



Puerto Rico Transportation Technology Transfer Center
Department of Civil Engineering and Surveying
University of Puerto Rico at Mayagüez
Call Box 9000 * Mayagüez, PR 00681
Tel. 787-834-6385 * Fax: 787-265-5695 * www.prltap.org



*29th Anniversary of Excellence in the Training of Transportation Officials at
Municipal, State and Federal Level in Puerto Rico and Virgin Islands*

URBAN DRAINAGE DESIGN

Part 2



Instructor

Dr. Walter F. Silva-Araya

Professor

Department of Civil Engineering and Surveying
University of Puerto Rico – Mayagüez Campus



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Module 5

Hydraulics: Pavement Drainage



PAVEMENT DRAINAGE

Dr. Walter F. Silva
UPR Mayaguez



Design Frequency and Spread

- Variables to consider:
 - Frequency of the design runoff event
 - Allowable spread of water on the pavement.

Spread and Design frequency are not independent.





Selection of Design Frequency and Design Spread

- Safety of travelling public is the primary consideration.
- Major considerations:
 1. Classification of the highway.
 2. Design speed (hydroplaning).
 3. Projected traffic volumes.
 4. Intensity of rainfall events.
 5. Capital costs.



Selection of Design Frequency and Design Spread cont.

- Relative elevation of the highway and surrounding terrain is considered where water can be drained only through a storm drainage system.
- Spread on traffic lanes can be tolerated to greater widths where traffic volumes and speeds are low.
- Spread of one-half are usually considered for low-volume local roads.



Selection of Design Frequency and Design Spread cont.

- Minimum design frequencies and spread based on the type of highway and traffic speed.

Suggested Minimum Design Frequency and Spread.			
Road Classification		Design Frequency	Design Spread
High Volume or Divided or Bi- Directional	< 70 km/hr (45 mph)	10-year	Shoulder + 1 m (3 ft)
	> 70 km/hr (45 mph)	10-year	Shoulder
	Sag Point	50-year	Shoulder + 1 m (3 ft)
Collector	< 70 km/hr (45 mph)	10-year	1/2 Driving Lane
	> 70 km/hr (45 mph)	10-year	Shoulder
	Sag Point	10-year	1/2 Driving Lane
Local Streets	Low ADT	5-year	1/2 Driving Lane
	High ADT	10-year	1/2 Driving Lane
	Sag Point	10-year	1/2 Driving Lane

[1]

ADT = Average daily traffic



Selection of Design Frequency and Design Spread cont.

- Design frequency for depressed sections and underpasses where ponded water can be removed only through the storm drainage system is a 50-year frequency event.
- The use of lesser frequency event such as 100-year storm, to assess hazards at critical locations is commonly referred to as a check storm or check event.
- Check storm should be used any time runoff could cause unacceptable flooding during less frequent events.



Surface Drainage

- When the rain falls on a sloped pavement surface, the depth of water on the pavement increases and so does the potential for vehicular hydroplaning.
- Design guidance of the following drainage elements is important for minimizing the effects of hydroplaning:
 1. Longitudinal pavement slope
 2. Cross or transverse pavement slope
 3. Curb and gutter design
 4. Roadside and median ditches
 5. Bridge decks
 6. Median Barriers
 7. Impact attenuators

Hydroplaning

- Water begins to build in front of the tire to a point where it can produce a hydrodynamic force which can lift the tire off the pavement.
- Hydroplaning is a function of :
 - water depth
 - roadway geometrics
 - vehicle speed
 - tread depth
 - tire inflation pressure
 - conditions of the pavement surface



[3]

Hydroplaning cont.

- Hydroplaning may be reduced by:
 1. Design the highway to reduce the drainage path lengths of the water flowing over the pavement.
 2. Increase the pavement surface texture depth.
 3. Use of open graded asphaltic pavements.
 4. Use of drainage structures along roadway to capture the flow of water over the pavement.



Open graded Asphalt



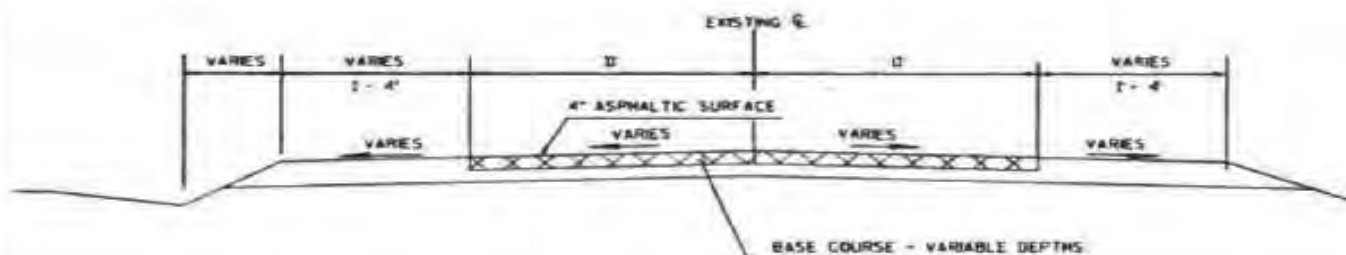
LOGITUDINAL SLOPE

- Minimum longitudinal gradient is more important for a curbed pavement than for an uncurbed pavement.
- Gutter grades should not be less than 0.5% (absolute minimum is 0.3%)
- In sag vertical curves, a minimum slope of 0.3% should be maintained within 15 meters (50 ft.) of the low point of the curve.

Cross (Transverse) Slope

- Acceptable range of cross slopes as specified in AASHTO's policy on geometric design of highway and streets.

Normal Pavement Cross Slopes.	
Surface Type	Range in Rate of Surface Slope m/m (ft/ft)
High-Type Surface	
2-lanes	0.015 - 0.020
3 or more lanes, each direction	0.015 minimum; increase 0.005 to 0.010 per lane; 0.040 maximum
Intermediate Surface	0.015 - 0.030
Low-Type Surface	0.020 - 0.060
Shoulders	
Bituminous or Concrete	0.020 - 0.060
With Curbs	≥ 0.040

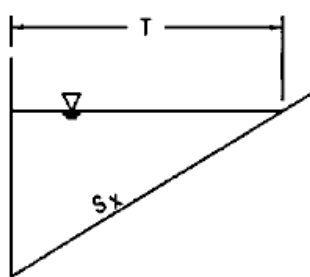


EXISTING TYPICAL CROSS SECTION

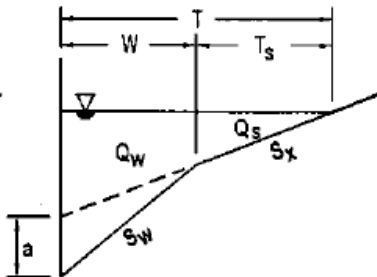
STA. 100+50 - 1080+70

Curb and Gutter

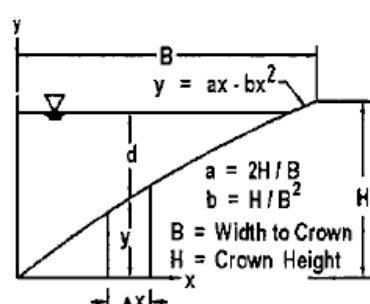
Typical Gutter Sections



1. Uniform Section

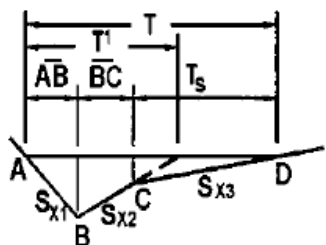


2. Composite Section

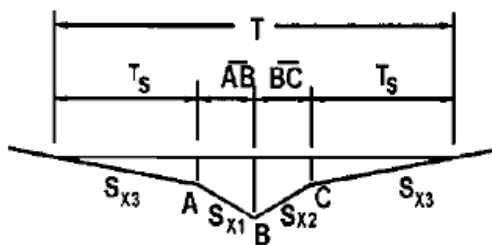


3. Parabolic Section

a. Conventional Curb And Gutter Section

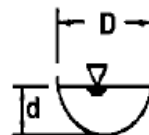


1. "V"-Shape Gutter



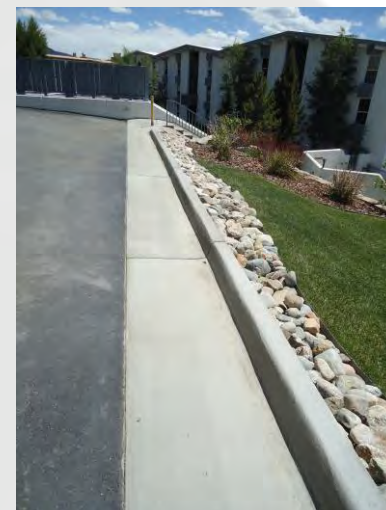
2. "V"-Shape Median

b. Shallow Swale Sections



3. Circular

Gutter and Curb Designs



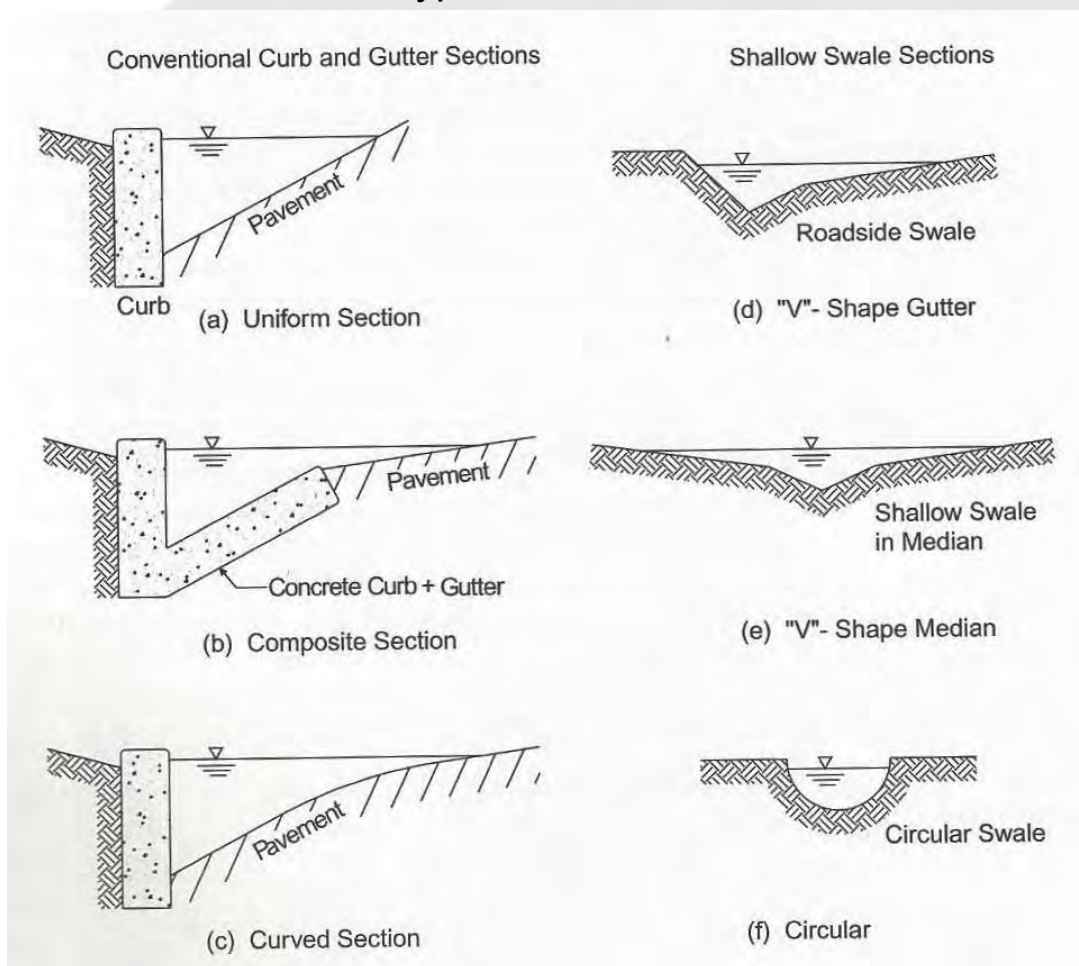
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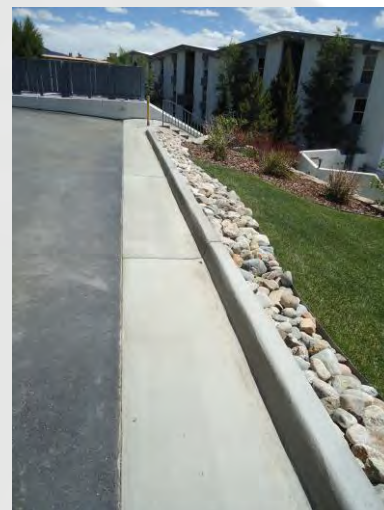
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Curb and Gutter

Typical Gutter Sections



Gutter and Curb Designs



[4]



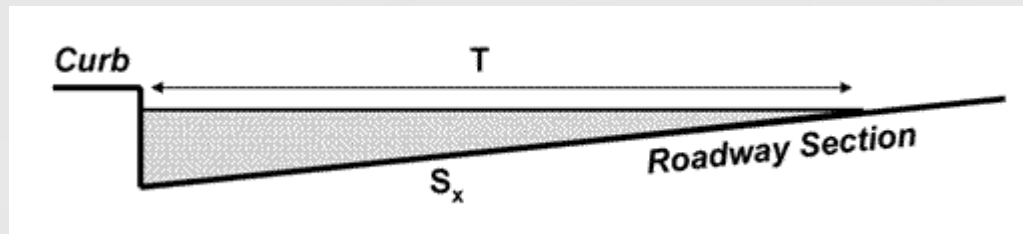
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Flow in Gutters

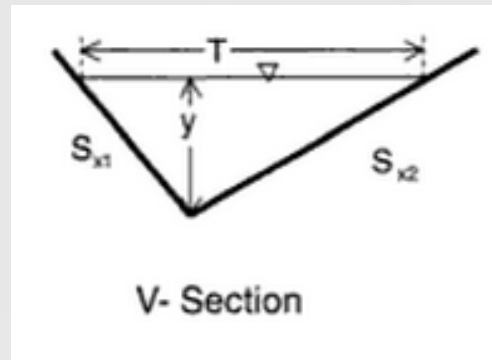
Gutter sections can be categorized as:

- Conventional – Usually have a triangular shape.



[9]

- Shallow swale type – Typically have a V-shape or circular sections.



[10]

Spread on pavement and flow depth at curb are often used as criteria for spacing pavement drainage inlets

Flow in Gutters

Capacity Relationship:

- Gutter flow calculations are necessary to establish the spread of water.
- Use a modification of Manning's equation:

$$Q = (K_u/n) S_x^{1.67} S_L^{0.5} T^{2.67} \quad T = [(Qn) / (K_u S_x^{1.67} S_L^{0.5})]^{0.375}$$

where:

$K_u = 0.376$ (0.56 in English units)

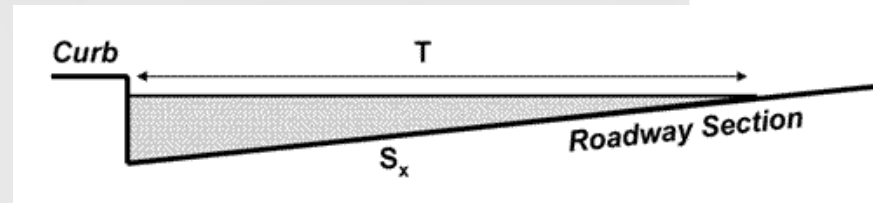
n = Manning's coefficient

Q = Flow rate m^3/s (ft^3/s)

T = Width of flow (spread)

S_x = Cross slope, m/m ($ft./ft.$)

S_L = Longitudinal slope, m/m ($ft./ft.$)



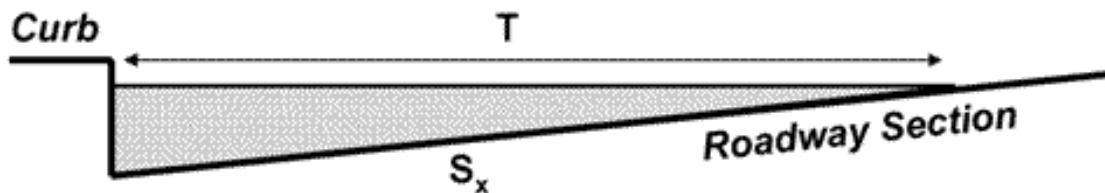


Flow in Gutters

Manning's n for Street and Pavement Gutters.	
Type of Gutter or Pavement	Manning's n
Concrete gutter, troweled finish	0.012
Asphalt Pavement:	
Smooth texture	0.013
Rough texture	0.016
Concrete gutter-asphalt pavement:	
Smooth	0.013
Rough	0.015
Concrete pavement:	
Float finish	0.014
Broom finish	0.016
For gutters with small slope, where sediment may accumulate, increase above values of "n" by	0.002

Example: Uniform Gutter Sec.

- **Given:** Gutter section illustrated in Figure 4-1 a.1.
 - $SL = 0.010 \text{ m/m (ft/ft)}$
 - $S_x = 0.020 \text{ m/m (ft/ft)}$
 - $n = 0.016$
- **Find:** (1) Spread at a flow of $0.05 \text{ m}^3/\text{s}$ ($1.8 \text{ ft}^3/\text{s}$)
- (2) Gutter flow at a spread of 2.5 m (8.2 ft)



Example: Uniform Gutter Sec.

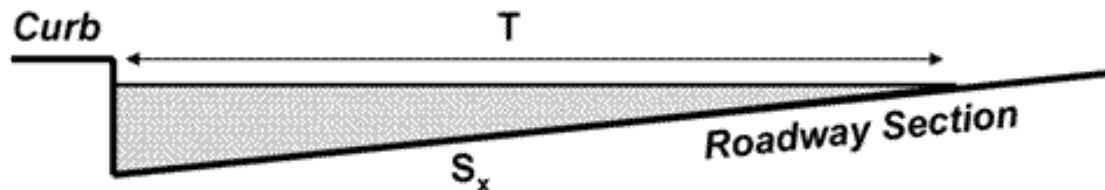
- Solution (1):**

Step 1. Compute spread, T, using Equation:

$$T = \left[\frac{Q n}{K_u S_x^{1.67} S_L^{0.5}} \right]^{0.375}$$

$$T = \left[\frac{(0.5)(0.016)}{(0.376)(0.02)^{1.67} (0.01)^{0.5}} \right]^{0.375}$$

$$T = 2.7m$$





Example: Uniform Gutter Sec.

- **Solution (2):**
- Step 1. Using Equation: $Q = (K_u/n) S_x^{1.67} S_L^{0.5} T^{2.67}$

with $T = 2.5$ m and the information given above, determine Qn .

$$Qn = K_u S_x^{1.67} S_L^{0.5} T^{2.67}$$

$$Qn = (0.376)(0.02^{1.67})(0.01)^{0.5}(2.5)^{2.67}$$

$$Qn = 0.00063 \text{ m}^3/\text{s}$$

$$Q = \frac{0.00063}{0.016}$$

$$Q = 0.039 \text{ m}^3/\text{s}$$

Composite Gutter Section

- Section divided into discharge within the depressed (gutter) section and the discharge in the roadway above depressed section

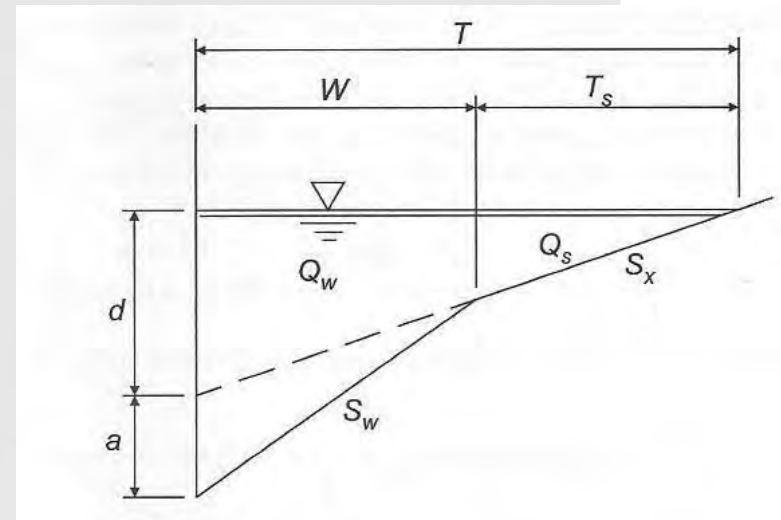
$$Q = Q_w + Q_s$$

- The spread: $T = W + T_s$
- Depth of flow at curb at faze

$$y = d + a = T S_x + a$$

- Gutter depression cross slope

$$S_w = S_x + a/W$$



Composite Gutter Section

- Use Manning's equation to get discharge within the roadway above depressed section

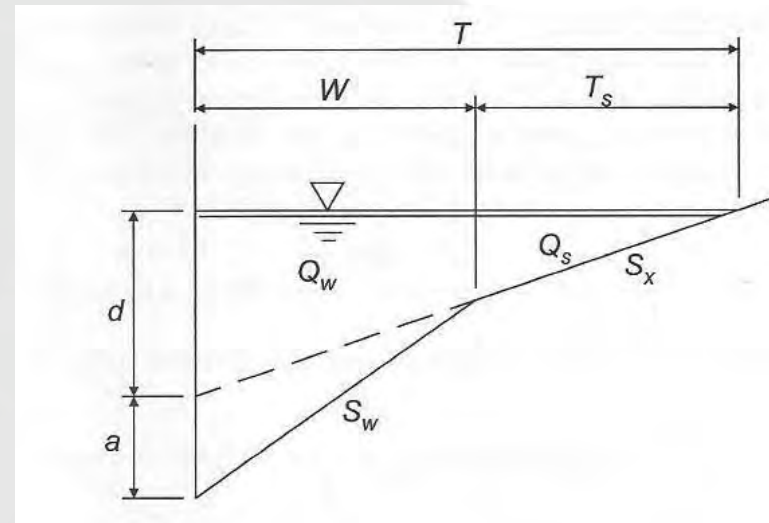
$$Q_s = \frac{K_u}{n} S_x^{5/3} T_s^{8/3} S_o^{1/2}$$

- Express discharge in the depressed section as a fraction of the total discharge

$$E_0 = \frac{Q_w}{Q}$$

- Substituting total Q and solving

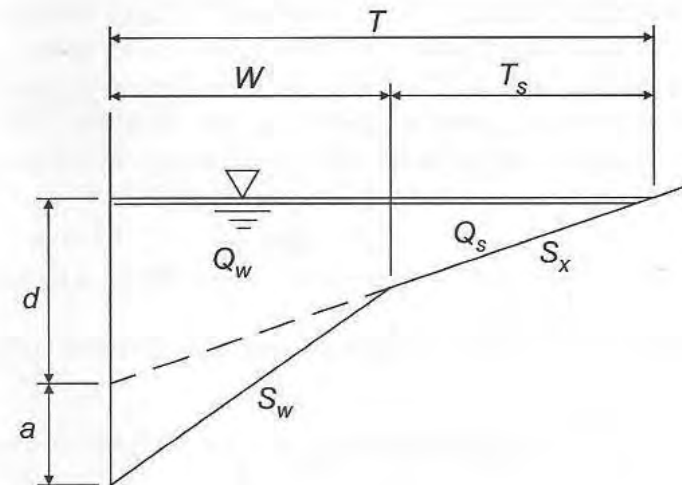
$$Q = \frac{Q_s}{1 - E_0}$$



Composite Gutter Section

- The fraction of the total discharge flowing in the depressed gutter section is:

$$E_0 = \left\{ 1 + \frac{\frac{S_w}{S_x}}{\left(1 + \frac{\frac{S_w}{S_x}}{\frac{T}{W} - 1} \right)^{8/3} - 1} \right\}^{-1}$$



Example: Composite Gutter Secn.

