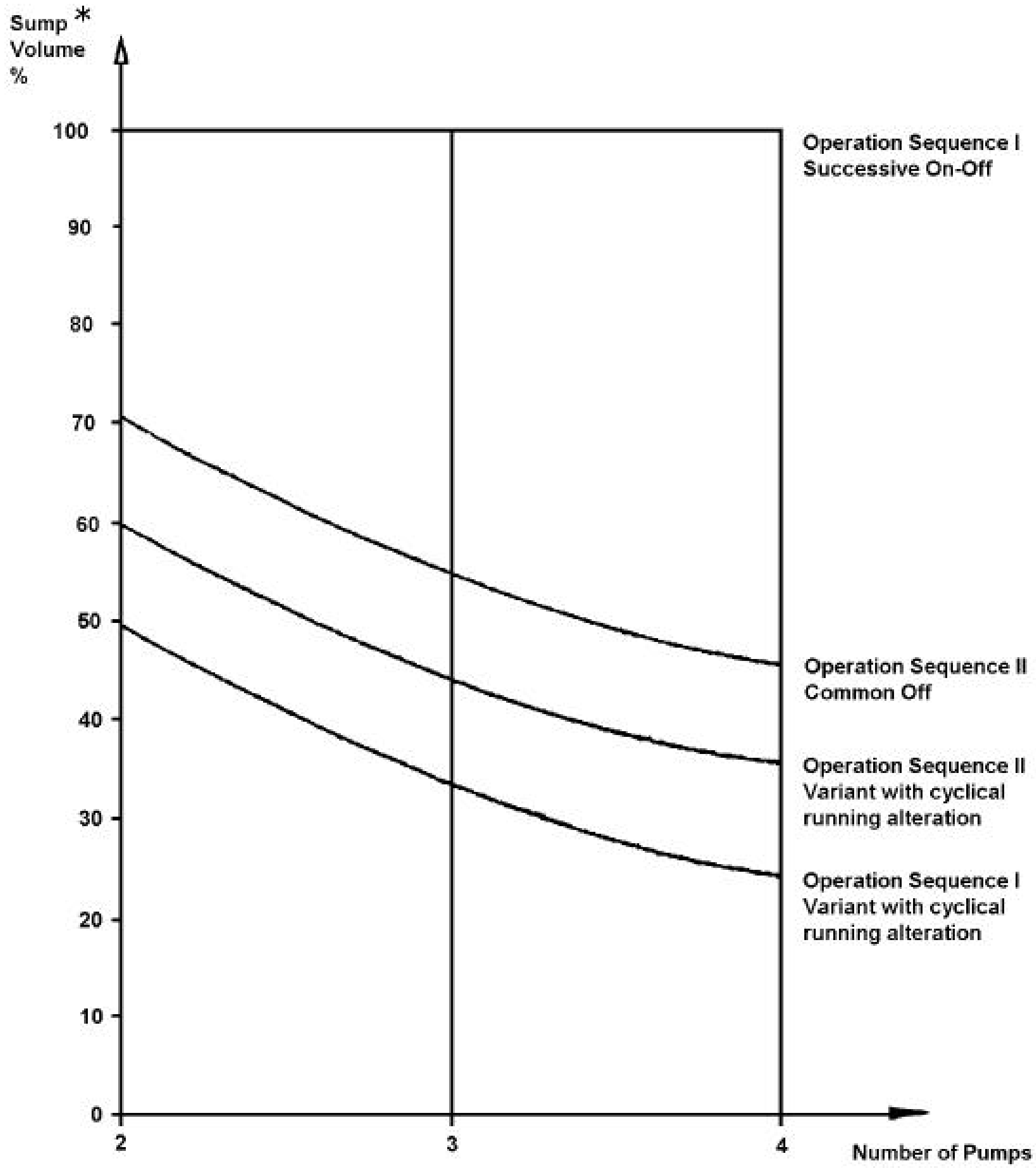



If Qin is greater than the capacity of two pumps but less than three pumps, the pumps are operating 5/8 of their cycle time.