- **Given:** Gutter section illustrated in Figure with:

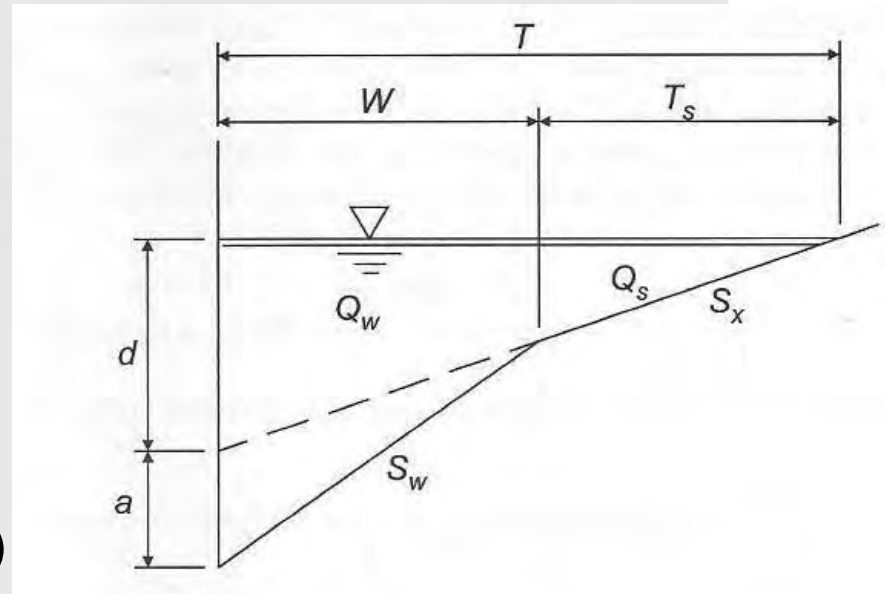
$$W = 0.6 \text{ m (2 ft)}$$

$$S_L = 0.01$$

$$S_x = 0.020$$

$$n = 0.016$$

Gutter depression, $a = 50 \text{ mm (2 in)}$



- **Find:**

1. (1) Gutter flow at a spread, T , of 2.5 m (8.2 ft)
2. (2) Spread at a flow of $0.12 \text{ m}^3/\text{s}$ ($4.2 \text{ ft}^3/\text{s}$)



Example: Composite Gutter Secn.

Solution (1):

Step 1. Compute the cross slope of the depressed gutter, S_w , and the width of spread from the junction of the gutter and the road to the limit of the spread, T_s .

$$S_w = \frac{a}{W} + S_x$$
$$S_w = \frac{50/1000}{0.6} + 0.02 = 0.103 \text{ m/m}$$
$$T_s = T - W = 2.5\text{m} - 0.6\text{m} = 1.9\text{m}$$

Step 2. From Manning's Equation (using T_s) compute Q_s :

$$Q_s n = K_u S_x^{1.67} S_L^{0.5} T_s^{2.67}$$

$$Q_s n = (0.56)(0.02^{1.67})(0.01)^{0.5}(6.2)^{2.67}$$

$$Q_s n = 0.011 \text{ ft}^3/\text{s}$$

$$Q_s = \frac{Q_s n}{n} = \frac{0.011}{0.016} = 0.69 \text{ ft}^3/\text{sec}$$



Example: Composite Gutter Secn.

Step 3. Determine the fraction of total flow to depressed gutter flow by using Equation

$$E_0 = \left\{ 1 + \frac{\frac{S_w}{S_x}}{\left(1 + \frac{\frac{S_w}{S_x}}{\frac{T}{W^{-1}}} \right)^{8/3} - 1} \right\}^{-1}$$

$$\frac{T}{W} = \frac{8.2}{2} = 4.10$$

$$\frac{S_w}{S_x} = \frac{0.103}{0.020} = 5.15$$

$$E_0 = 1 / \left\{ 1 + \left[\frac{5.15}{\left(1 + \frac{5.15}{4.10 - 1} \right)^{2.67} - 1} \right] \right\}$$

$$E_0 = 0.70$$

Step 4. Determine the composite gutter flow, Q, using equation

$$Q = \frac{Q_s}{1 - E_0}$$

$$Q = \frac{0.69}{1 - 0.7}$$

$$Q = 2.3 \text{ ft}^3/\text{sec}$$

Example: Composite Gutter Secn.

Solution (2):

Since the spread cannot be determined by a direct solution, an iterative approach must be used.

Step 1. Try $Q_s = 1.4 \text{ ft}^3/\text{s}$

Step 2. Compute Q_w

$$Q_w = Q - Q_s = 4.2 - 1.4$$

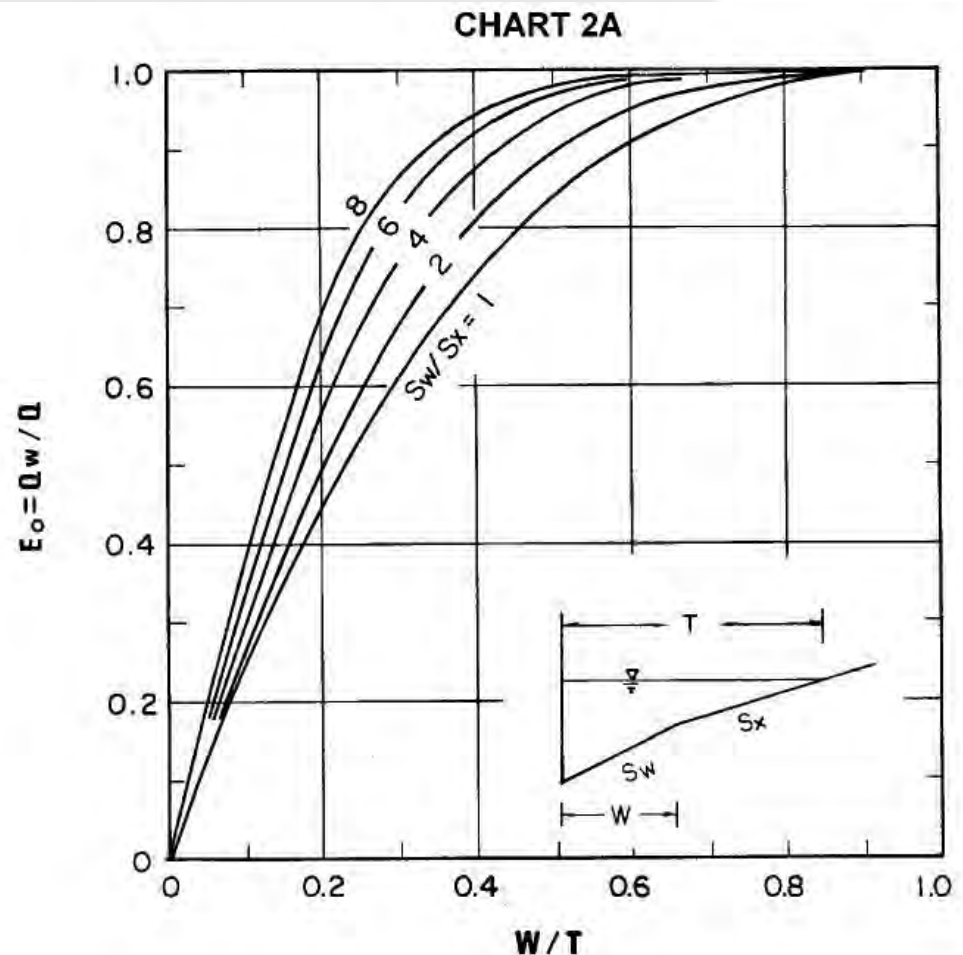
$$Q_w = 2.8 \text{ ft}^3/\text{s}$$

Step 3. Using Equation or Chart determine W/T ratio

$$E_o = Q_w / Q = 2.8 / 4.2 = 0.67$$

$$S_w/S_x = 0.103 / 0.020 = 5.15$$

$$W/T = 0.23$$



Example: Composite Gutter Secn.

Solution (2):

Step 4. Compute spread based on the assumed Q_s

$$T = W / (W/T) = 2.0 / 0.23$$

$$T = 8.7 \text{ ft}$$

Step 5. Compute T_s based on assumed Q_s

$$T_s = T - W = 8.7 - 2.0 = 6.7 \text{ ft}$$

Step 6. Use Manning's Eq. to determine Q_s for computed T_s

$$Q_s n = K_u S_x^{1.67} S_L^{0.5} T_s^{2.67}$$

$$Q_s n = (0.56)(0.02^{1.67})(0.01)^{0.5}(6.7)^{2.67}$$

$$Q_s n = 0.0131 \text{ ft}^3/\text{s}$$
$$Q_s = \frac{Q_s n}{n} = \frac{0.0131}{0.016} = 0.82 \text{ ft}^3/\text{sec}$$

Step 7. Compare computed Q_s with assumed Q_s .

$$Q_s \text{ assumed} = 1.4 > 0.82 = Q_s \text{ computed}$$

Not close - try again

Example: Composite Gutter Secn.

Step 8. Try a new assumed Q_s and repeat

Steps 2 through 7.

Assume $Q_s = 1.9 \text{ ft}^3/\text{s}$

$Q_w = 4.2 - 1.9 = 2.3 \text{ ft}^3/\text{s}$

$E_o = Q_w / Q = 2.3/4.2 = 0.55$

$S_w / S_x = 5.15$

$W / T = 0.18$

$T = 2.0/0.18 = 11.1 \text{ ft}$

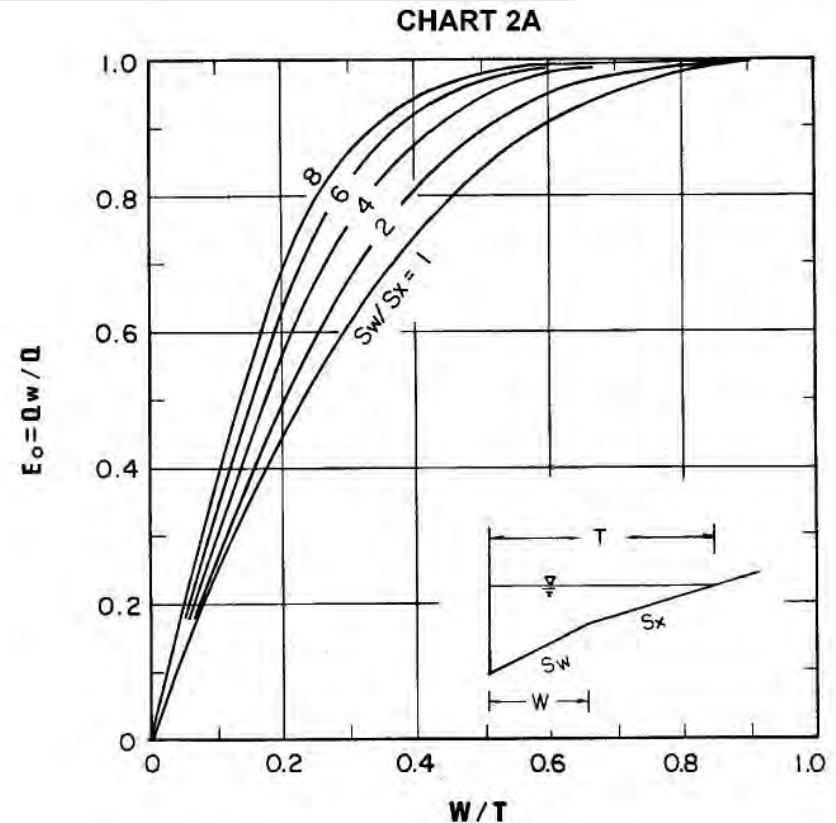
$T_s = 11.1 - 2.0 = 9.1 \text{ ft}$

$Q_{s,n} = 0.30 \text{ ft}^3/\text{s}$

$Q_s = 0.30 / 0.016 = 1.85 \text{ ft}^3/\text{s}$

$Q_s \text{ assumed} = 1.9 \text{ ft}^3/\text{s}$ close to

$1.85 \text{ ft}^3/\text{s} = Q_s \text{ computed}$







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Module 8

Hydraulics: Pump Systems



PUMP SYSTEMS



Introduction

- Used to remove storm water from highway sections that can not be drained by gravity.
- High operational costs.
- Recommended only where other systems are not feasible.



Design Considerations

- Pump station design presents the designer with a challenge to provide a cost-effective drainage system that meets the need of the project.
 - Wet-pit vs. dry-pit
 - Type of pumps
 - Number and capacity of pumps
 - Motor
 - Peak flow vs. storage
 - Force main vs. gravity
 - Above grade vs. below grade
 - Monitoring Systems
 - Backup systems
 - Maintenance requirements
- To assure cost-effectiveness, the designer should assess each choice and develop economic comparisons of alternatives based on engineering judgment and experience.



Location

- Pump stations should be located near the low point in the highway drainage system they are intended to serve.
- The stations should be located on high ground to obtain access if the highway becomes flooded.
- Soil borings should be made during the selection of the site to determine the allowable bearing capacity and identify any potential problems.
- Architectural and landscaping decisions should be made in the location for above-round stations so the station will blend into the surrounding community.



Hydrology

- Pump stations serving major expressways and arterials are usually designed to accommodate 50-year storm.
- It is desirable to check the drainage system for the 100-year storm to determine the extent of flooding and the associated risk.
- Drainage area tributary to the station should be kept as small as possible.
- By-pass or pass-through all possible drainage to reduce pumping requirements.
- Avoid future increases in pumping by isolating the drainage area.
- Hydrologic design should be based on the ultimate development of the area which must drain to the station.



Hydrology cont.

- Storage must be considered in addition to that which exists in the wet well at all pump station sites.
- Additional storage, skillfully designed, may greatly reduce the peak pumping rate required.
- Storage well is usually located below normal ground level because of the nature of the sites where highway pump stations are located.
- If storage is used to reduce peak flow rates, a routing procedure must be used to design the system. This procedure integrates the following elements to determine the required pump rate:
 - Inflow hydrograph
 - Stage-storage relationship
 - Stage-discharge relationship



Collection Systems

- To avoid siltation problems in the collection system, a minimum grade that produces a velocity of 1 m/s (3 ft./s) in the pipe while flowing full is suggested.
- The inlet pipe should enter the station perpendicular to the line of pumps.
- The inflow should distribute itself equally to all pumps.
- Collector lines should preferably terminate at a forebay or storage box. However they may discharge directly into the station.



Collection Systems cont.

- The capacity of the collectors and storage are critical to provide cycling time for the pump and must be carefully calculated.
- To minimize siltation problems in storage units, a minimum grade of 2% should be used.
- It is recommended that screens be used to prevent large objects from entering the system and possible damaging the pumps.
- Larger debris may also be screened at the surface or inside the wet well/storage system.

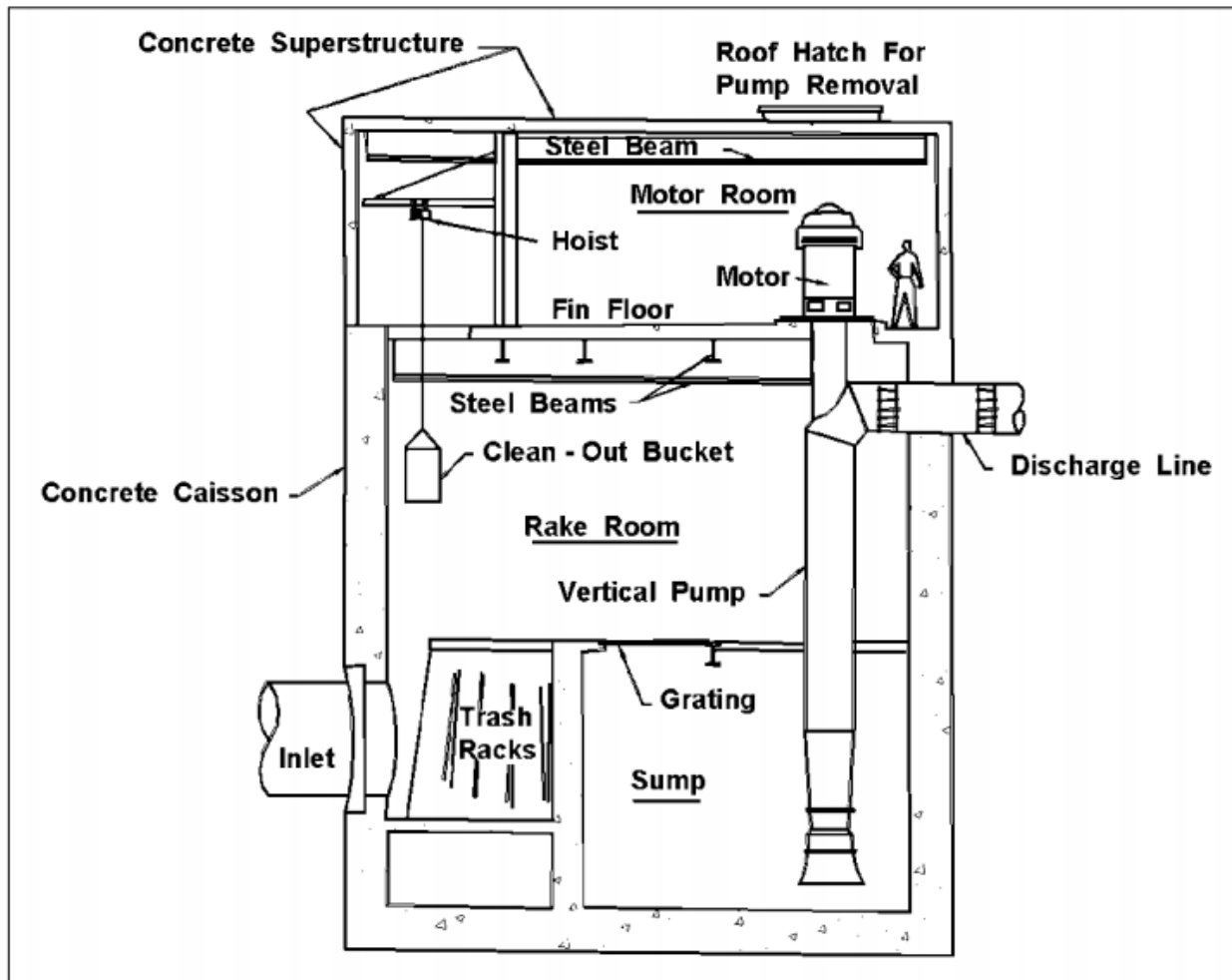


Station Types

- **Wet-Pit Station**

- In this type of station, the pumps are submerged in a wet well or sump with the motors and the controls located overhead.
- The motor is commonly connected to the pump by a long drive shaft located in the center of the riser pipe.
- Another type of wet-pit design involves the use of submersible pumps.
- This pumps requires less maintenance and less horsepower because a long drive shaft is not required.
- Submersible pumps allow for convenient maintenance in wet-pit stations because of easy pump removal.

Typical Axial Pump Wet-Pit Station





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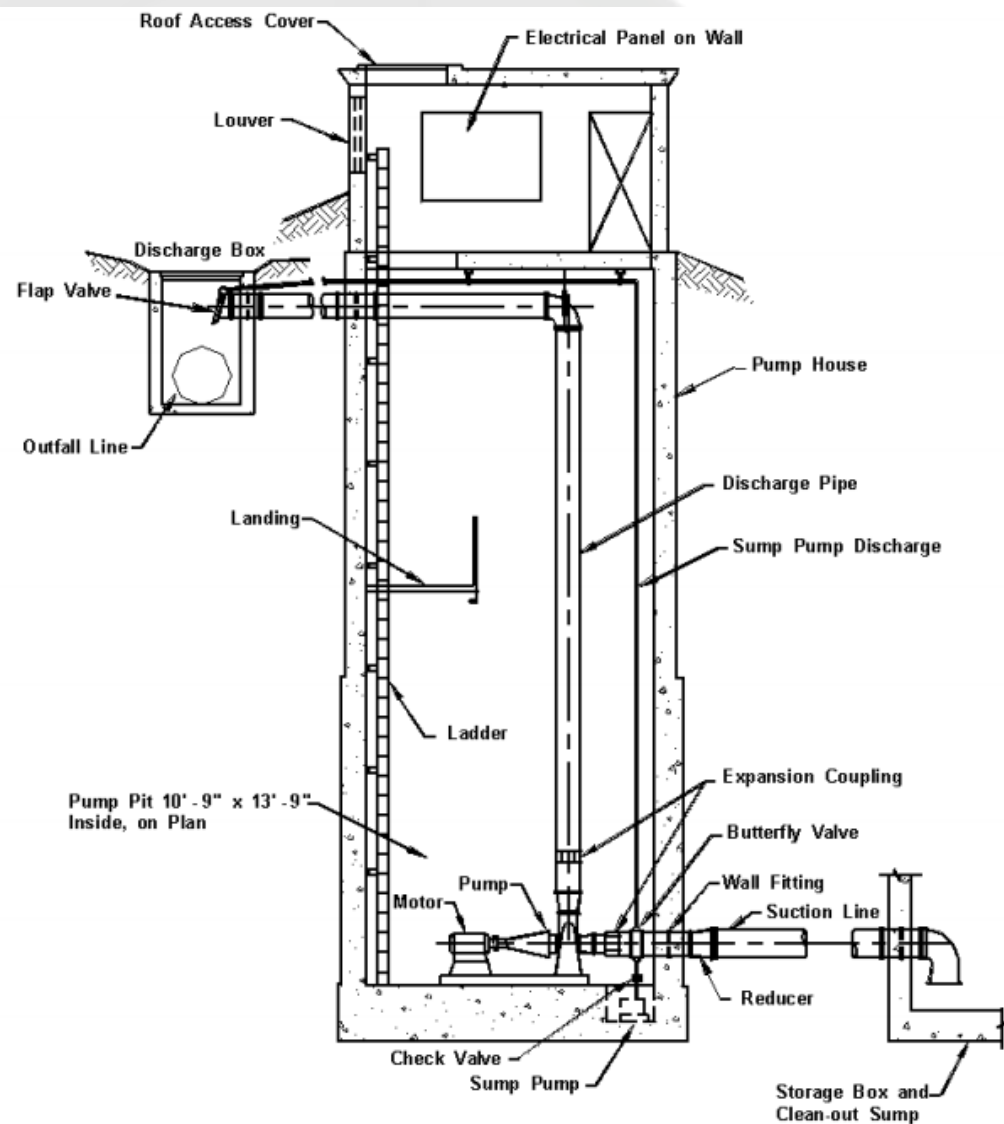


Station Types cont.

- **Dry-Pit Station**

- This type of station consist of two separate elements: The storage box or wet well and the dry well.
- Storm water is stored in the wet well which is connected to the dry well by horizontal suction piping.
- Centrifugal pumps are usually used and located on the floor of the dry well.
- The main advantage of this type of station is the availability of a dry area for personnel to perform routine and emergency pump and pipe maintenance.

Typical Centrifugal Pump Dry-Pit Station



Pump Types

- The most common types of storm water pumps are:
 - **Axial flow (propeller)**
 - **Radial flow (impeller)**
 - **Mixed flow (combination of axial and radial flow).**



[7]



[6]



Submergence

- The submergence depth is very important if a submersible pump is used.
- Is the depth of water above the pump inlet necessary to prevent cavitation or vortexing.
- Varies with pump type and speed and atmospheric pressure.
- The dimension is usually provided by manufacturer.
- The available net positive suction head (NPSH) should be calculated and compared to the manufacturer's requirement.



Centrifugal Pumps

- **Terminology:**
- **Head:** Energy per unit weight. Have units of length.
- **Pressure Head:** Amount of energy in water due to water pressure.

$$h_{press} = \frac{P}{\gamma}$$

P = static pressure.

γ = specific weight



Terminology

- **Friction head: (hf):**

$$h_v = \frac{V^2}{2g}$$

- Head required to overcome resistance to flow in the pipes.

- **Velocity head: (hv):**

- Specific kinetic energy of fluid. Also called dynamic head.

- **Static suction head (hz(s)):**

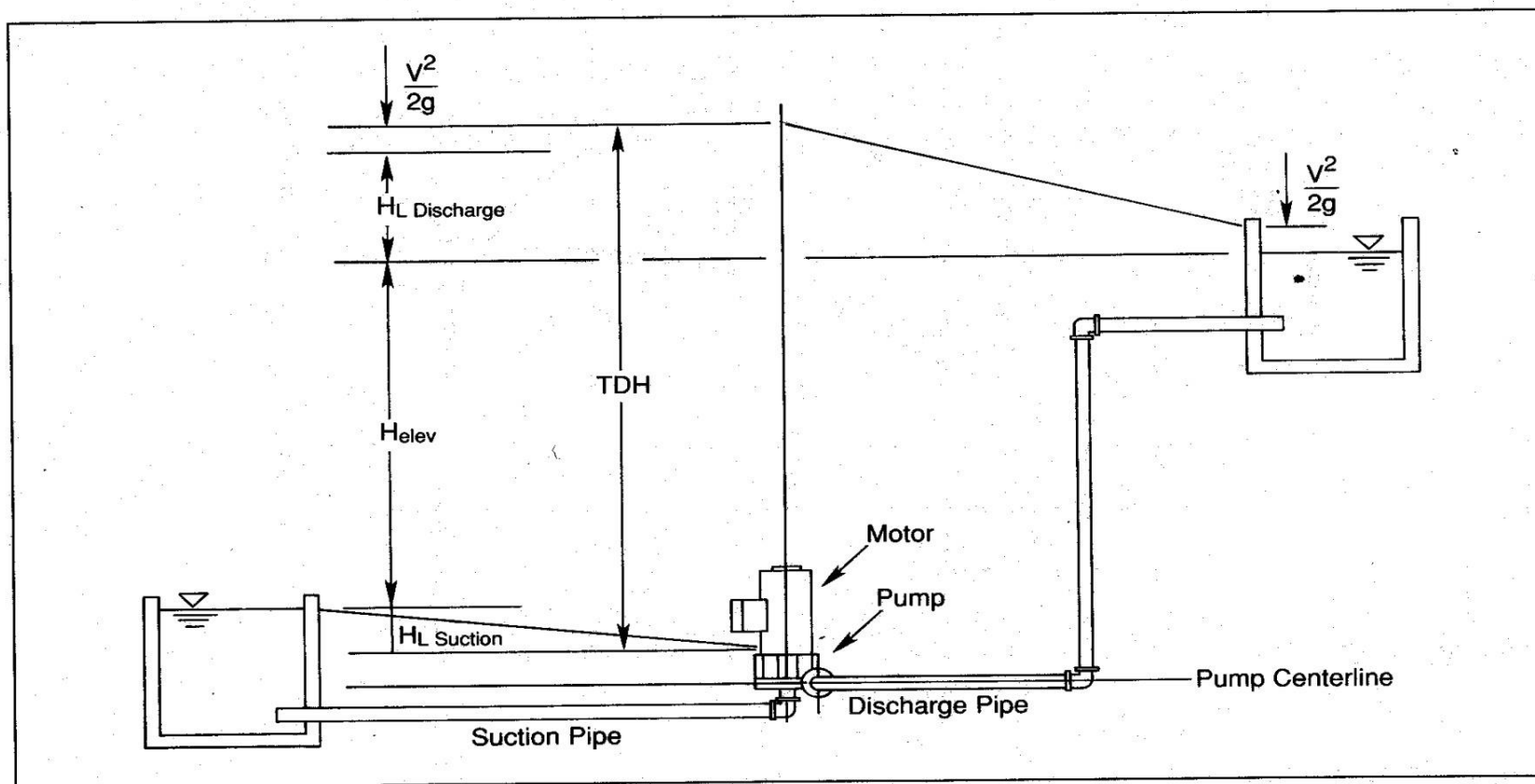
- Vertical distance above the centerline of the pump inlet to the free level of the fluid source. If the free level of the fluid is below the pump inlet hz will be negative and is known as **static suction lift**.



Terminology

- **Static discharge head (hz(d)):**
 - The vertical distance above the centerline of the pump inlet to the point of free discharge.
- **Total Dynamic Head: (TDM):**
 - Difference in height between the hydraulic grade line (HGL) on the discharge side of the pump and the (HGL) on the suction side of the pump.

Pump System





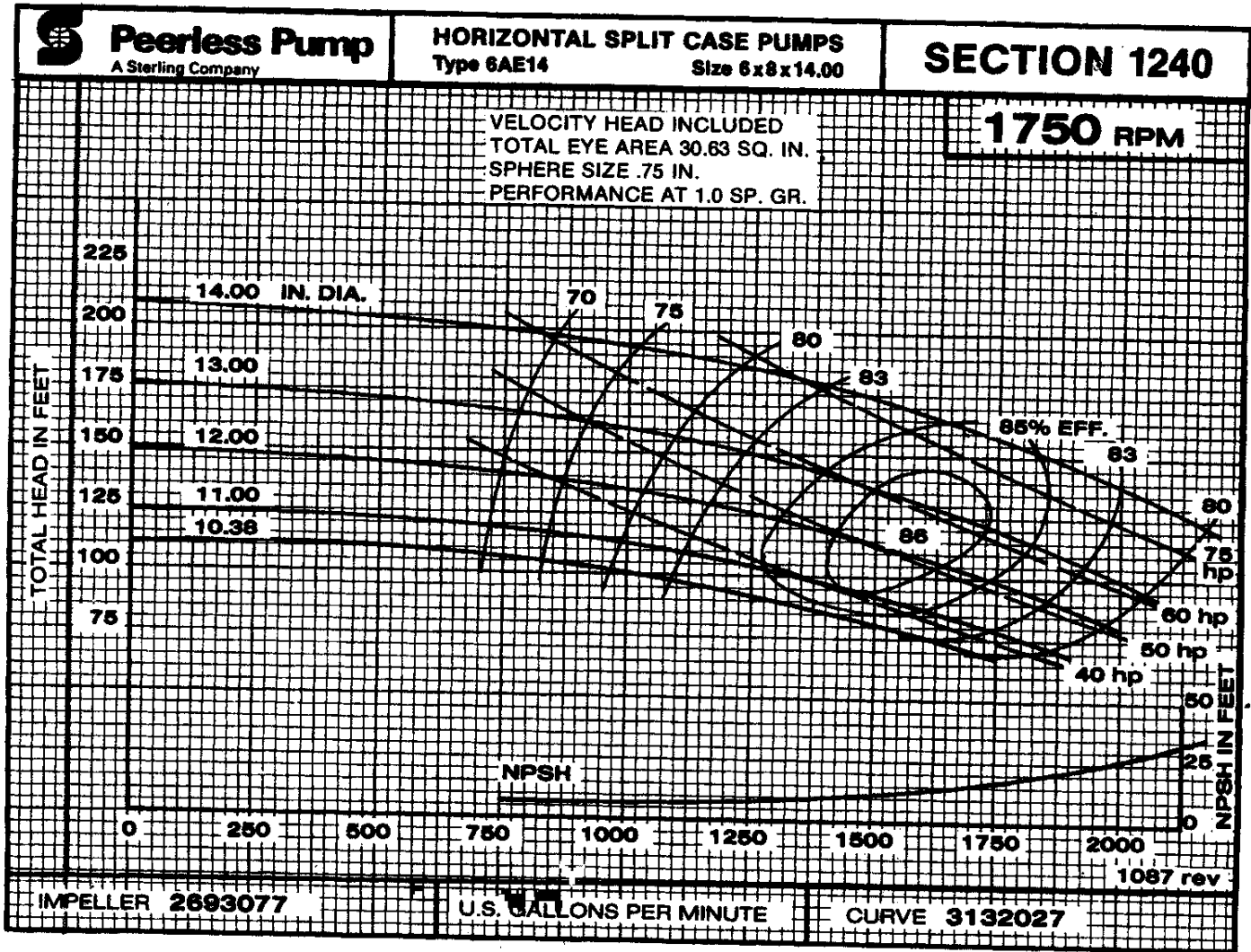
Pump Characteristic Curve

Pump Curves

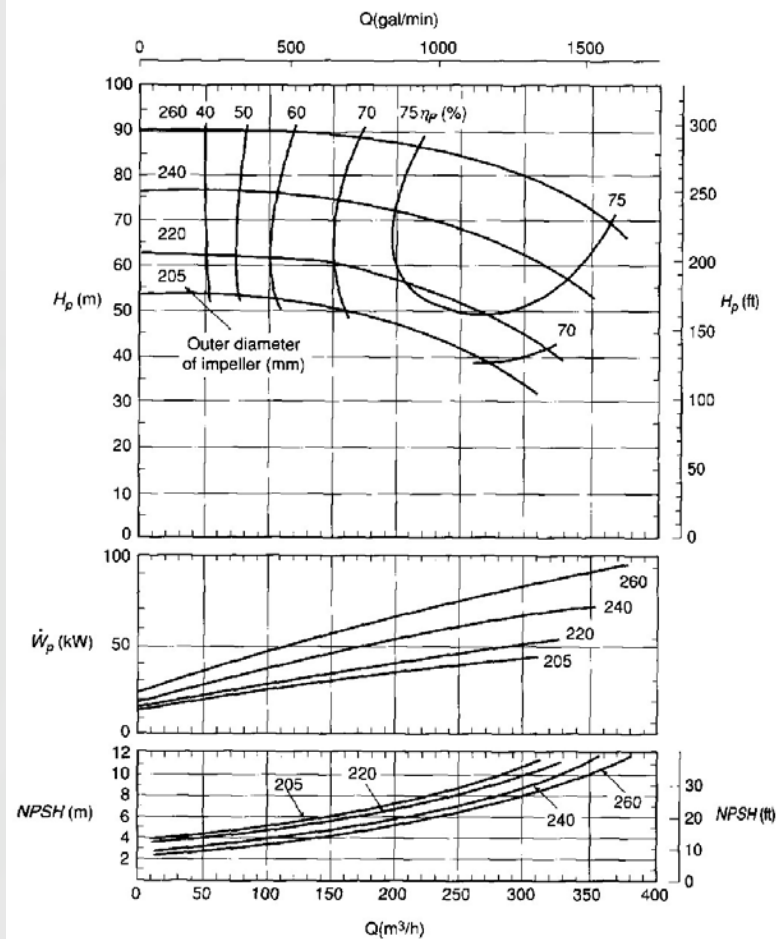
- Manufacturers test each centrifugal pump, type and publish a pump curve for each pump type and size.
- Every centrifugal pump operates at less than 100% efficiency, generally between 50% and 85%
- The pump's efficiency can decrease dramatically if the pump is operated at other than its optimum discharge.



Pump Characteristic Curve



Pumps Operation

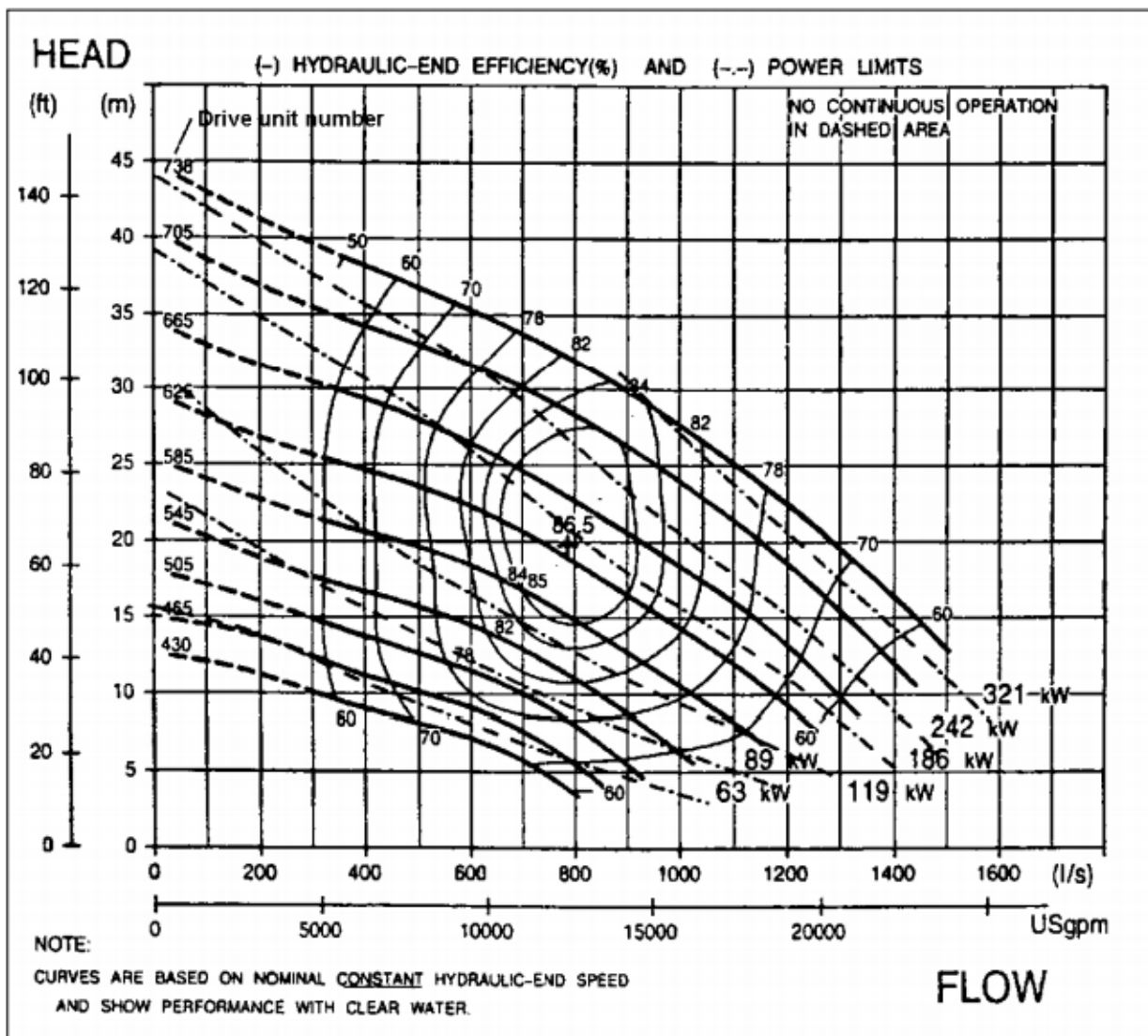




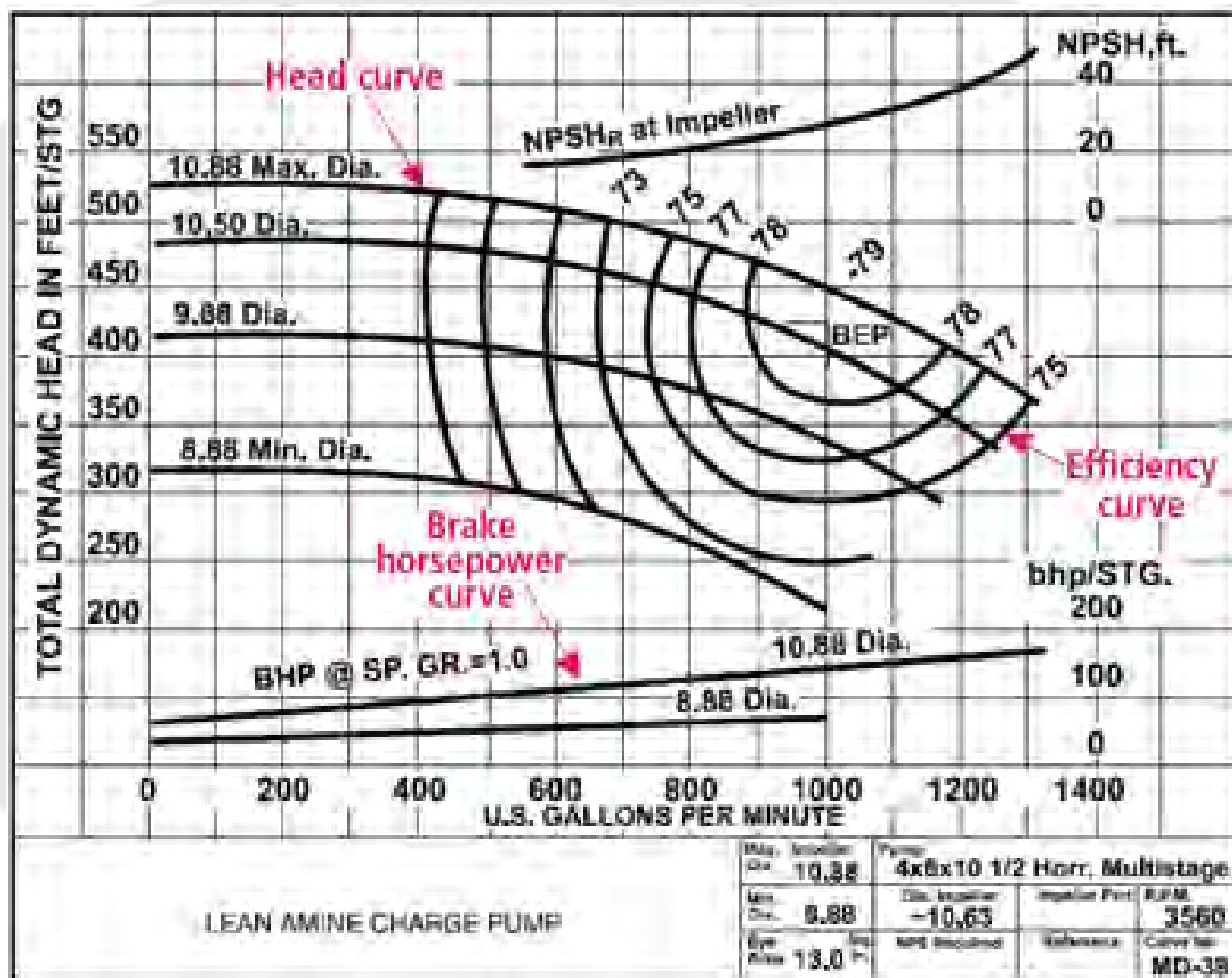
Discharge Head and System Curve cont.

- Because pumps operate over a range of water levels, the designer is required to specify at least three (3) points on the performance curve.
- These points are the conditions for the highest TDH, the design head and the lowest head expected over the full operating range of the pump.
- The designers must study the curves of TDH vs. pump capacity provided by manufacturers in order to understand the pump conditions throughout the full range of head that the pump will operate under.