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 12-8 Pumps With Cyclical Running Alternation — A Variant Of Operation Sequence I**




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

**Figure 12-9 A Comparison Of The Pump Volume With And Without Cyclical Running Alternation**

Pump "On" Elevations

These should be set at the elevations which 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:

$$\mathbf{H_x = V_x/plan\ area} \quad \mathbf{(12.3)}$$

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 0.3 to 0.6 m ( 1 to 2 ft) of freeboard below the roadway grate.

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 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.

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, since 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.

## 12.5.10 Stormwater Pump Station Storage

The development of the wetwell design as discussed in Section 12.5.9 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. In the case of many of the highway storm drain situations, it has been the practice to store substantial parts of the flow in order to minimize pumping requirements as well as 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 in the case of larger storage volume development. However, it will be noted that differences exist as the volume of storage become 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 pre-development 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.

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 since 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 quite 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 0.62 $m^3/s$ to 0.40 $m^3/s$ as shown in Figure 12-10. Using the assumed storage pipe shown in Figure 12-11, the stage-storage curve in Figure 12-12, the stage discharge curve in Figure 12-13 and the inflow hydrograph in Figure 12-10, the storage can be determined.

The inflow mass curve is developed in Figure 12-14. Since 0.40 $m^3/s$ was to be pumped, it was assumed that two 0.2-$m^3/s$ pumps would be used. The pumping conditions are as follows:

| | Pump-Start Elevation | Pump-Stop Elevation |
|---|---|---|
| Pump No. 1 (0.2 $m^3/s$) | 0.61 (57.3) | 0.0 (0) |
| Pump No. 2 (0.2 $m^3/s$) | 0.91 (119.7) | 0.30 (16.9) |

The numbers in parentheses are the storage volumes ($m^3$) associated with the respective elevations.

Figure 12-15 shows the plotting of the pump discharge curve on the inflow mass diagram. Note that the first pump is turned on at about hour 11.4 when a storage volume of 57 $m^3$ has accumulated. At about hour 11.5 pump number one has emptied the storage basin and the pump turns off. At about hour 11.7 the storage volume has again reached 57 $m^3$ 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 about hour 11.8 the volume in storage has increased to 120 $m^3$ which is associated with a turn on elevation of 0.91 m. Both pumps operate until about 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.