Typical Performance Curve



Typical Performance Curve

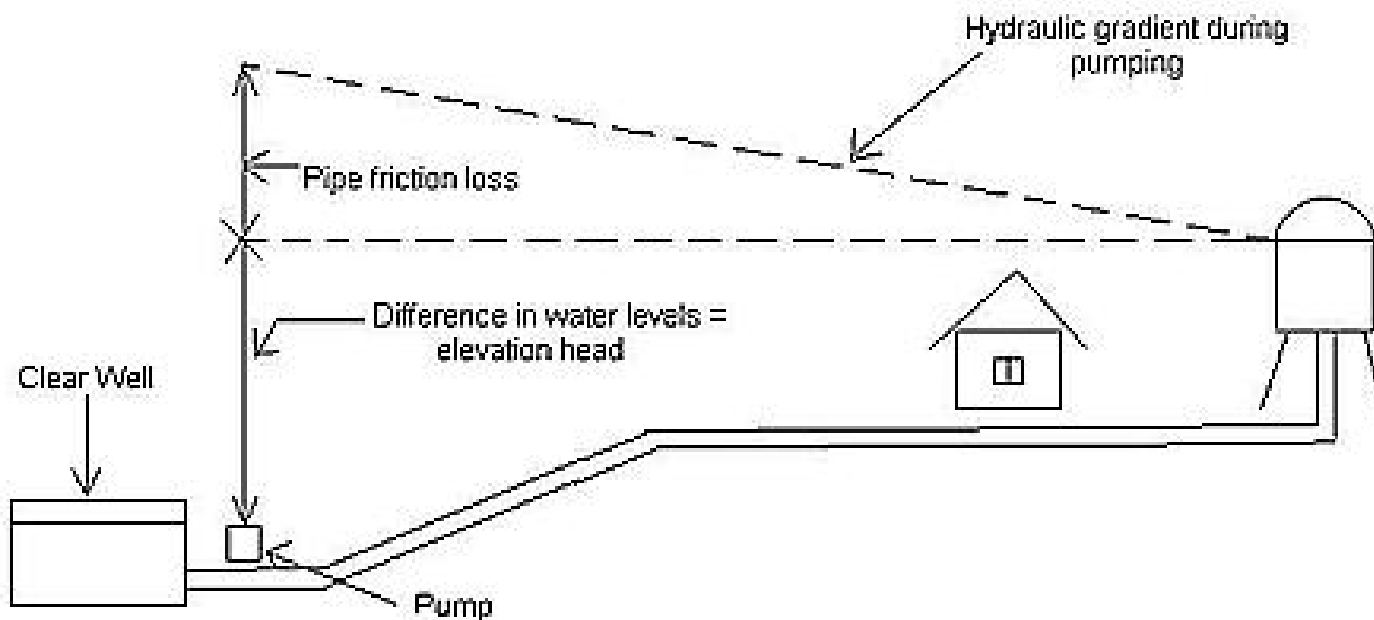




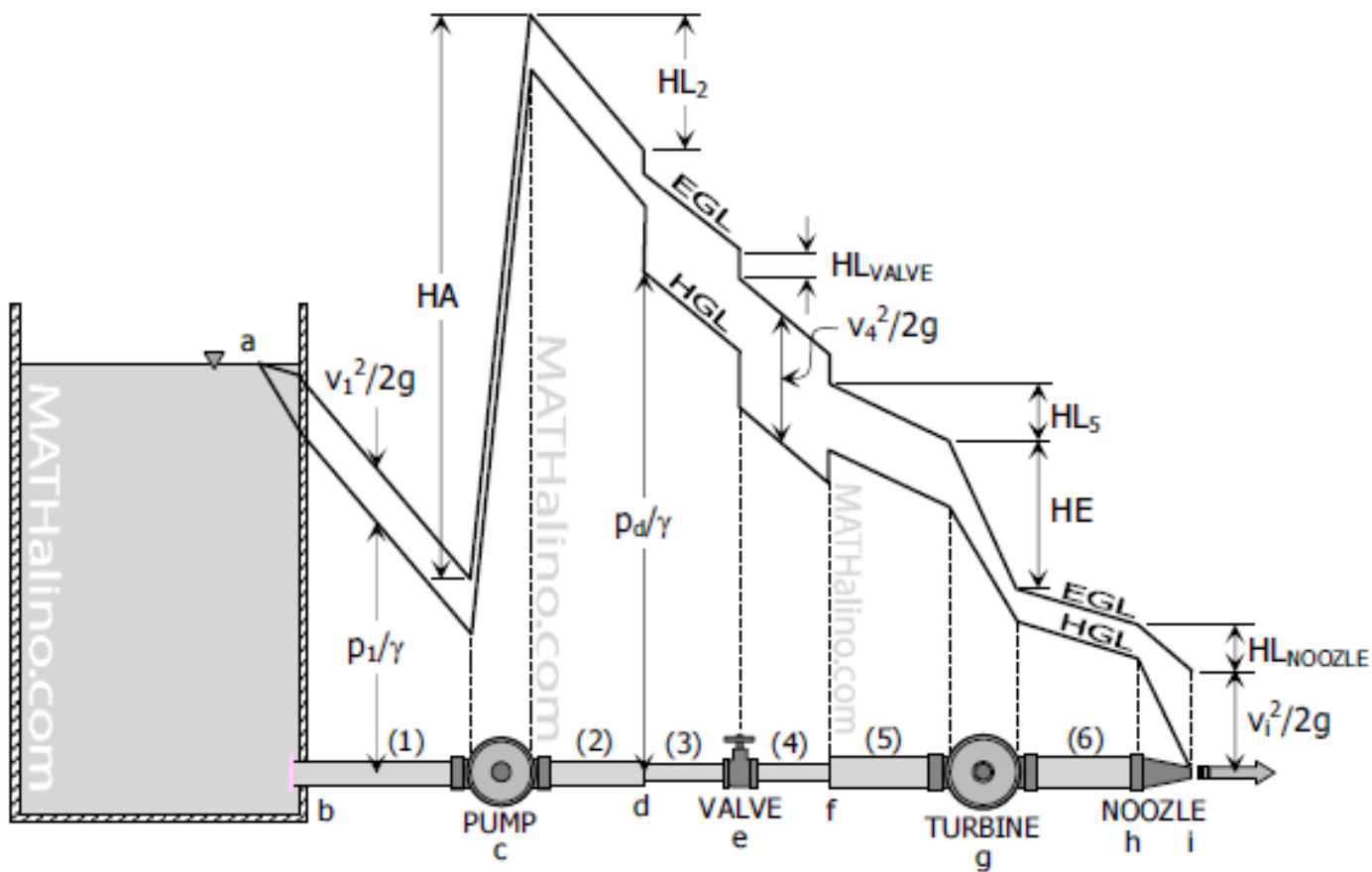
Discharge Head and System Curve

- Pumps are sensitive to changes in head, therefore, all valve and bend losses should be considered in computations.
- Total dynamic head (TDH) is the combination of static head, velocity head and various head losses in the discharge system due to friction.
- TDH is computed as:
 - **$TDH = H_s + H_f + H_v + K_l$**
 - H_s = Static Head, m (ft)
 - H_f = Friction head, m (ft)
 - H_v = Velocity head, m(ft) $[V^2 / 2g]$
 - H_l = Losses through fittings, valves, etc., m (ft)

Discharge Head and System Curve cont.



Discharge Head and System Curve cont.





Pumps Operation

System curves

- Is a plot the energy required to operate the hydraulic system for different flow rates.
- Depends on the configuration of the section and discharge lines.



Pumps Operation

Operating Point

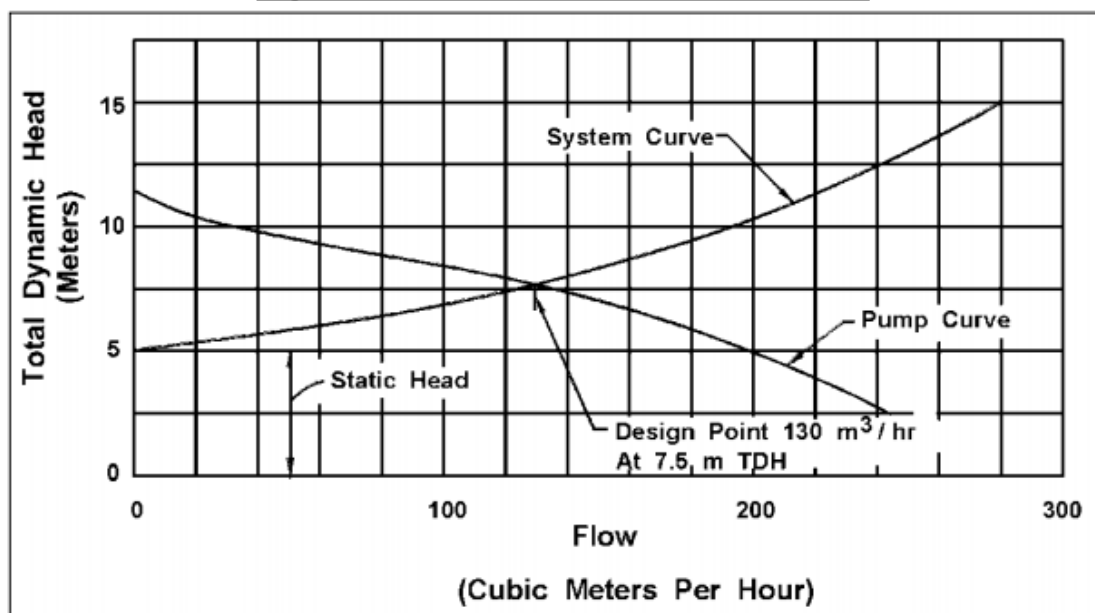
- The intersection of the pump curve (supplied by the manufacturer) and system curve locates the OPERATING POINT. This point defines the pumping head and the system discharge.
- The design operating point should be close to the highest pump efficiency.



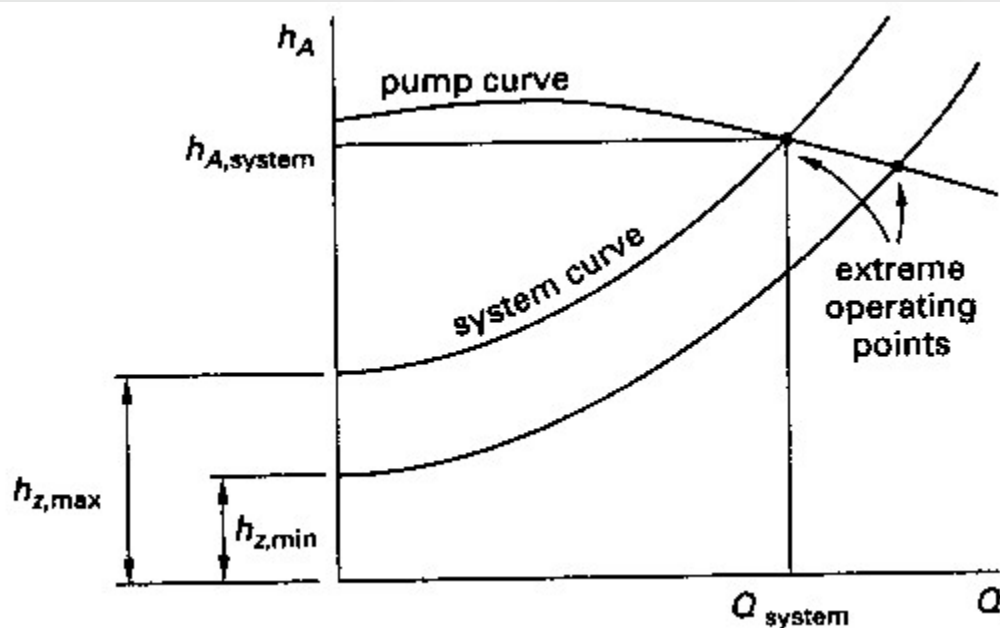
Discharge Head and System Curve cont.

- It is usual to minimize the head losses by selection of correctly sized discharge lines and other components.
- Once the head losses are computed, the system curve (Q vs. TDH) can be plotted to define the energy required to pump any flow through the system.

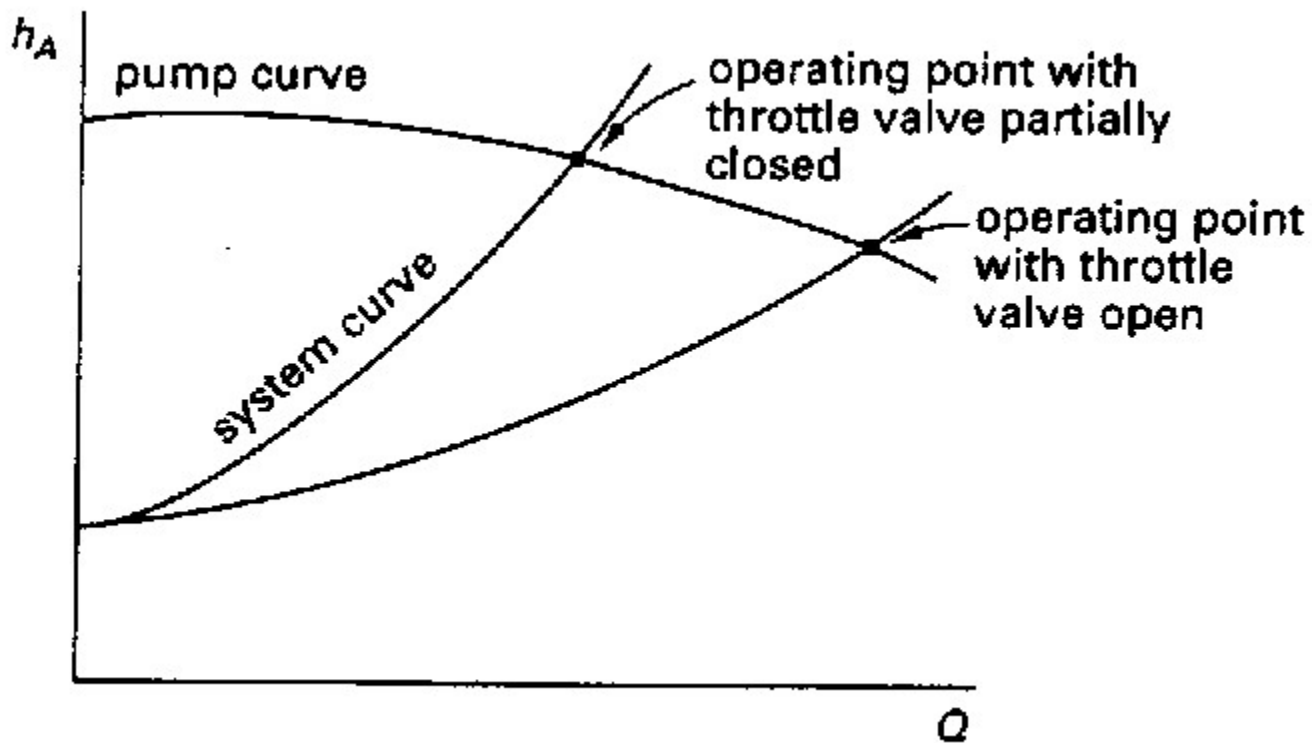
System curve Flow vs TDH



Pumps Operation



Pumps Operation





Pumping Power

- Pump output power (hydraulic power or waterpower) is the net power actually transferred to the fluid.
- Pump input power is the net power at the motor-pump. This is known as the **BRAKE POWER**.
- The difference between input and output power are the friction or mechanical losses, volumetric losses and electrical losses.



Pumping Power

- **The Pump Efficiency:**
- Is the ratio of the hydraulic power and the break-power:

$$\gamma_P = \frac{\text{hydraulic power}}{\text{brake pump power}} = \frac{\gamma Q h_p}{BPP}$$

- Pump efficiency is a function of the specific speed and the flow rate. The overall efficiency of the pump installation is the product of the pump and motor efficiencies

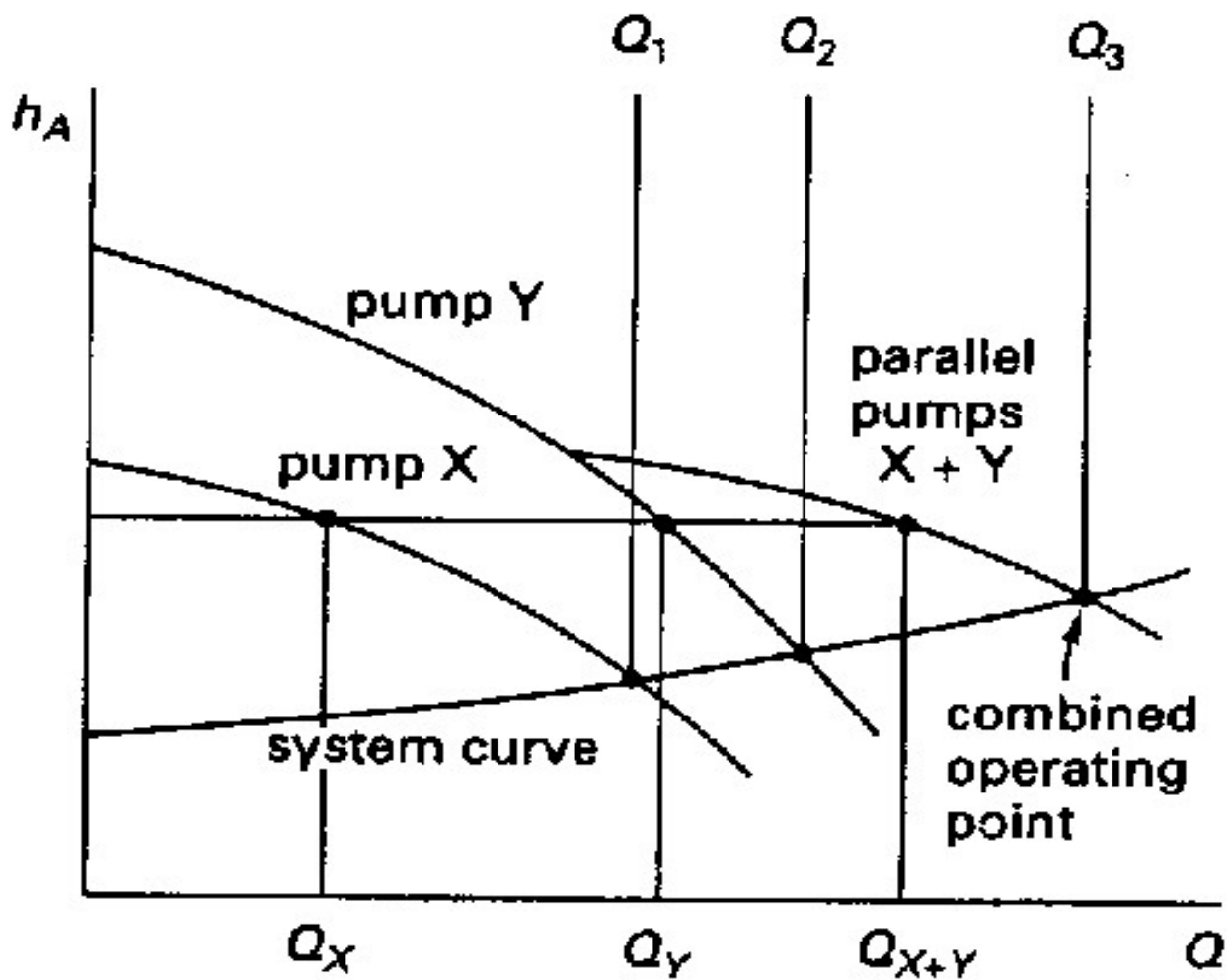
$$n = n_p n_m$$



Pumps in Parallel

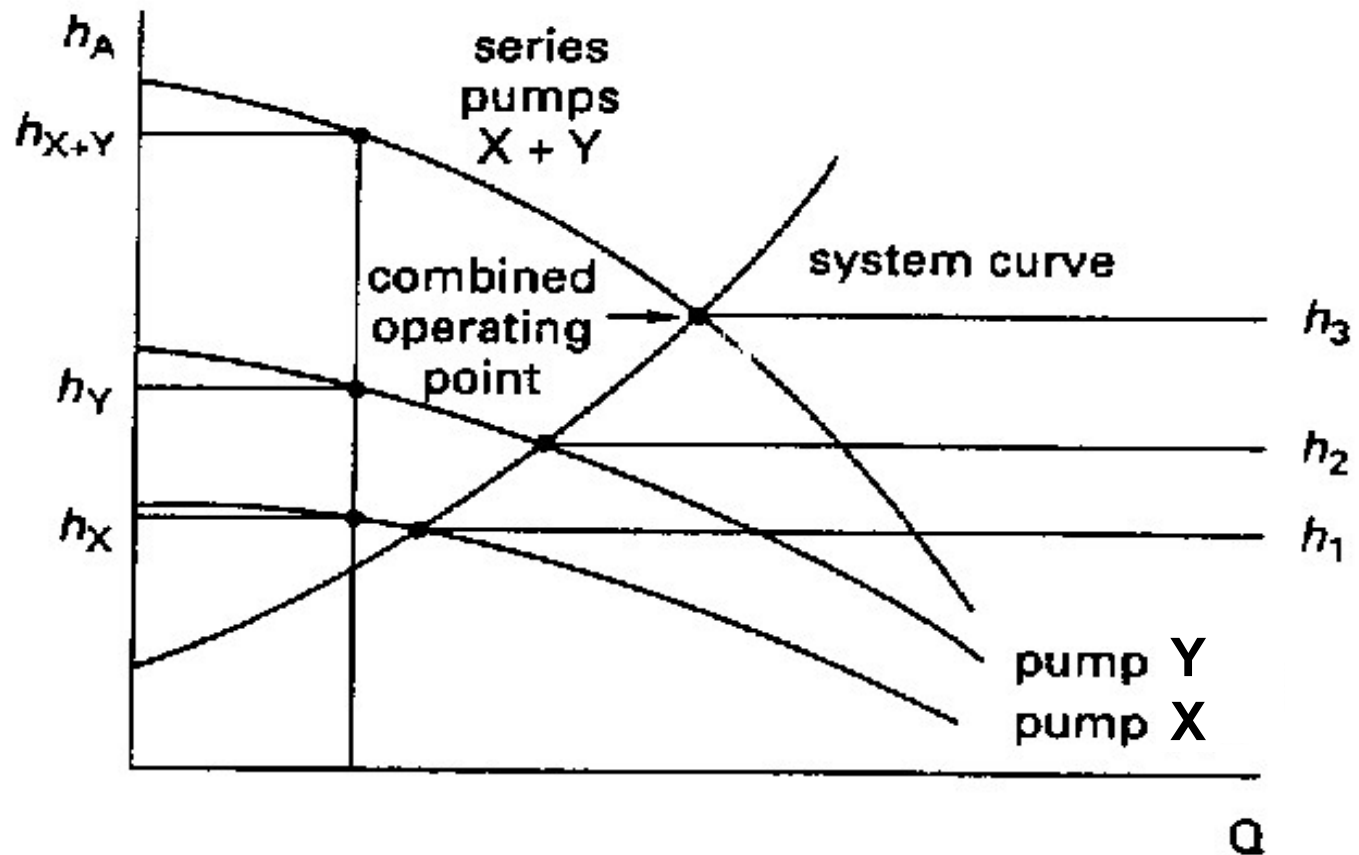
- Useful for systems with variable demand.
- Add the capacity to the number of pumps (assuming all pumps are equal), keeping the same head.
- With pump X operating alone the capacity is given by Q_1 . When pump Y is added, the capacity increases to Q_3 (with an increase in total head!).