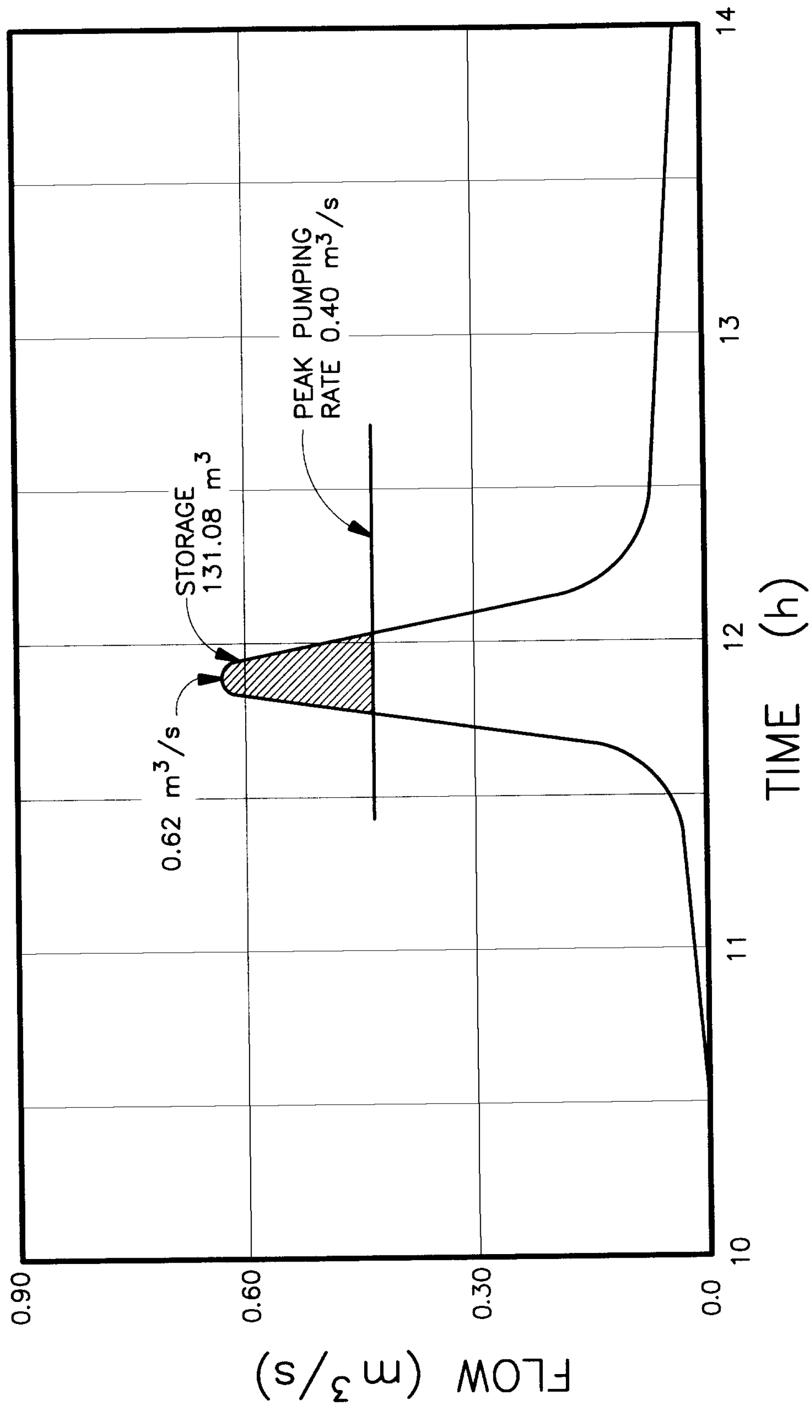



**Figure 12-10 Estimating Required Storage**

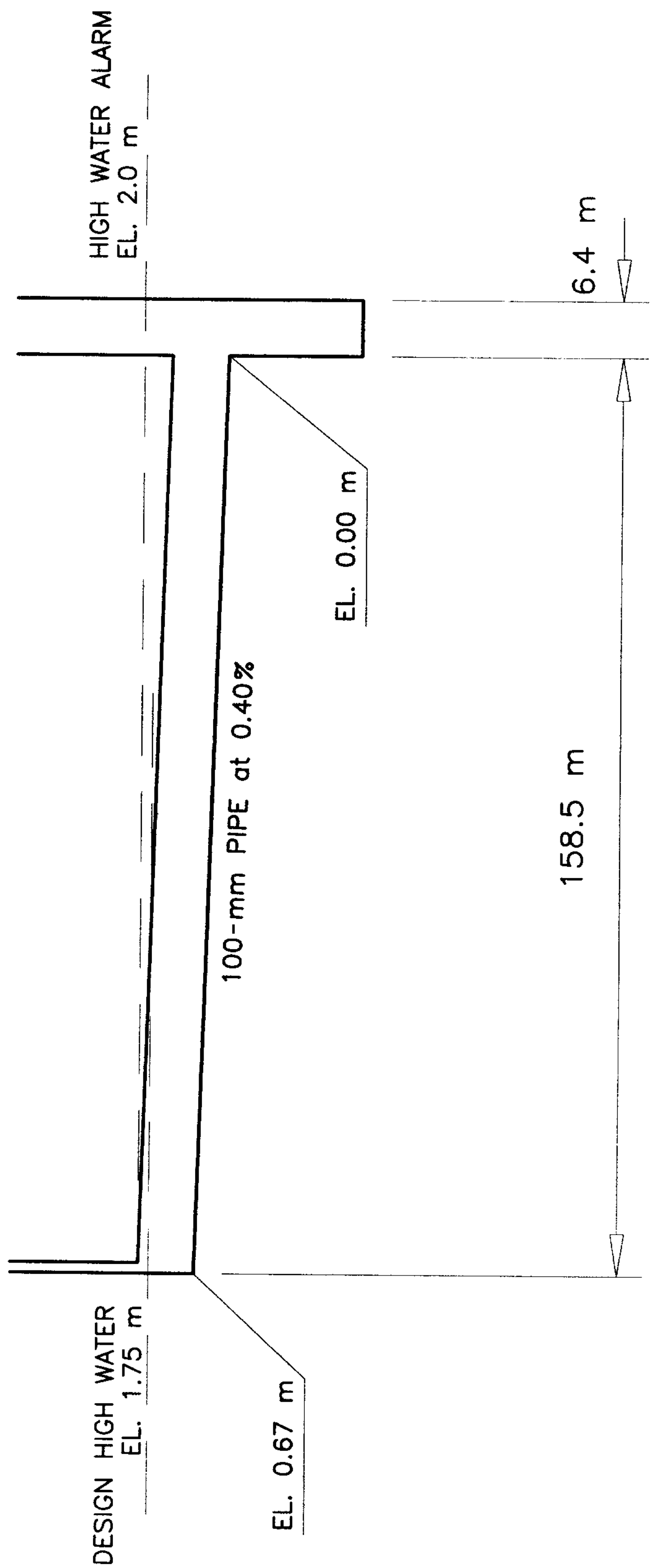



**Figure 12-11  Storage Pipe Sketch**

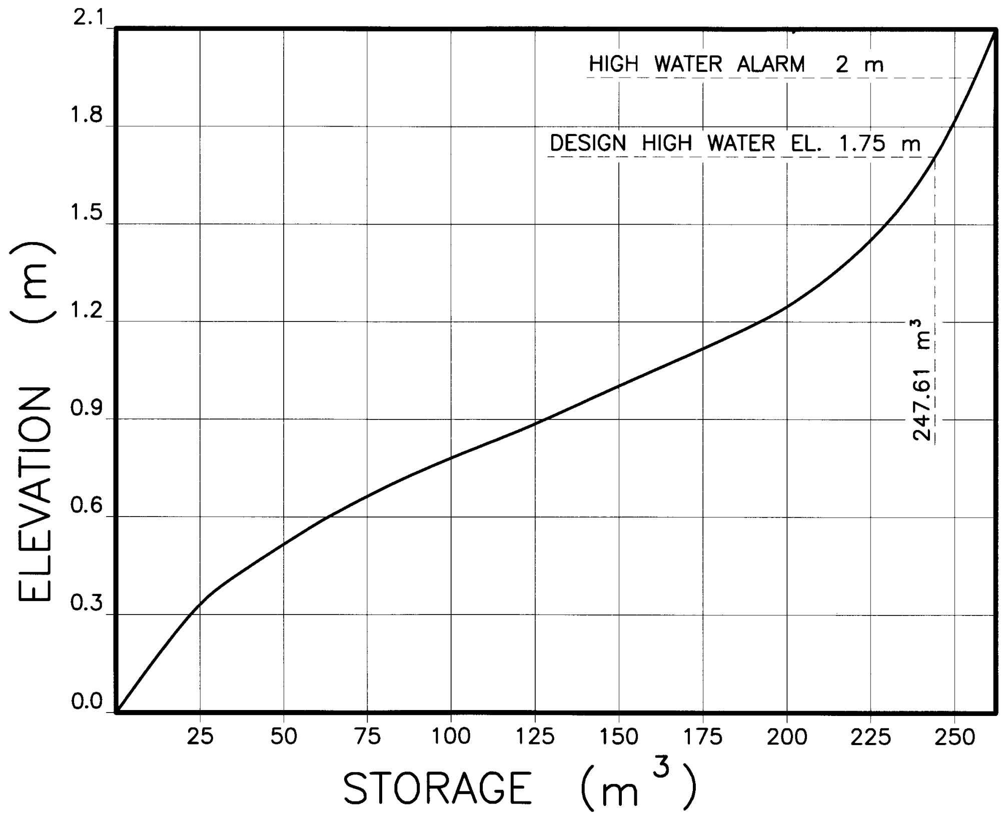



**Figure 12-12  Stage-Storage Curve**

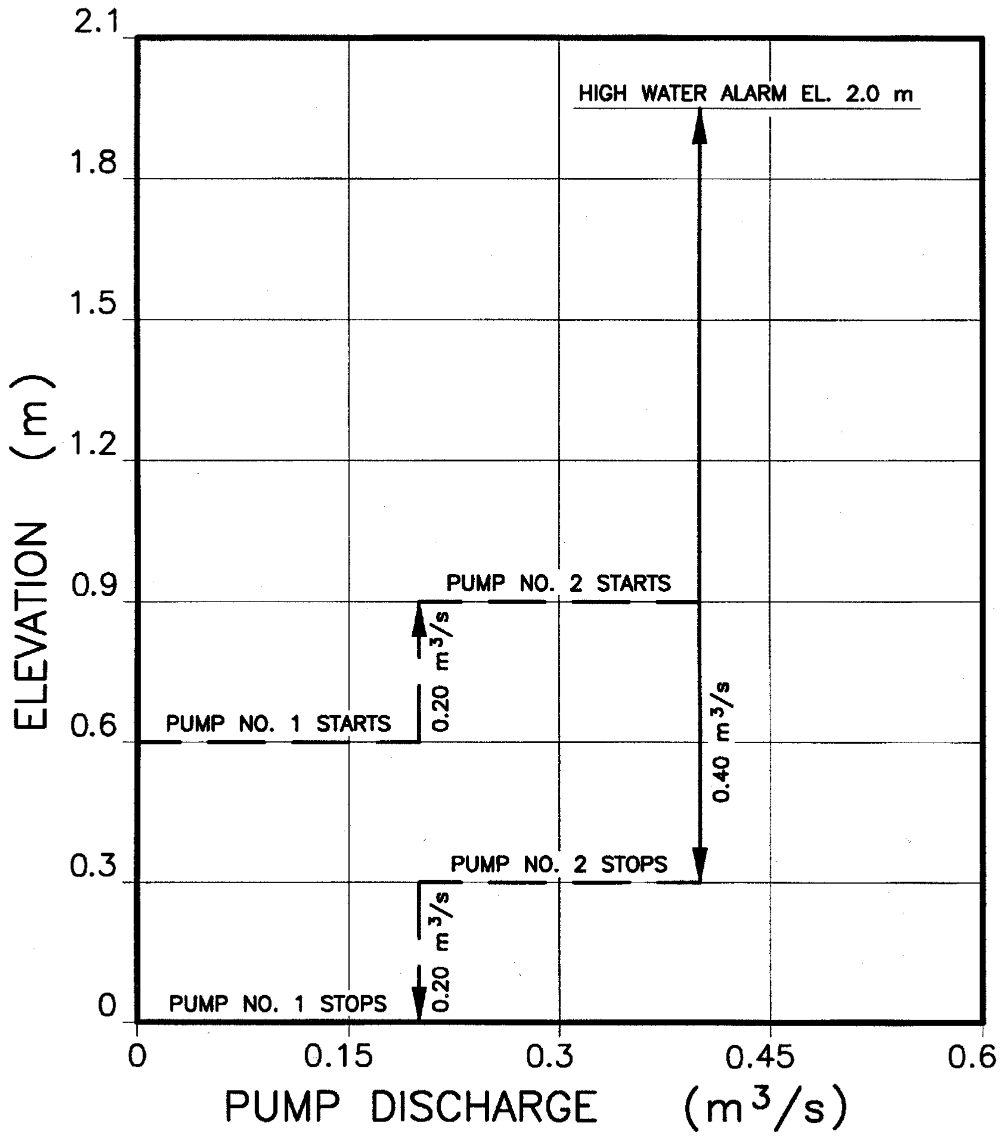



**Figure 12-13 Stage-Discharge Curve**

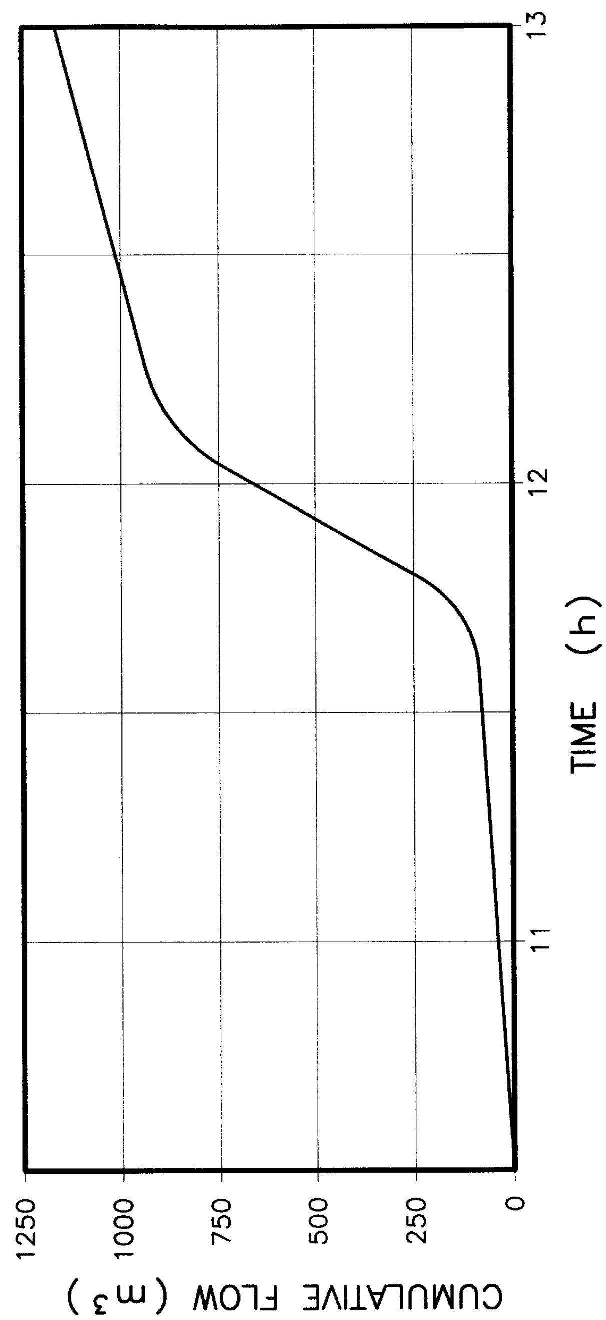