Pumps in Parallel



Pumps in Series

- Useful to increase discharge head





Cavitation

Causes:

- High suction lift or low suction head
- Excessive pump speed
- High liquid temperature (high vapor pressure)

NPSH (Net positive suction head)

- Cavitation occurs when the available head is less than the required head for satisfactory operation.
- The minimum fluid energy required at the pump inlet for satisfactory operation is the NPSH required.



Cavitation and NPSH

- The NPSHR is a function of the pump and is given by the manufacturer
- The net positive suction head available (NPSHA) is the actual fluid energy at the inlet.

$$\text{NPSH}_A = H_s - H_{\text{loss}} = h_{res} + h_{atm} + h_{z(s)} - h_{f(s)} - h_{vp}$$

h_{res} = reservoir head

h_{atm} = atmospheric head

$h_{z(s)}$ = potential head at the suction side

$h_{f(s)}$ = friction plus minor losses in the suction side

h_{vp} = vapor pressure head (function of temperature)

Affinity or similarity laws

- Assumptions for both pumps

1. Operate in turbulent region
2. Have the same efficiency

In reality, larger pumps will be more efficient than smaller pumps.

Simplified versions are obtained if pumps have the same impeller diameter or the same speed.

$$\frac{n_1 D_1}{\sqrt{h_1}} = \frac{n_2 D_2}{\sqrt{h_2}}$$

$$\frac{Q_1}{D_1^2 \sqrt{h_1}} = \frac{Q_2}{D_2^2 \sqrt{h_2}}$$

$$\frac{P_1}{\rho_1 D_1^2 h_1^{1.5}} = \frac{P_2}{\rho_2 D_2^2 h_2^{1.5}}$$

$$\frac{Q_1}{n_1 D_1^3} = \frac{Q_2}{n_2 D_2^3}$$

$$\frac{P_1}{\rho_1 n_1^3 D_1^5} = \frac{P_2}{\rho_2 n_2^3 D_2^5}$$

$$\frac{n_1 \sqrt{Q_1}}{(h_1)^{0.75}} = \frac{n_2 \sqrt{Q_2}}{(h_2)^{0.75}}$$



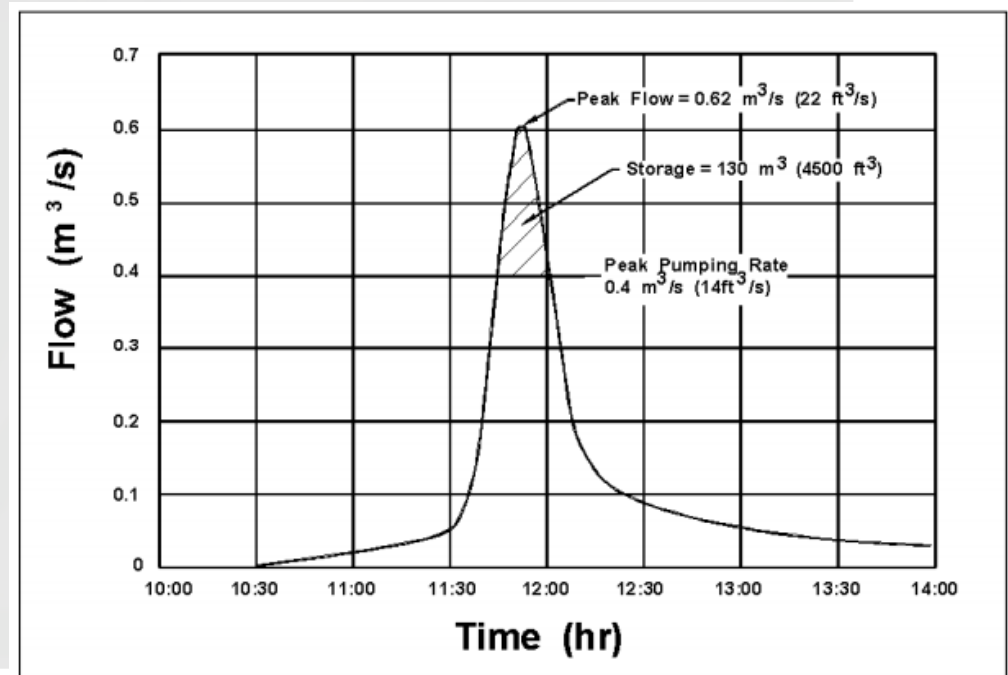
Main and Spare Pumps

- The designer should determine the number of pumps needed.
- Is recommended to use two to three pumps as minimum.
- Use equal-size pumps so they can be freely alternated into service.
- Considering short duration of high flows, the low frequency of the design storm, spare or standby pumps are typically not warranted in storm water applications.



Storage

- Is important to evaluate how much storage capacity should be provided in a pump station.
- The total storage can be determine comparing inflow hydrograph to the controlling pump discharge rate.
- The volume of water in the shaded area is beyond the capacity of the pumps and must be stored.





Storage cont.

- For cases of highway with either short underpasses or long depressions sections, is not reasonable to consider above ground storage.
- Water that originates outside the depressed areas should not be allowed to enter the depressed areas because of the need to pump all this water.
- In this situations the simplest form of storage is either the enlargement of the collection system or the construction of an underground storage facility.



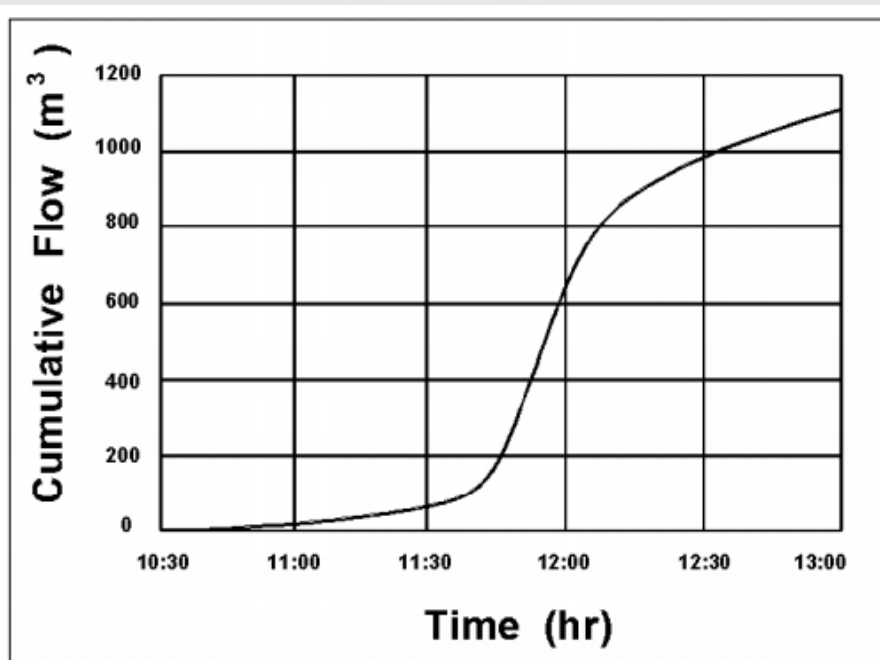
Pump Station Storage Requirement

- Storage capacity is required as part of pump station design to permit the use of smaller, more economical pumps.
- Increasing the storage will minimize the required pump size.
- The approach to evaluate the relationship between pump station storage and pumping rate requires development of an inflow mass curve and execution of a mass curve routing procedure.



Inflow Mass Curve

- The inflow mass curve is developed by:
 - Dividing the inflow hydrograph into uniform time increments.
 - Computing the inflow volume over each time step.
 - Summing the inflow volumes to obtain cumulative inflow volume and plotted against time to produce the curve.

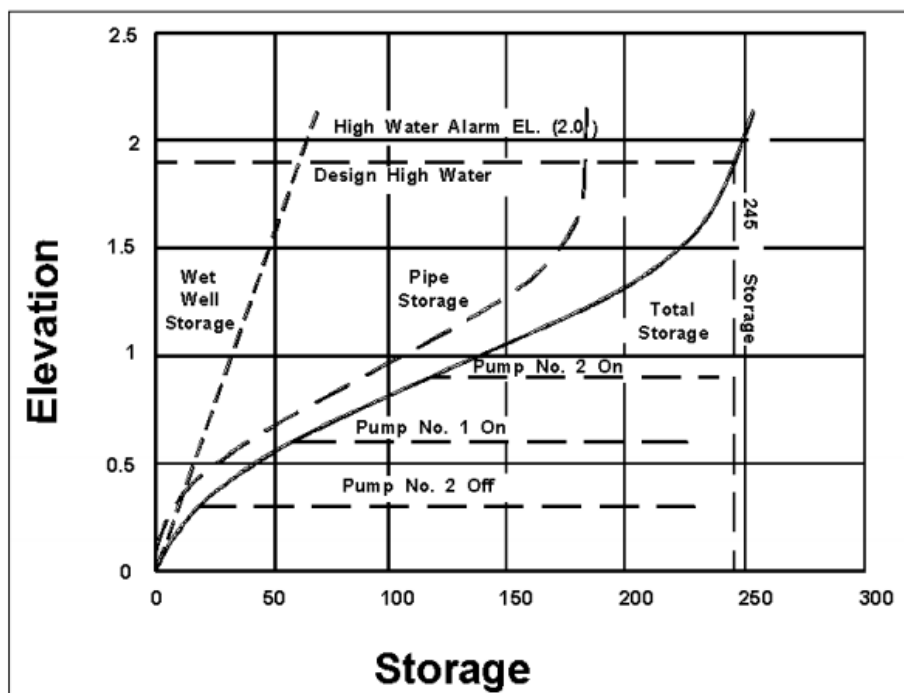




Mass Curve Routing

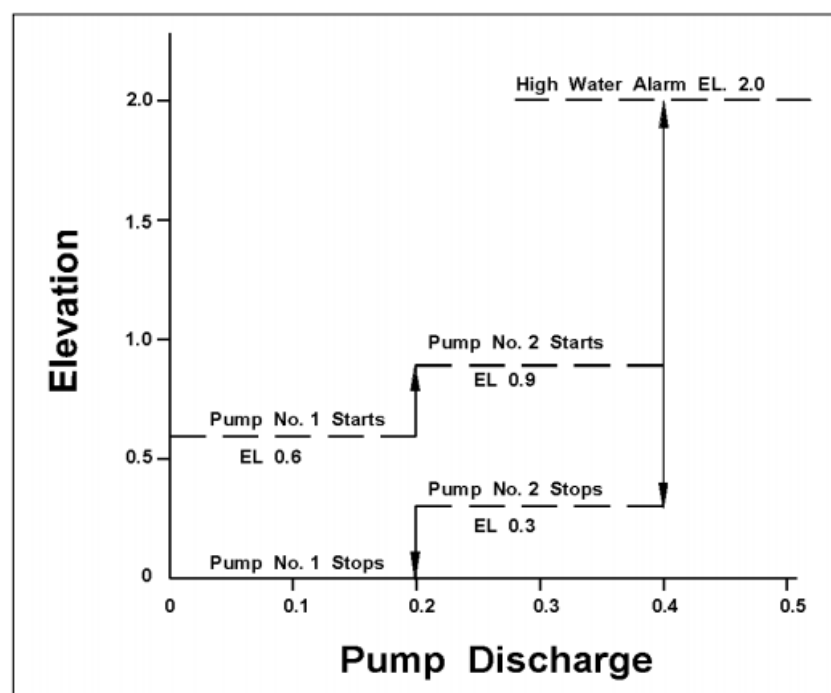
- Is important to the designer to understand how a typical pump station operates prior to start the mass curve routing.
- As storm water flow to the pump station, the water will be stored and the water level will rise to an elevation which activates the first pump.
- If the inflow is greater than the pump rate, the water level will continue to rise and activates the second pump.
- This process continues until the inflow rate subsides.
- To develop the mass curve routing, is necessary to have an inflow hydrograph, a stage-storage curve and a stage-discharge curve.

Mass Curve Routing cont.



Stage-storage curve

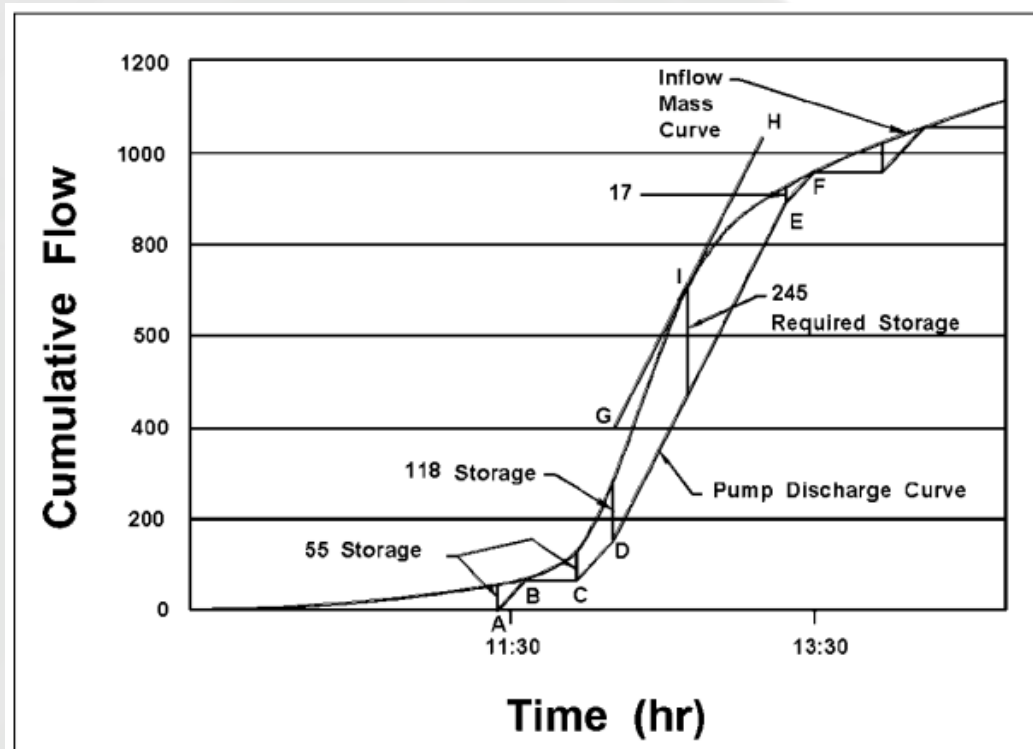
[1]



Stage-discharge curve

Mass Curve Routing cont.

- At point A, the first pump will start.
- At point B, storage has been emptied and pump turns off.
- At point C, volume started accumulated and first pump starts again.
- At point D, storage has filled to the elevation where second pump starts.
- At point E, second pump turns off.
- At point F, first pump turns off.



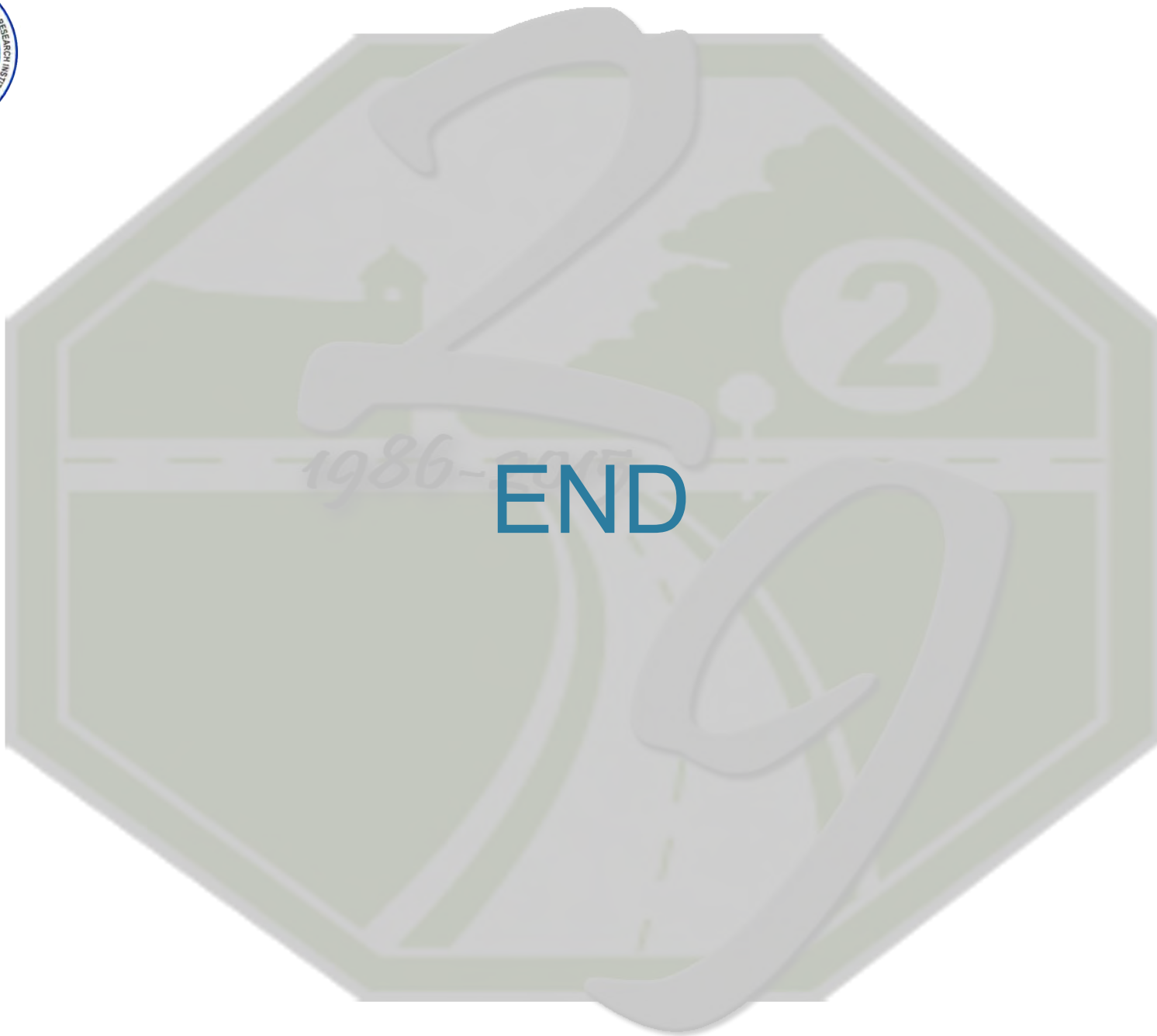
Mass curve routing diagram

[1]



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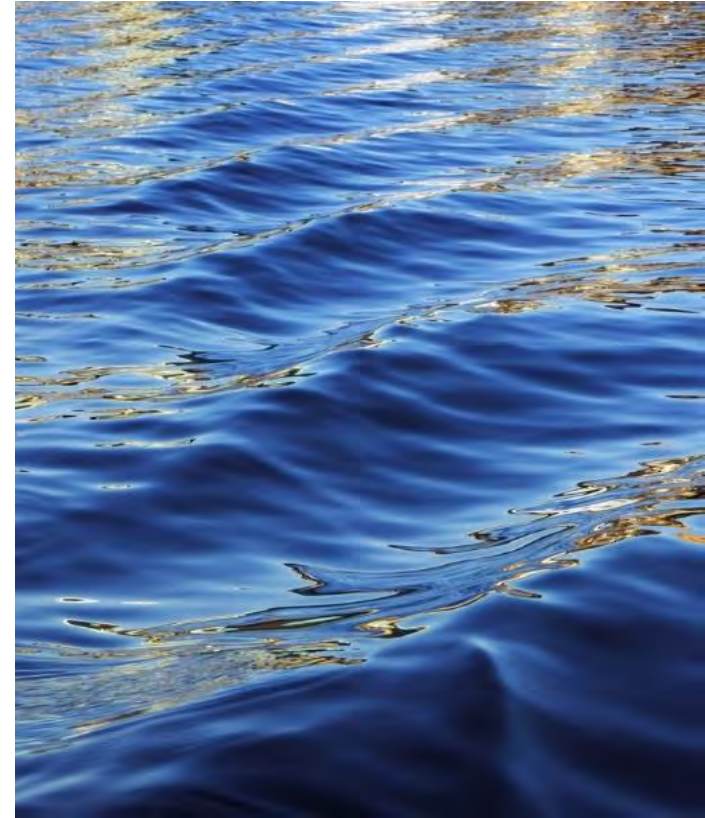
Module 9

Hydraulics: Erosion Control Practices and Measures



Erosion Control Practices and Measures

Part I
Classification and Construction Sequence



Erosion and sediment impacts

- Pollution
- Ecological impacts
- Hydraulic modifications
- Property damage
- Construction costs and schedule
- Lawsuits

Pollution



Ecological impacts



Hydraulic modifications



Property damage



Construction costs and schedule

- Stop work ordered for noncompliance
- Fines/ penalties for noncompliance
- Repair damage to adjacent properties
- Missed deadlines, litigation, other additional work.



Lawsuits



Classifications of E & SC Practices

- Primary vs Secondary
 - Primary: Keep soil in place and protect from erosive forces
 - Secondary or backup: Attempt to control the sediment
- Structural vs Non-Structural
 - Structural measures: Engineering practices adequate to prevent erosion or controlling runoff requiring design and installation
 - Non-Structural: Management alternatives to prevent soil erosion and sediment generation mostly through vegetative measures
- Type of Problem Area
 - Slopes
 - Receiving waters
 - Open drainage ways
 - Culverts and outfalls
 - Adjacent properties

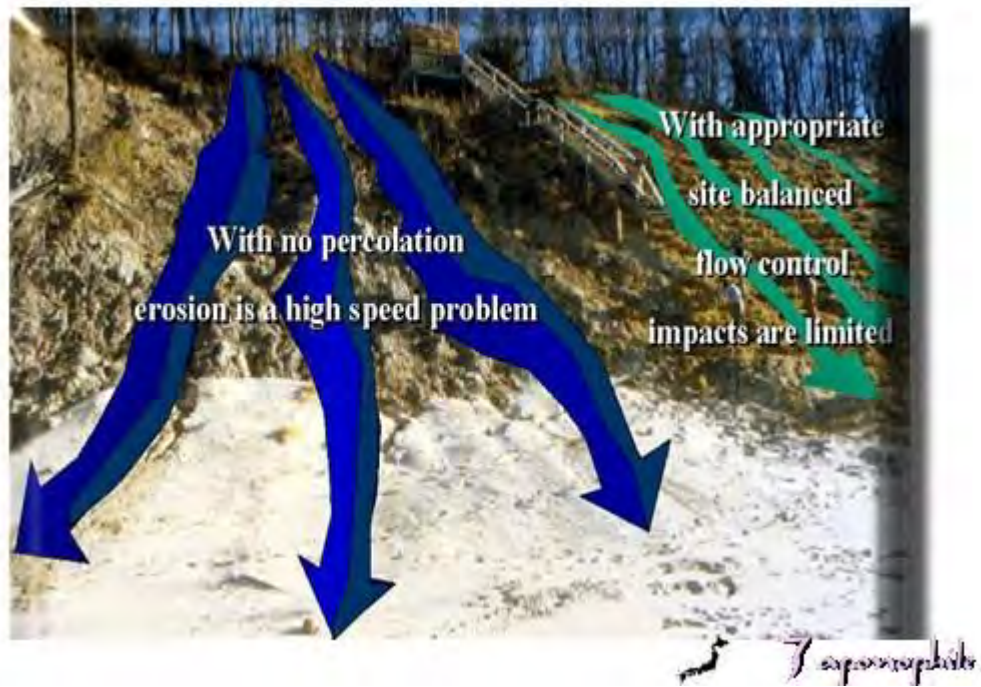
Erosion on Slopes

- slope length is long, the slope is steep, the soil is highly erodible, or that the soil cover (vegetation) has been removed and will take some time to be reestablished.
- Slope erosion can occur on cuts, fills, stockpiles, or cleared but ungraded surfaces.
- Measures include some combination of:
 - vegetative and structural protective covers (temporary seeding, permanent. seeding, groundcover, mulch, sodding, erosion control matting, and topsoiling)
 - water conveyance (temporary or permanent diversions and slope drains)
 - temporary construction road stabilization.