**Figure 12-14 Development Of Inflow Mass Curve**

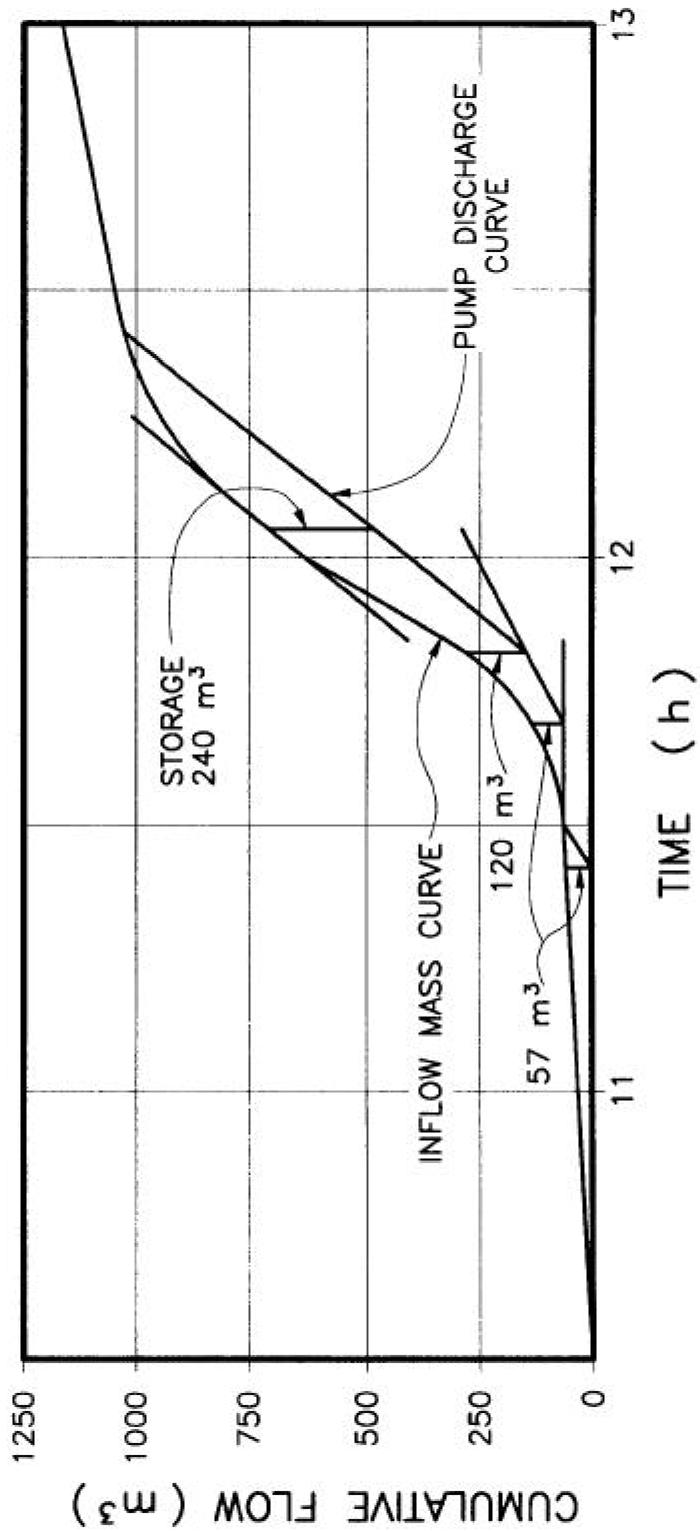



**Figure 12-15 Mass Curve Routing Diagram**

The shaded area between the curves (see Figure 12-16) represents stormwater that is going into storage. Pump cycling at the end of the storm has been omitted in order to simplify the illustration. When the stored volume remaining is equal to the volume (17 $m^3$) associated with the Pump No. 2 stop elevation (0.3 m), pump number 2 shuts off; Pump No. 1 shuts off when the storage pipe is emptied at Pump No. 1 stop elevation (0.0).

The maximum vertical distance between the pump discharge curve and the inflow mass hydrograph is 240 $m^3$. This represents the maximum storage required for the reduction of the 0.62-$m^3$/s peak to 0.40 $m^3$/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 since the available storage at the high water alarm is 249 $m^3$. 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. 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 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.

The designer now has a complete design that allows the problem to be studied in-depth. The peak rate of runoff has been reduced from 0.62 $m^3$/s, the inflow hydrograph peak, to 0.40 $m^3$/s, the maximum pump discharge rate. A reduction of 46.5% is accomplished by providing for 240 $m^3$ of storage. This is only one possible design option. The designer may wish to reduce the pumping rate further by providing more storage, and additional combinations of pump discharge and storage can be considered.

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 12-16.