Erosion on Slopes

Slope Erosion



Receiving Waters

- Include streams, lakes and waterways
- protect from increased runoff quantities, and sediment loads from the construction site.
- Increase in runoff could lead to stream bank erosion and downstream flooding.
- Measures include combination of:
 - sediment barriers (straw or hay bale barriers, silt/geotextile fence, brush barrier)
 - water conveyance (outlet protection)
 - sediment detention ponds and basins
 - stream and stream bank protection



Sediment Barriers



water conveyance (outlet protection)



Sediment detention ponds and basins



Stream and stream bank protection



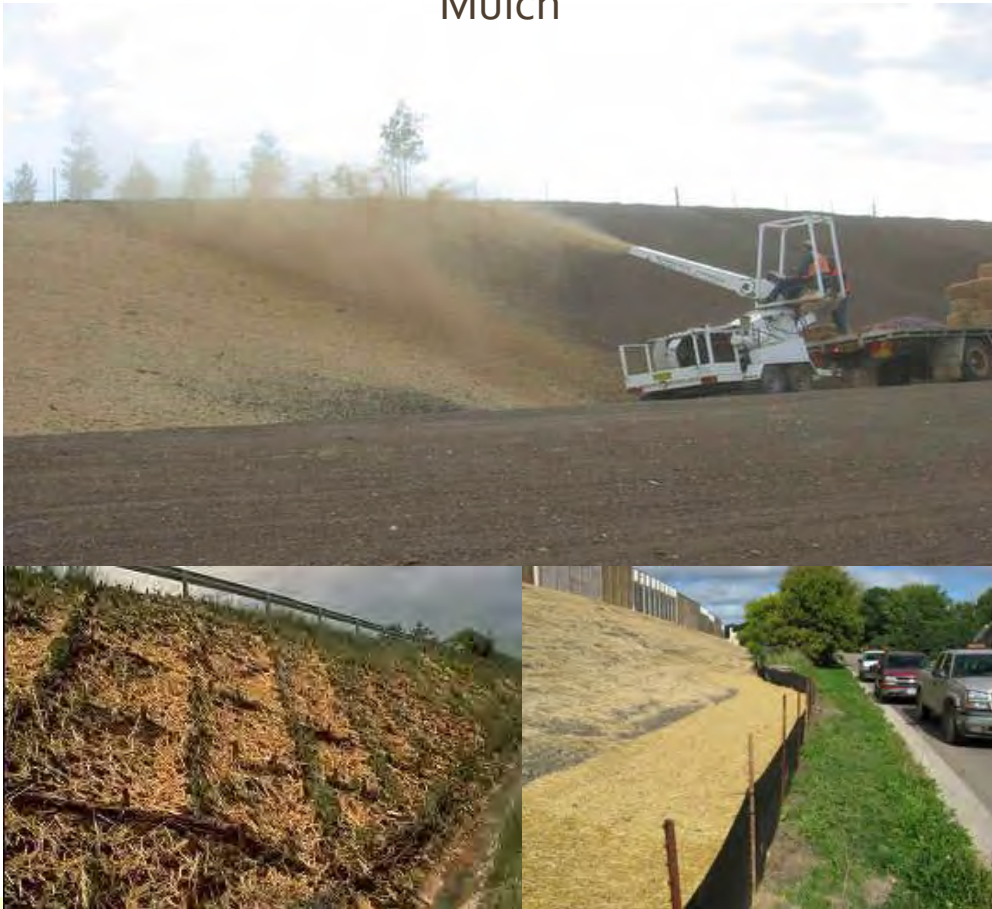
Open Drainage ways

- Open drainage ways can become significant sources of sediment
- Drainage ways need to be designed, constructed, and maintained.
- They should not be overtopped and do not transport the runoff at
- Velocities that will erode the bottom or sides of the drainage way.
- Measures include some combination of:
 - vegetative and structural protective cover (mulch, sodding, and erosion control matting)
 - water (conveyance check dams, inlet protection, and outlet protection)
 - stream and stream bank protection (riprap).

Open Drainage ways

vegetative and structural protective cover

Mulch



Sodding



Erosion control matting



Open Drainage ways

water (conveyance check dams, inlet protection, and outlet protection)

Conveyance check dams



Inlet protection



Outlet protection



Open Drainage ways

stream and stream bank protection (riprap).



Culverts and outfalls

- Culverts and outfalls need specific measures to ensure that their entrances are not blocked, that they do not become filled with sediment, and that their outlets do not erode down slope or downstream areas.
- Erosion control measures with application to culverts and outfalls include:
 - water conveyance (temporary slope drains, inlet protection, and outlet protection).



Adjacent properties

- Adjacent properties need to be protected from increased runoff and sediment load.
- Slopes within the construction site need to be protected from runoff from adjacent properties.
- Erosion control measures include:
 - sediment barriers
 - water conveyance (temporary and permanent diversions)
 - Sediment detention ponds and basins (temporary sediment traps and sediment basins).



Minimum recommendations for BMP's in construction sites

- Minimize the amount of existing vegetation that you must disturb for construction. Keep out of sensitive areas and their buffers.
- Before construction activities begin adjacent to sensitive areas
- Perimeter protection to filter sediment for sheet erosion shall be located downslope of all disturbed areas and properly installed prior to upslope grading.
- One hundred linear feet of silt fence per 0.25 acre and the necessary stakes to hold the fence in place shall be stockpiled onsite.
- Stabilized construction entrances will be installed as the first step of clearing and grading.

Minimum recommendations for BMP's in construction sites

- Stabilized construction entrances will be installed as the first step of clearing and grading.



Minimum recommendations for BMP's in construction sites

- Roads and parking areas will be stabilized immediately after the initial grading.
- Unsurfaced driveway entrances, access roads and parking areas used by construction traffic will be stabilized to minimize erosion and prevent tracking mud.
- The paved areas shall not be cleaned by washing/flushing streets.
- Dirty equipment, especially concrete trucks, **should not be cleaned in or near** waterways.
- Sediment retention facilities will be installed before grading.
- If sediment retention facilities need to be removed for grading, additional ponds/traps/systems to accommodate storage capacity need to be installed onsite. This will be **prior to removal of existing facility.**

Minimum recommendations for BMP's in construction sites

- Dust is to be controlled on construction sites.
- Water truck is used to control dust on dirty/grades areas only.
- Water truck will only drop enough water to control the dust. No runoff is to be generated.
- Erosion and sediment measures will be inspected a minimum of once a week and within 24 hours of significant storms (0.5 inches/24-hour, or where runoff is generated).
- A 24-hour phone number for the Erosion Sediment and Control Designated Inspector will be posted in clearly visible location on the project site.

Minimum recommendations for BMP's in construction sites

- Water truck is used to control dust on dirty/grades areas only



SUMMARY

Construction Activity	Schedule Consideration
1. Identify and label protection areas (e.g., riparian buffer zones, filter strips, trees).	Site delineation should be completed before construction begins.
2. Construction accesses. Construction entrance, construction routes, equipment parking areas (necessary perimeter controls).	First land-disturbing activity – Stabilize bare areas immediately with gravel and temporary vegetation as construction takes place.
3. Sediment traps and barriers. Sediment fences, straw bale barriers, and outlet protection.	Install principal basins after construction site is accessed. Install additional traps and barriers as needed during grading.
4. Runoff control. Diversions and outlet protection.	Install key practices after principal sediment traps and before land grading. Install additional runoff-control measures during grading.

SUMMARY

5. Runoff conveyance system. Stabilize stream banks, storm drains, channels, inlet and outlet protection, slope drains.	Where necessary, stabilize stream banks as early as possible. Install principal runoff conveyance system with runoff-control measures. Install remainder of system after grading.
6. Land clearing and grading. Site preparation - cutting, filling and grading, sediment basins, barriers, diversions, drains, surface roughening.	Begin major clearing and grading after principal sediment and key runoff-control measures are installed. Clear borrow and disposal areas only as needed. Install additional control measures as grading progresses. Mark trees and buffer areas for preservation.
7. Surface stabilization. Temporary and permanent seeding, mulching, sodding, riprap.	Apply temporary or permanent stabilization measures immediately on all disturbed areas where work is delayed or completed.
8. Building construction. Buildings, utilities, paving.	Install necessary erosion and sedimentation control practices as work takes place.
9. Landscaping and final stabilization. Topsoiling, trees and shrubs, permanent seeding, mulching, sodding, riprap.	Last construction phase-stabilize all open areas, including borrows and spoil areas. Remove and stabilize all temporary control measures.
10. Maintenance	Maintenance inspections should be performed weekly, and maintenance repair should be made immediately after period of rainfall.

Module 10

Channel Data Tables and Graphs

Channel Data

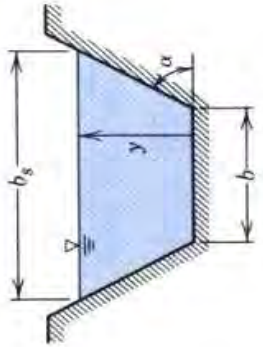
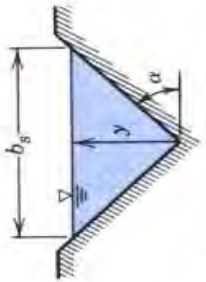
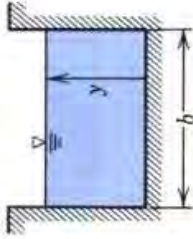
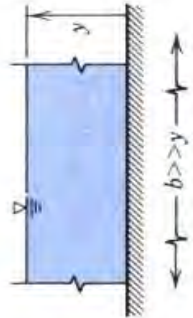
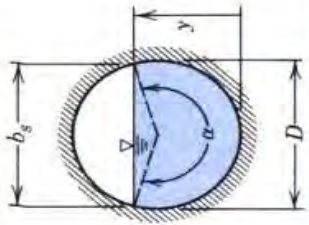
Tables and graphs

Open Channel Flow

Design Tables

CHARACTERISTICS OF OPEN CHANNELS.

Geometric Properties of Common Open-Channel Shapes

Shape	Section	Flow Area, A	Wetted Perimeter, P	Hydraulic Radius, R_h
Trapezoidal		$y(b + y \cot \alpha)$	$b + \frac{2y}{\sin \alpha}$	$\frac{y(b + y \cot \alpha)}{b + \frac{2y}{\sin \alpha}}$
Triangular		$y^2 \cot \alpha$	$\frac{2y}{\sin \alpha}$	$\frac{y \cos \alpha}{2}$
Rectangular		by	$b + 2y$	$\frac{by}{b + 2y}$
Wide Flat		by	b	y
Circular		$(\alpha - \sin \alpha) \frac{D^2}{8}$	$\frac{\alpha D}{2}$	$\frac{D}{4} \left(1 - \frac{\sin \alpha}{\alpha} \right)$

VALUES of DISCHARGE FACTOR K in $Q = (K/n)y^{5/3}S^{1/2}$ for TRAPEZOIDAL CHANNELS

(y = depth of flow, b = bottom width of channel)

Side Slopes of Channel Section (horizontal to vertical)

y/b	Vertical	$\frac{1}{4}:1$	$\frac{1}{2}:1$	$\frac{2}{3}:1$	1:1	$1\frac{1}{2}:1$	2:1	$2\frac{1}{2}:1$	3:1	4:1
.01	146.7	147.2	147.6	148.0	148.3	148.8	149.2	149.5	149.9	150.5
.02	72.4	72.9	73.4	73.7	74.0	74.5	74.9	75.3	75.6	76.3
.03	47.6	48.2	48.6	49.0	49.3	49.8	50.2	50.6	50.9	51.6
.04	35.3	35.8	36.3	36.6	36.9	37.4	37.8	38.2	38.6	39.3
.05	27.9	28.4	28.9	29.2	29.5	30.0	30.5	30.9	31.2	32.0
.06	23.0	23.5	23.9	24.3	24.6	25.1	25.5	26.0	26.3	27.1
.07	19.5	20.0	20.4	20.8	21.1	21.6	22.0	22.4	22.8	23.6
.08	16.8	17.3	17.8	18.1	18.4	18.9	19.4	19.8	20.2	21.0
.09	14.8	15.3	15.7	16.1	16.4	16.9	17.4	17.8	18.2	19.0
.10	13.2	13.7	14.1	14.4	14.8	15.3	15.7	16.2	16.6	17.4
.11	11.83	12.33	12.76	13.11	13.42	13.9	14.4	14.9	15.3	16.1
.12	10.73	11.23	11.65	12.00	12.31	12.8	13.3	13.8	14.2	15.0
.13	9.80	10.29	10.71	11.06	11.37	11.9	12.4	12.8	13.3	14.1
.14	9.00	9.49	9.91	10.26	10.57	11.1	11.6	12.0	12.5	13.4
.15	8.32	8.80	9.22	9.57	9.88	10.4	10.9	11.4	11.8	12.7
.16	7.72	8.20	8.61	8.96	9.27	9.81	10.29	10.75	11.20	12.1
.17	7.19	7.67	8.08	8.43	8.74	9.28	9.77	10.23	10.68	11.6
.18	6.73	7.20	7.61	7.96	8.27	8.81	9.30	9.76	10.21	11.1
.19	6.31	6.78	7.19	7.54	7.85	8.39	8.88	9.34	9.80	10.7
.20	5.94	6.40	6.81	7.16	7.47	8.01	8.50	8.97	9.43	10.3
.22	5.30	5.76	6.16	6.51	6.82	7.36	7.86	8.33	8.79	9.70
.24	4.77	5.22	5.62	5.96	6.27	6.82	7.32	7.79	8.26	9.18
.26	4.32	4.77	5.16	5.51	5.82	6.37	6.87	7.35	7.81	8.74
.28	3.95	4.38	4.77	5.12	5.43	5.98	6.48	6.96	7.43	8.36
.30	3.62	4.05	4.44	4.78	5.09	5.64	6.15	6.63	7.10	8.04
.32	3.34	3.77	4.15	4.49	4.80	5.35	5.86	6.34	6.82	7.75
.34	3.09	3.51	3.89	4.23	4.54	5.10	5.60	6.09	6.56	7.50
.36	2.88	3.29	3.67	4.01	4.31	4.87	5.38	5.86	6.34	7.28
.38	2.68	3.09	3.47	3.81	4.11	4.67	5.17	5.66	6.14	7.09
.40	2.51	2.92	3.29	3.62	3.93	4.48	4.99	5.48	5.96	6.91
.42	2.36	2.76	3.13	3.46	3.77	4.32	4.83	5.32	5.80	6.75
.44	2.22	2.61	2.98	3.31	3.62	4.17	4.68	5.17	5.66	6.60
.46	2.09	2.48	2.85	3.18	3.48	4.04	4.55	5.04	5.52	6.47
.48	1.98	2.36	2.72	3.06	3.36	3.91	4.43	4.92	5.40	6.35
.50	1.87	2.26	2.61	2.94	3.25	3.80	4.31	4.81	5.29	6.24
.55	1.65	2.02	2.37	2.70	3.00	3.55	4.07	4.56	5.05	6.00
.60	1.46	1.83	2.17	2.50	2.80	3.35	3.86	4.36	4.84	5.80
.70	1.18	1.53	1.87	2.19	2.48	3.03	3.55	4.04	4.53	5.49
.80	.982	1.31	1.64	1.95	2.25	2.80	3.31	3.81	4.30	5.26
.90	.831	1.15	1.47	1.78	2.07	2.62	3.13	3.63	4.12	5.08
1.00	.714	1.02	1.33	1.64	1.93	2.47	2.99	3.48	3.97	4.93
1.20	.548	.836	1.14	1.43	1.72	2.26	2.77	3.27	3.76	4.72
1.40	.436	.708	.998	1.29	1.57	2.11	2.62	3.12	3.60	4.57
1.60	.357	.616	.897	1.18	1.46	2.00	2.51	3.00	3.49	4.45
1.80	.298	.546	.820	1.10	1.38	1.91	2.42	2.91	3.40	4.36
2.00	.254	.491	.760	1.04	1.31	1.84	2.35	2.84	3.33	4.29
2.25	.212	.439	.700	.973	1.24	1.77	2.28	2.77	3.26	4.22

•Values from King's "Handbook of Hydraulics", 4th edition, McGraw-Hill Co.

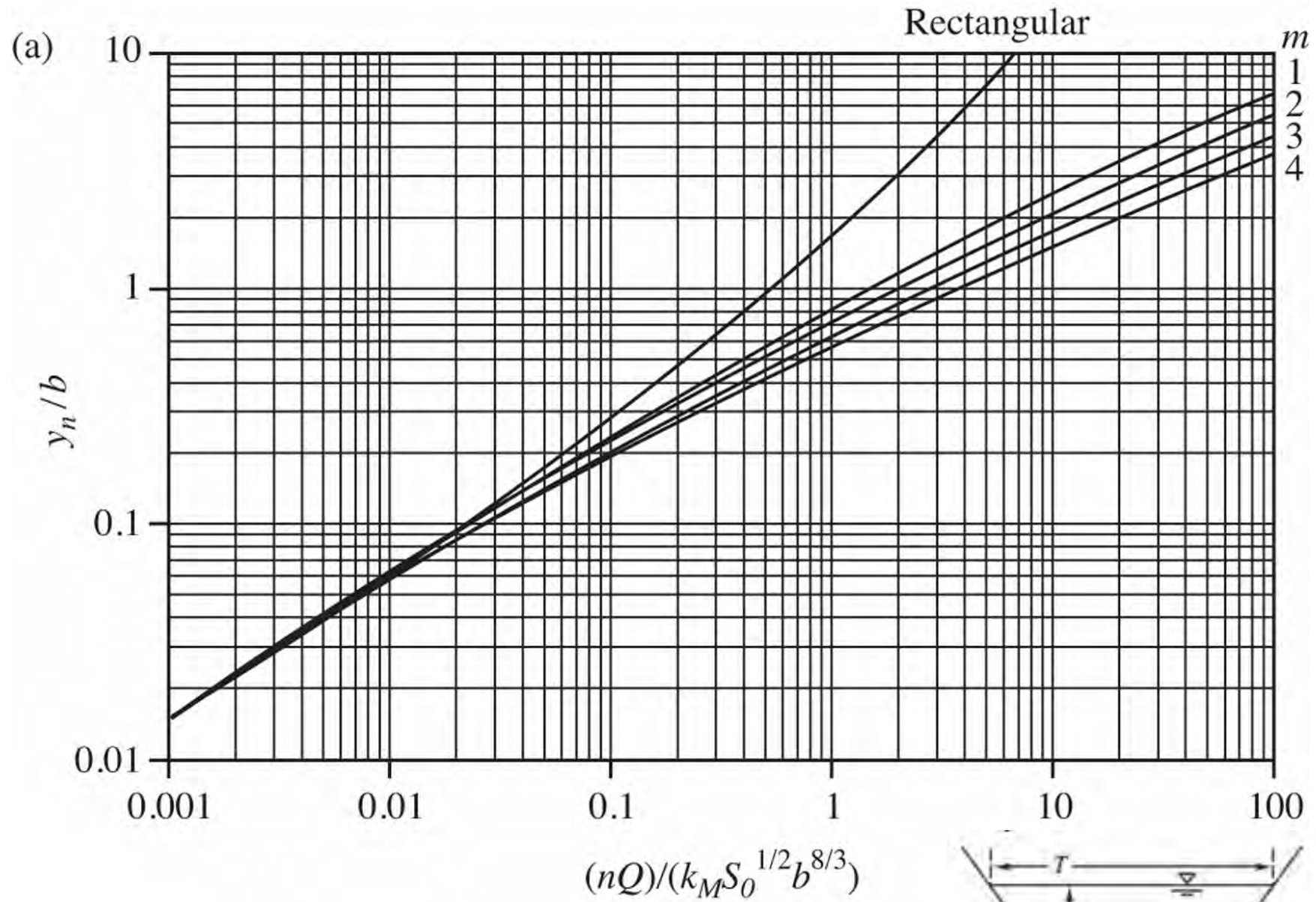
**VALUES of DISCHARGE FACTOR K' in $Q = (K'/n)b^{5/3}S^{1/2}$
for TRAPEZOIDAL CHANNELS**

(y = depth of flow, b = bottom width of channel)

Side Slopes of Channel Section (horizontal to vertical)

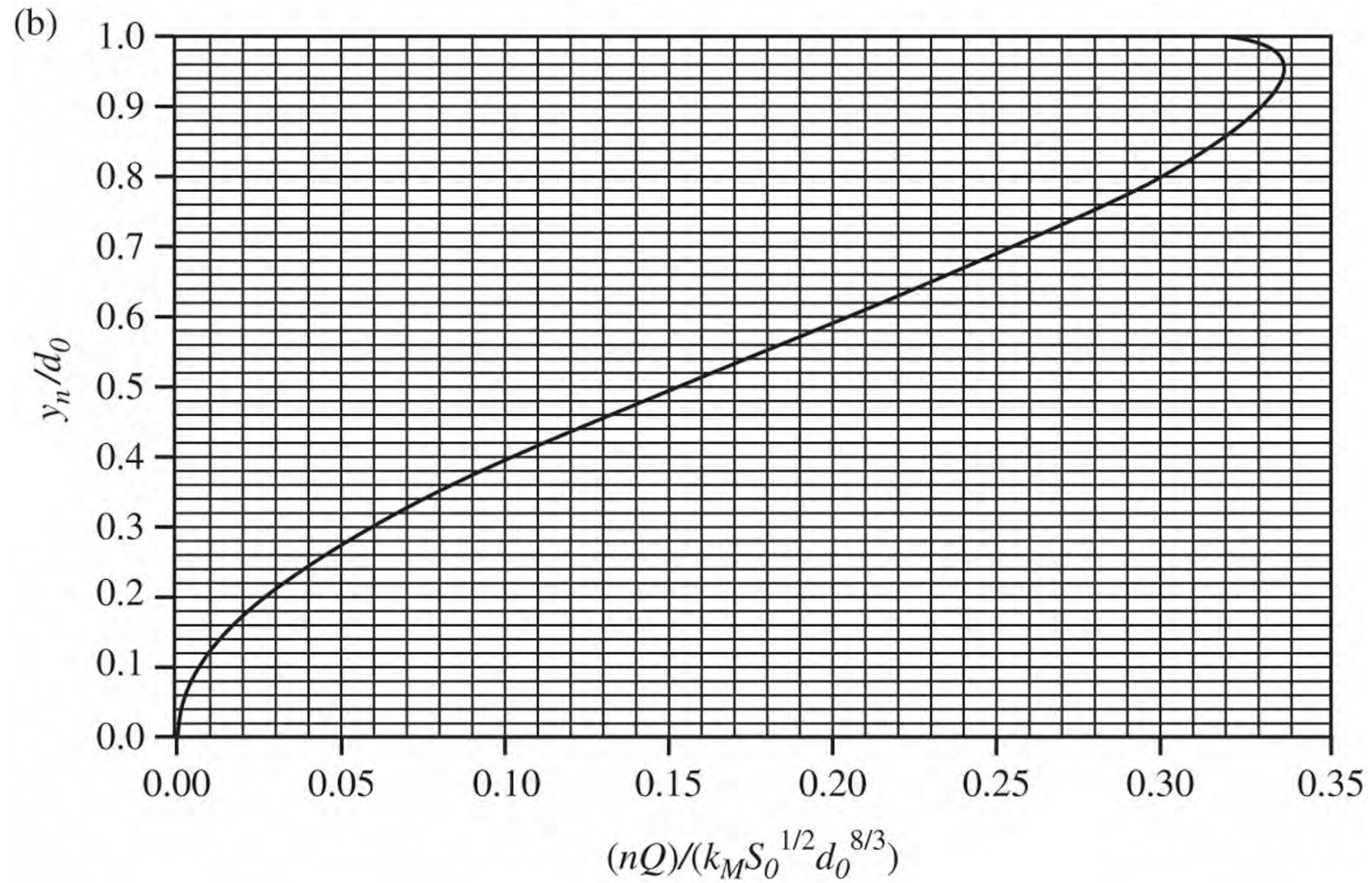
y/b	Verti- cal	1 : 1	$\frac{1}{2}$: 1	$\frac{2}{3}$: 1	1 : 1	1 $\frac{1}{2}$: 1	2 : 1	2 $\frac{1}{2}$: 1	3 : 1	4 : 1
.01	.00068	.00068	.00069	.00069	.00069	.00069	.00069	.00069	.00070	.00070
.02	.00213	.00215	.00216	.00217	.00218	.00220	.00221	.00222	.00223	.00225
.03	.00414	.00419	.00423	.00426	.00428	.00433	.00436	.00439	.00443	.00449
.04	.00660	.00670	.00679	.00685	.00691	.00700	.00708	.00716	.00723	.00736
.05	.00946	.00964	.00979	.00991	.01002	.01019	.01033	.01047	.01060	.01086
.06	.0127	.0130	.0132	.0134	.0136	.0138	.0141	.0143	.0145	.0150
.07	.0162	.0166	.0170	.0173	.0175	.0180	.0183	.0187	.0190	.0197
.08	.0200	.0206	.0211	.0215	.0219	.0225	.0231	.0236	.0240	.0250
.09	.0241	.0249	.0256	.0262	.0267	.0275	.0282	.0289	.0296	.0310
.10	.0284	.0294	.0304	.0311	.0318	.0329	.0339	.0348	.0358	.0376
.11	.0329	.0343	.0354	.0364	.0373	.0387	.0400	.0413	.0424	.0448
.12	.0376	.0393	.0408	.0420	.0431	.0450	.0466	.0482	.0497	.0527
.13	.0425	.0446	.0464	.0480	.0493	.0516	.0537	.0556	.0575	.0613
.14	.0476	.0502	.0524	.0542	.0559	.0587	.0612	.0636	.0659	.0706
.15	.0528	.0559	.0585	.0608	.0627	.0662	.0692	.0721	.0749	.0806
.16	.0582	.0619	.0650	.0676	.0700	.0740	.0777	.0811	.0845	.0912
.17	.0638	.0680	.0716	.0748	.0775	.0823	.0866	.0907	.0947	.1026
.18	.0695	.0744	.0786	.0822	.0854	.0910	.0960	.1008	.1055	.1148
.19	.0753	.0809	.0857	.0899	.0936	.1001	.1059	.1115	.1169	.1277
.20	.0812	.0876	.0931	.0979	.1021	.1096	.1163	.1227	.1290	.1414
.22	.0934	.1015	.109	.115	.120	.130	.139	.147	.155	.171
.24	.1061	.1161	.125	.133	.140	.152	.163	.173	.184	.204
.26	.119	.131	.142	.152	.160	.175	.189	.202	.215	.241
.28	.132	.147	.160	.172	.182	.201	.217	.234	.249	.281
.30	.146	.163	.179	.193	.205	.228	.248	.267	.287	.324
.32	.160	.180	.199	.215	.230	.256	.281	.304	.327	.371
.34	.174	.198	.219	.238	.256	.287	.316	.343	.370	.423
.36	.189	.216	.241	.263	.283	.319	.353	.385	.416	.478
.38	.203	.234	.263	.288	.312	.353	.392	.429	.465	.537
.40	.218	.253	.286	.315	.341	.389	.434	.476	.518	.600
.42	.233	.273	.309	.342	.373	.427	.478	.526	.574	.668
.44	.248	.293	.334	.371	.405	.467	.525	.580	.633	.740
.46	.264	.313	.359	.401	.439	.509	.574	.636	.696	.816
.48	.279	.334	.385	.432	.474	.553	.625	.695	.763	.897
.50	.295	.355	.412	.463	.511	.598	.679	.757	.833	.983
.55	.335	.410	.482	.548	.609	.722	.826	.926	1.025	1.22
.60	.375	.468	.557	.640	.717	.858	.990	1.117	1.24	1.49
.70	.457	.592	.722	.844	.959	1.17	1.37	1.56	1.75	2.12
.80	.542	.725	.906	1.078	1.24	1.54	1.83	2.10	2.37	2.90
.90	.628	.869	1.11	1.34	1.56	1.98	2.36	2.74	3.11	3.83
1.00	.714	1.022	1.33	1.64	1.93	2.47	2.99	3.48	3.97	4.93
1.20	.891	1.36	1.85	2.33	2.79	3.67	4.51	5.32	6.11	7.67
1.40	1.07	1.74	2.45	3.16	3.85	5.17	6.42	7.64	8.84	11.2
1.60	1.25	2.16	3.14	4.14	5.12	6.99	8.78	10.52	12.2	15.6
1.80	1.43	2.62	3.93	5.28	6.60	9.15	11.6	14.0	16.3	20.9
2.00	1.61	3.12	4.82	6.58	8.32	11.7	14.9	18.1	21.2	27.3
2.25	1.84	3.81	6.09	8.46	10.8	15.4	19.8	24.1	28.4	36.7

*Values from King's "Handbook of Hydraulics", 4th edition, McGraw-Hill Co.



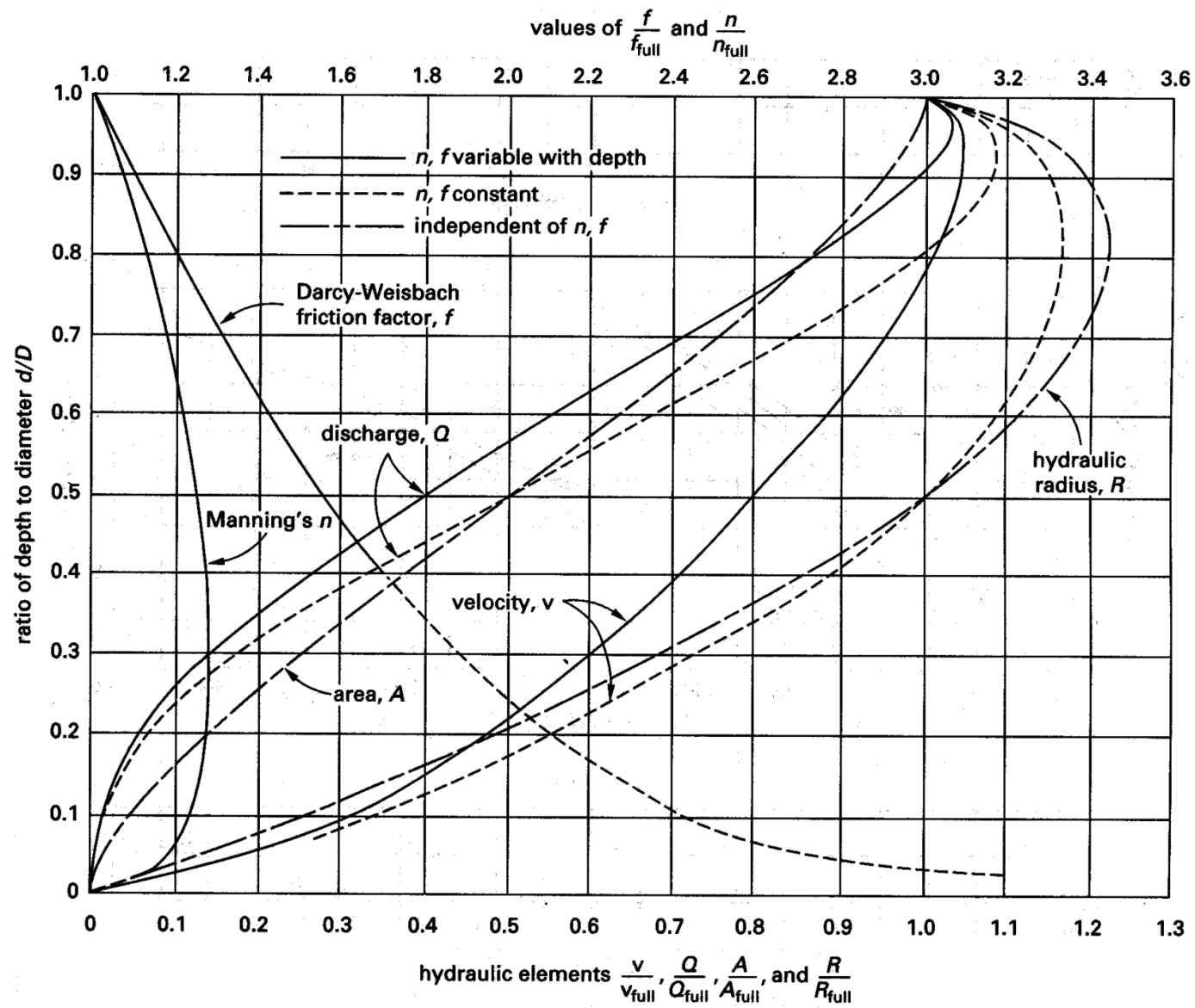
Normal depth solution procedure: (a) trapezoidal channels (m side slope)

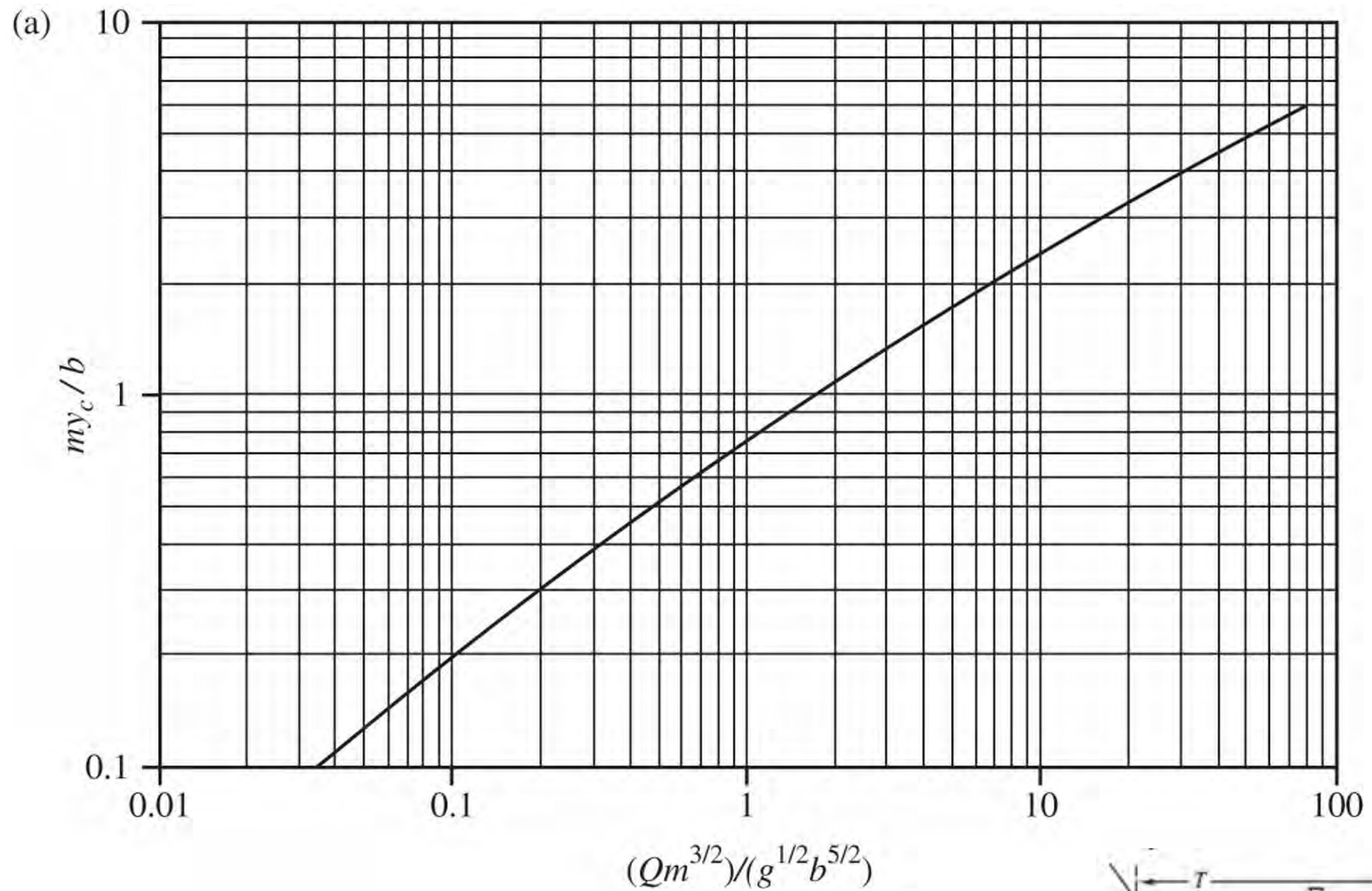
$k_M = 1.0$ for SI $k_M = 1.49$ for BS



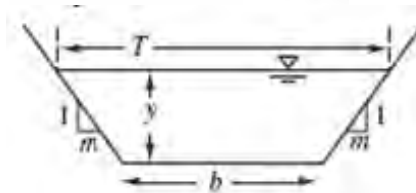
Normal depth solution procedure: (b) circular channels (d_0 diameter)

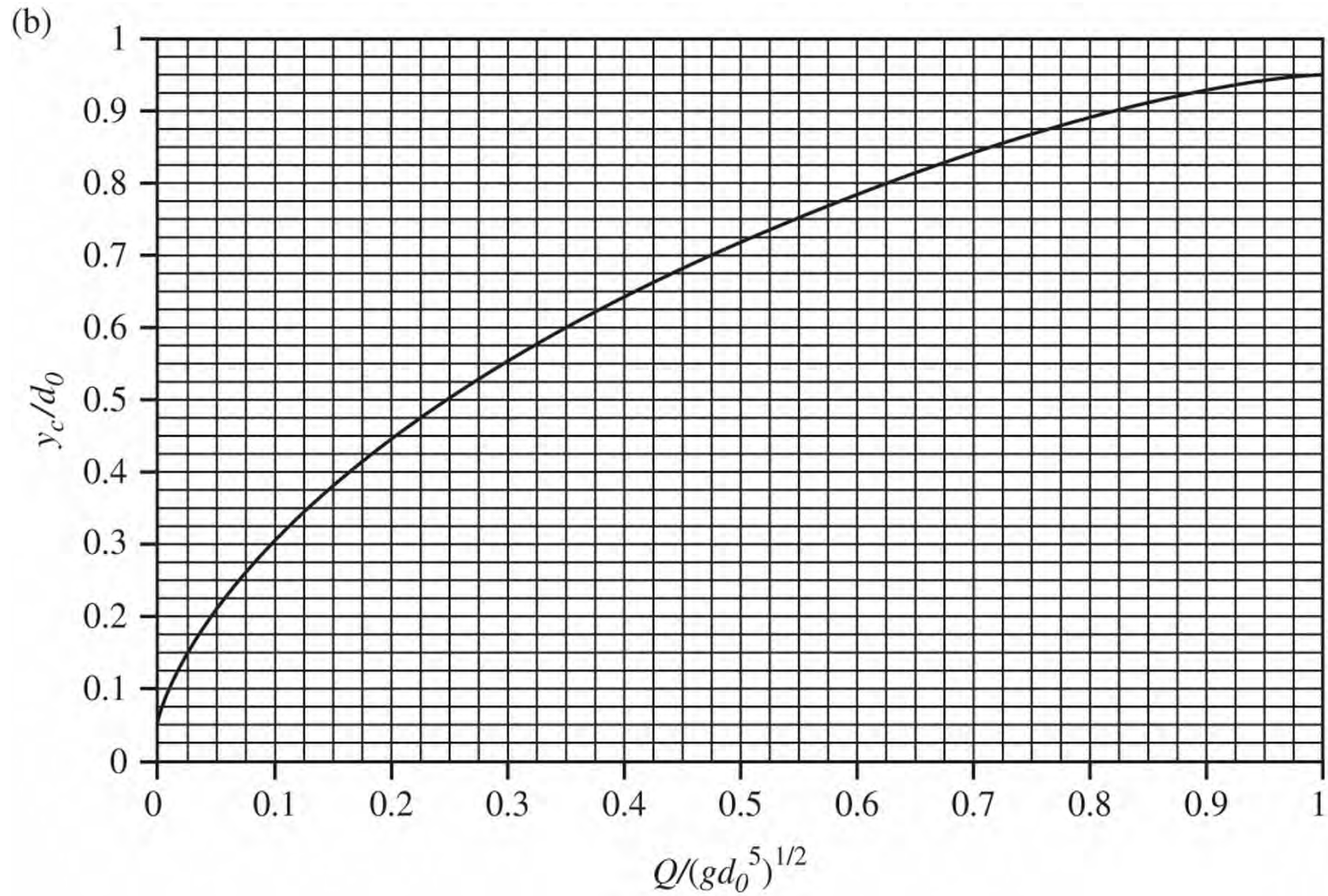
$k_M = 1.0$ for SI
 $k_M = 1.49$ for BS





Critical depth solution procedure: trapezoidal channels





Critical depth solution procedure: circular channels

Module 11

Hydraulic Machines

Problems

HYDRAULIC MACHINES

Problems

HYDRAULIC MACHINES

1) A centrifugal pump has a power of 20 hp and pumpst at the rate of 400 gpm producing a total dynamic head of 75 psi. What is the overall efficiency of the pump?

Answer:

- bhp = Pump horsepower
- γ = specific weight
- Q = pump discharge
- H = total dynamic head
- e = pump efficiency

$$bhp = \frac{\gamma Q H}{550 e}$$

$$Q = 400 \left(\frac{1}{448.8} \right) = 0.8913 \text{ ft}^3/\text{s}$$

$$H = 75 \text{ psi} (2.31 \text{ ft/psi}) = 173.25 \text{ ft}$$

Solving for e:

$$20 \text{ hp} = \frac{62.4 \times 0.8913 \times 173.25}{550 e}$$

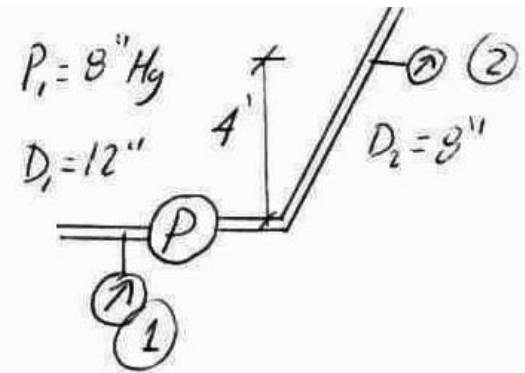
$$e = 0.88$$

HYDRAULIC MACHINES

2) A discharge of 2000 gal/min of thickened sludge with a specific gravity of 1.2 and a temperature of 60° F is pumped through a pump with an inlet diameter of 12 in and an outlet diameter of 8 in. The centerlines of the inlet and the outlet are at the same elevation. The inlet pressure is 8 in of mercury (vacuum). A discharge pressure gauge located 4 ft above the pump discharge centerline reads 20 psig. The pump efficiency is 85%. All the pipes are schedule 40. How much is the pump input power ?

SOLUTION

Draw a Sketch:



Definition of power: $Power = \frac{\gamma Q h_p}{\eta}$

Apply Energy Equation between point 1 and 2:

$$h_p + \frac{P_1}{\gamma} + \frac{V_1^2}{2g} + z_1 = \frac{P_2}{\gamma} + \frac{V_2^2}{2g} + z_2 + h_{f,1-2} + h_{m,1-2} \quad \text{Assume } h_m \approx 0, h_{f,1-2} = 0 \quad \text{Solve for } h_p$$

Conversion of units: $Q = 2000 \text{ gal/min} \times 0.002228 = 4.456 \text{ ft}^3/\text{s}$

Obtain velocities:

$$V_1 = \frac{Q}{A_1} = \frac{4Q}{\pi D_1^2} = \frac{4 \times 4.456}{\pi \times 12^2} = 5.673 \text{ ft/s}$$

$$V_2 = V_1 \left(\frac{D_1}{D_2} \right)^2 = 5.673 \times \left(\frac{1}{0.667} \right)^2 = 12.764 \text{ ft/s}$$

HYDRAULIC MACHINES

SOLUTION

Convert manometer readings to pressure:

$$P_{vac} = \left(\frac{9}{12}\right) \times 13.6 \times 62.4 = 565.34 \text{ lb/ft}^2 \text{ VACUUM}$$

\uparrow SG_{Hg \uparrow γ_{WATER}}

Convert vacuum pressure to absolute pressure:

$$P_{vac} = P_{atm} - P_{vac} = 2116.8 - 565.34 = 1551 \text{ lb/ft}^2 \text{ abs}$$

Convert absolute pressure to pressure head:

$$P_1/\gamma = \frac{1551}{1.2 \times 62.4} = 20.71 \text{ ft (absolute)}$$

Repeat the same for the outlet pressure:

$$P_2 = 20 \text{ psig} = 20 \times 144 + 2116.8 = 4996.8 \text{ lb/ft}^2 \text{ abs.}$$

$$P_2/\gamma = 66.73 \text{ ft (absolute)}$$

Substitute pressures and velocities in the energy equation and solve for h_p :

$$h_p = (z_2 - z_1) + \left(\frac{P_2}{\gamma} + \frac{P_1}{\gamma}\right) + \left(\frac{V_2^2}{2g} - \frac{V_1^2}{2g}\right)$$

$$h_p = 4 + (66.73 - 20.71) + \left(\frac{(12.76)^2 - (5.67)^2}{2 \times 32.2}\right)$$

$$h_p = 4 + 46.02 + 2.029 = 52.05 \text{ ft}$$

Convert to power:

$$Power = \frac{\gamma Q h_p}{\eta} = \frac{(1.2 \times 62.4) \times 4.456 \times 52.05}{0.85}$$

$$P = 20431.7 \text{ ft-lb/s} = 37.15 \text{ hp}$$

- (a) 26hp
- (b) 31 hp
- (c) 37 hp
- (d) 53 hp

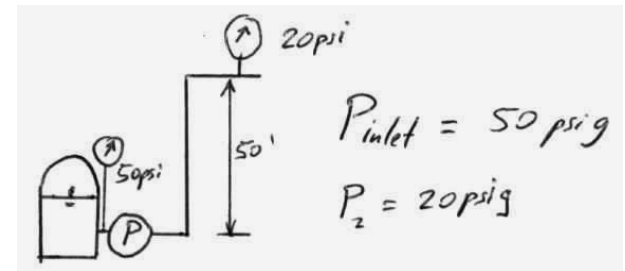
HYDRAULIC MACHINES

3) A discharge of 1.25 ft³/s (35 l/s) of water at 70°F are pumped from the bottom of a tank through 700 ft (230 m) of steel pipe schedule 40 with a nominal diameter of 4 in (10.2 cm). The pipeline includes 50 ft (15 m) of rise elevation, two right angle elbows and one wide-open gate valve. The working pressure at the end of the line must be close to 20 psig (140 kPa). How much is the hydraulic power required for this system?

Solution

- Draw a sketch:
- Estimate minor losses:

<u>Minor Losses</u> (Equiv. Length)		
		L_{eq}
2	90° Elbow	13' x 2 = 26'
1	Open Gate V.	2.5'
1	Swing Check	38 ft
		$\sum L_{eq} = 66.5 \text{ ft}$



$$L_T = L + \sum L_{eq} = 700 + 66.5 = 766.5$$

- Write energy equation between inlet and outlet:

$$h_p + z_1 + \frac{P_1}{\gamma} + \frac{V_1^2}{2g} = z_2 + \frac{P_2}{\gamma} + \frac{V_2^2}{2g} + h_f + h_m$$

HYDRAULIC MACHINES

- Compute Reynolds number and relative roughness:

$$Re = \frac{VD}{\nu} = \frac{4Q}{\pi D \nu} = \frac{4 \times 1.25}{\pi (4/12) 1.059 \times 10^{-5}} = 4.51 \times 10^5$$

Roughness: For steel: $\epsilon = 0.0002 \text{ ft}$

$$\text{Relative Roughness: } \epsilon/D = \frac{0.0002}{0.33} = 0.0006$$

- Compute flow velocity: $V = Q/A = \frac{1.25 \times 4}{\pi (0.33)^2} = 14.61 \text{ ft/s}$
- Go to Moody diagram to get friction factor:

From Moody diagram or Jain's equation: $f = 0.0183$

- Compute total losses: $h_f + h_m = f \frac{L_T V^2}{2gD} = h_T$

$$h_T = \frac{(0.01835) (766.5) (14.61)^2}{2 \times 32.2 \times 0.33}$$

$$h_T = 141.26 \text{ ft}$$

HYDRAULIC MACHINES

- Get elevation, pressure head and velocity head differences:

$$z_2 - z_1 = 50'$$

$$P_{2/f} - P_{1/f} = \frac{20 \times 144}{62.4} - \frac{50 \times 144}{62.4} = -69.23 \text{ ft}$$

$$\frac{V_2^2 - V_1^2}{2g} = \frac{V_2^2}{2g} \quad (V_1 \approx 0) \text{ TANK!}$$

$$= \frac{(14.61)^2}{2 \times 32.2} = 3.31 \text{ ft.}$$

- Substitute in the energy equation and solve for pump head:

$$h_p = (z_2 - z_1) + \left(\frac{P_2 - P_1}{\gamma} \right) + \left(\frac{V_2^2 - V_1^2}{2g} \right) + h_f$$

$$h_p = 50 - 69.23 + 3.31 + 141.26 = 125.35 \text{ ft}$$

- Convert pump head in pump power: $P = \frac{80 h_p}{550} = 17.77 h_p$

NOTE: If you use exact diameter for steel (sch-40)

$$D = 4.5 - 2 \times 0.237 = 4.026'' = 0.3355'$$

$$A = 0.0884 \Rightarrow V_1 = 14.139 \text{ ft/s.}$$

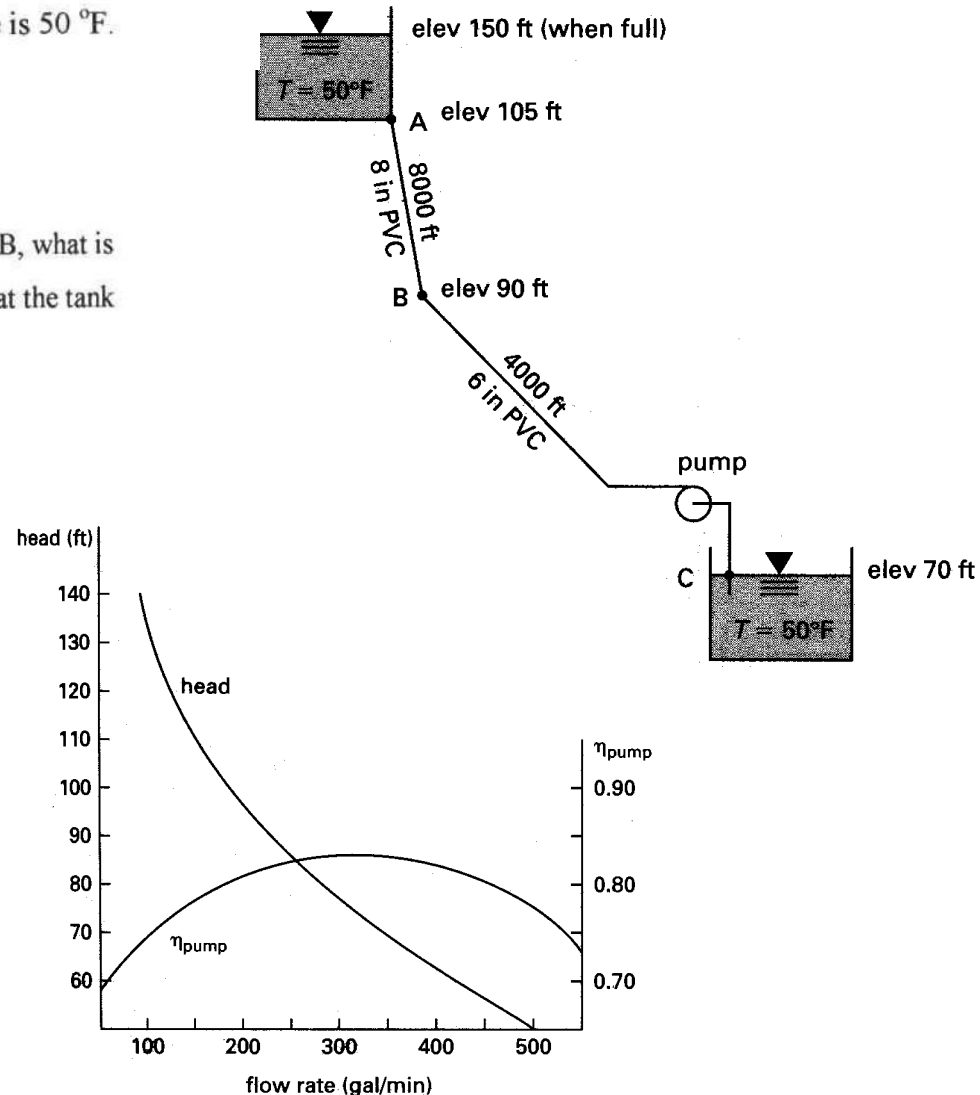
This changes h_p to 113.9 ft and $P = 16.13 h_p$

HYDRAULIC MACHINES

6)

A tank supplies water to a small town during the day. At night, when the demand is low, a pump is used to refill the tank. New PVC pipe connects the pump and tank. All minor losses are insignificant. The water temperature is 50 °F.

- 1) What is the minimum rate at which the tank will be refilled.
- 2) If a minimum pressure of 20 psi is needed at a hydrant located at point B, what is the maximum flow rate that can be drawn from the hydrant? Assume that the tank is full and the pump is not in use during fire-fighting.



HYDRAULIC MACHINES

a) Minimum rate at which tank is refilled:

The minimum rate or discharge occurs when the tank is full because that implies increased pump head and lower discharge (review pump curves)

Energy Equation between both : $h_p = (z_2 - z_1) + h_{f_{A-B}} + h_{f_{B-C}}$

- Assume value for the friction factors: $f_{AB} = f_{BC} = 0.02$

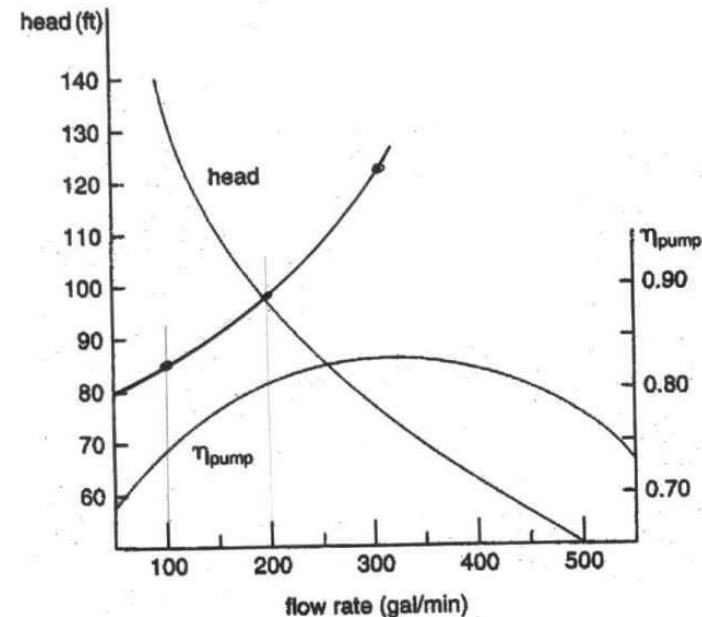
$$\Rightarrow h_p = (150 - 70) + \frac{f}{2g} \left[\frac{V_{AB}^2 L_{AB}}{D_{AB}} + \frac{V_{BC}^2 L_{BC}}{D_{BC}} \right]$$

$$h_p = 80 + \frac{0.02 \times 16}{2 \times 32.2} \left[\frac{8000 Q^2}{\pi^2 (8/12)^5} + \frac{4000 Q^2}{(\frac{6}{12})^5 \pi^2} \right]$$

$$h_p = 80 + 94.86 Q^2$$

The operating point is at $Q = 190 \text{ gal/min}$, $h_A = 97 \text{ ft}$.

Q_{gpm}	$Q \text{ ft}^3/\text{s}$	$h_p \text{ ft}$
100	0.228	84.71
200	0.446	98.83
300	0.669	122.38



HYDRAULIC MACHINES

- The friction factors must be checked. The velocities and Reynolds numbers are:

$$v_{AB} = \frac{Q}{\pi \frac{D_{AB}^2}{4}} = \frac{0.42 \frac{\text{ft}^3}{\text{sec}}}{\pi \frac{(0.6667 \text{ ft})^2}{4}} = 1.2 \text{ ft/sec}$$

$$\text{Re}_{AB} = \frac{v_{AB} D_{AB}}{\nu} = \frac{\left(1.2 \frac{\text{ft}}{\text{sec}}\right) (0.6667 \text{ ft})}{1.41 \times 10^{-5} \frac{\text{ft}^2}{\text{sec}}} = 5.7 \times 10^4$$

$$v_{BC} = \frac{Q}{\pi \frac{D_{BC}^2}{4}} = \frac{0.42 \frac{\text{ft}^3}{\text{sec}}}{\pi \frac{(0.5 \text{ ft})^2}{4}} = 2.1 \text{ ft/sec}$$

$$\text{Re}_{BC} = \frac{v_{BC} D_{BC}}{\nu} = \frac{\left(2.1 \frac{\text{ft}}{\text{sec}}\right) (0.5 \text{ ft})}{1.41 \times 10^{-5} \frac{\text{ft}^2}{\text{sec}}} = 7.4 \times 10^4$$

For PVC pipe, the specific roughness is $\epsilon = 5 \times 10^{-6} \text{ ft}$, so the relative roughness is

$$\frac{\epsilon}{D_{AB}} = \frac{5 \times 10^{-6} \text{ ft}}{0.6667 \text{ ft}} = 0.0000075$$

$$\frac{\epsilon}{D_{BC}} = \frac{5 \times 10^{-6} \text{ ft}}{0.5 \text{ ft}} = 0.00001$$

From the Moody diagram, $f_{AB} = 0.020$ and $f_{BC} = 0.019$, which are close to the initial estimate. The flow through the system is

$$Q = \boxed{190 \text{ gal/min}}$$

HYDRAULIC MACHINES

• SOLUTION

b) Write the energy equation between upper tank and point B

Assume that all water can be taken out from the pipe at the hydrant site.

$$z_1 = z_B + \frac{P_B}{\gamma} + \frac{V_B^2}{2g} + h_{AB}$$

$$(z_1 - z_B) - \frac{P_B}{\gamma} = \frac{V_B^2}{2g} + \frac{f_{AB} L_{AB} V_{AB}^2}{2g D_{AB}}$$

$$(150 - 90) - \frac{20 \times 144}{62.4} = \frac{V_B^2}{2g} \left(1 + \frac{f_{AB} L_{AB}}{D_{AB}} \right)$$

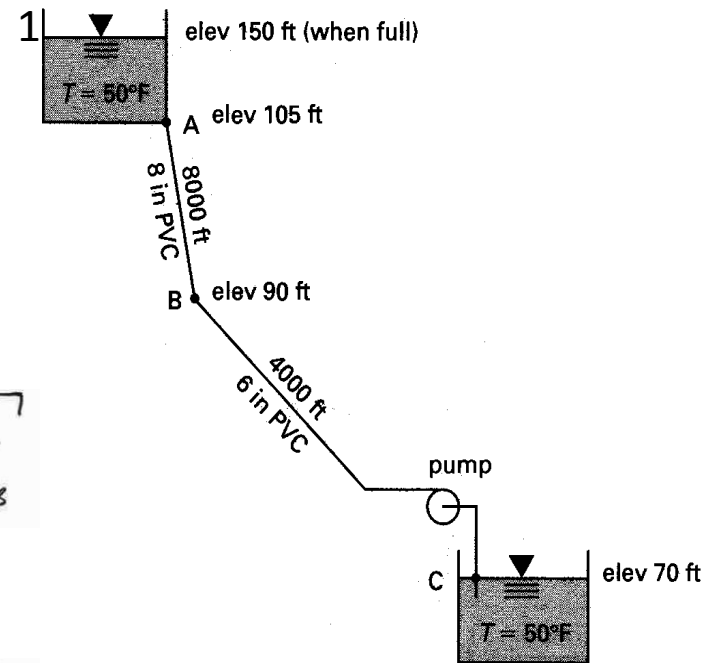
$$V_B^2 = \frac{2 \times 32.2 \times 13.85}{1 + \frac{8000 f_{AB}}{0.667}} \quad \Rightarrow \quad V_B = 29.865 \sqrt{\frac{1}{1 + 11994 f_{AB}}}$$

- Solve for velocity iteratively assuming a value of $f_{AB} = 0.02$ for the first iteration

$$f_{AB} = 0.02 \text{ (assumed)} \Rightarrow V_B = 1.924 \text{ ft/s} \Rightarrow Re = \frac{V_B D_B}{\nu}$$

$$Re = \frac{1.924 \times 0.667}{1.407 \times 10^{-5}} = 91222, \quad \epsilon/D = 0.0000075$$

$$f_{NEW} = 0.0183 \text{ (from Jain's eq.)}$$



HYDRAULIC MACHINES

- SOLUTION**

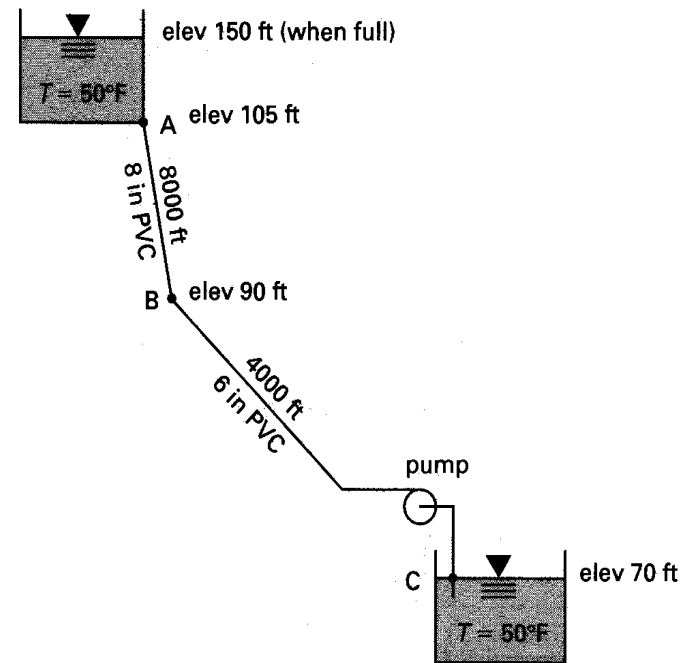
- Since assumed $f_{AB} = 0.02$ and computed $f_{AB} = 0.0183$ another iteration is required:

Iteration II:

$$f_{AB} = 0.0183 \Rightarrow V_B = 2.014 \text{ ft/s} \Rightarrow Re = 95465 \Rightarrow$$

$$f_{new} = 0.0181 \text{ OK} \Rightarrow Q_B = V_B A_B = 2.014 \times 0.349 = 0.703 \text{ ft}^3/\text{s}$$

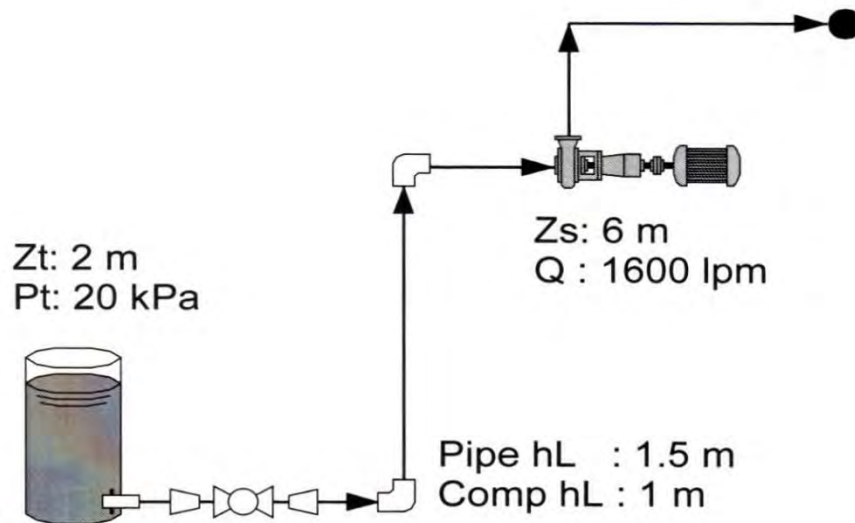
- Therefore the answer is
- $Q_B = 0.703 \text{ ft}^3/\text{s} = 315 \text{ gpm}$



HYDRAULIC MACHINES

Computation of NPSHA

7) A pump located 6 m above the sea level, has a NPSHr of 6.5 m at 1,600 lpm and is fed by a tank with water at 15°C pressurized at 20 kPa and with a liquid level of 4 m below the suction of the pump. The head losses in the suction pipeline is 1.5m and the losses in the other components (local losses) is 1.0 m. The discharge is 1,600 liters per minute. The schematic of Figure shows the pump suction. How much is the NPSH available?



Note: The NPSHA should be calculated to ensure that it EXCEEDS the NPSHR provided by the manufacturer to prevent cavitation in the pump.

HYDRAULIC MACHINES

$$h_{res} + h_{atm} = (20 + 101.3) \times 1000 / (999 \times 9.81) = 12.38 \text{ m}$$

$$h_{z(s)} = Z_t - Z_s = 2 - 6 = -4 \text{ m}$$

Vapor pressure (15°C) = 1.7057 kPa $\longrightarrow h_{vp} = 1705.7 / (999 \times 9.81) = 0.174 \text{ m}$

Head losses (given) = $h_{f(s)} = 1.5 + 1.0 = 2.5 \text{ m}$

$$\text{NPSHA} = 12.38 - 4 - 0.174 - 2.5 = 5.706 \text{ m}$$

Pump is under risk of cavitation!

