Introduction

Water Supply, sewerage and sanitation are not only the basic necessities of life, they are also crucial for achieving the goal of "Health for All". Increased sanitation coverage is directly linked to improvement of health status.

Water Supply is perhaps the most important and basic need that has to be provided with reliability, sustainability and affordability.

Water supply coupled with sanitation is essential in order to facilitate the citizens to lead a healthy and productive life. Integration of sanitation and sewerage schemes with water supply so that it is given adequate priority during the Plan period.

Treating of wastewater from storm water drains and industrial effluents before they enter the watercourses. Adoption of innovative and alternate technologies for safe disposal, recycling and reuse of waste water wherever possible.

Water supply components

- Water sources structures (Dams, wells, reservoirs)
- Surface water & Groundwater
- Pipelines from source
- Water treatment plant components
- Pumping stations
- Storage (elevated tanks)
- Distribution System

Preliminary studies for water supply projects:

The main factors required to be studied to supply a city with water system are:

- Sources of water available
- Quantity of water
- Population (present and future)
- Water consumption (present and future)
- Design Period (30 – 50 years)

Quantity of Water

The quantity of water to be supplied to any community depends on:

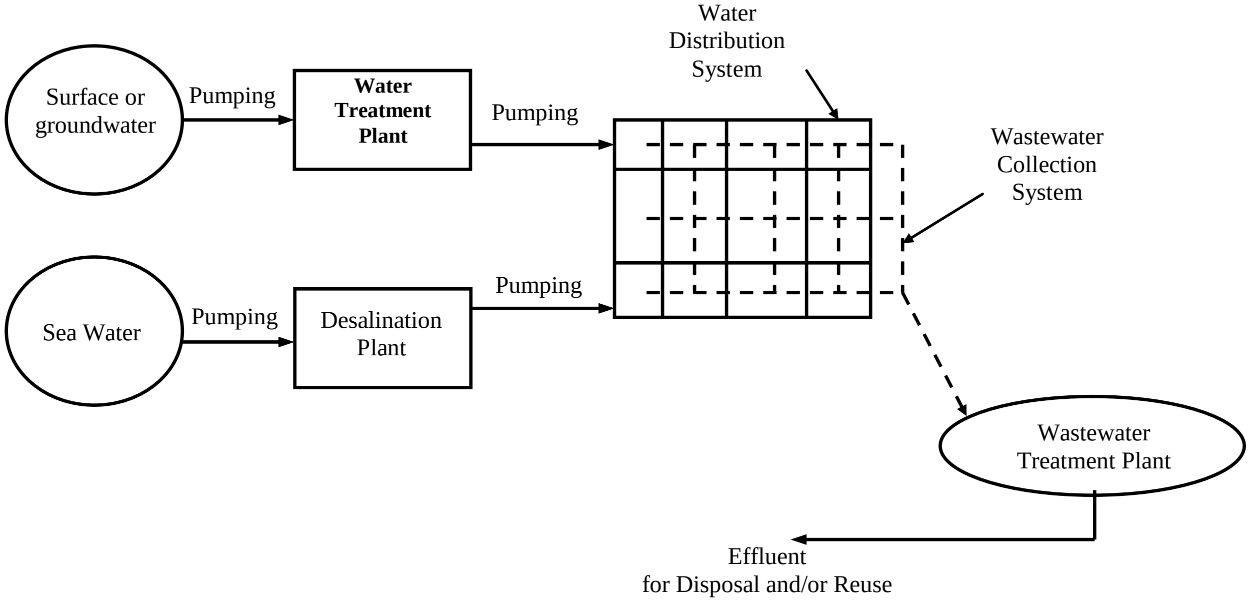
- The population
- The per capita consumption of water

Water supply works must to be designed to serve the present population as well as the future population

In the design of water works, it is necessary to estimate the quantity of water needed in the future (water demand).

- The number of people that will be served at the end of the design period
- The consumption rate
    - Average daily consumption in a community (m^3/day) = Total water use in one year /365 days
    - Per capita average daily consumption: the amoint of water used by individual per day, Lpcd = Avg. daily consumption in a community (Lpd) /mid year population

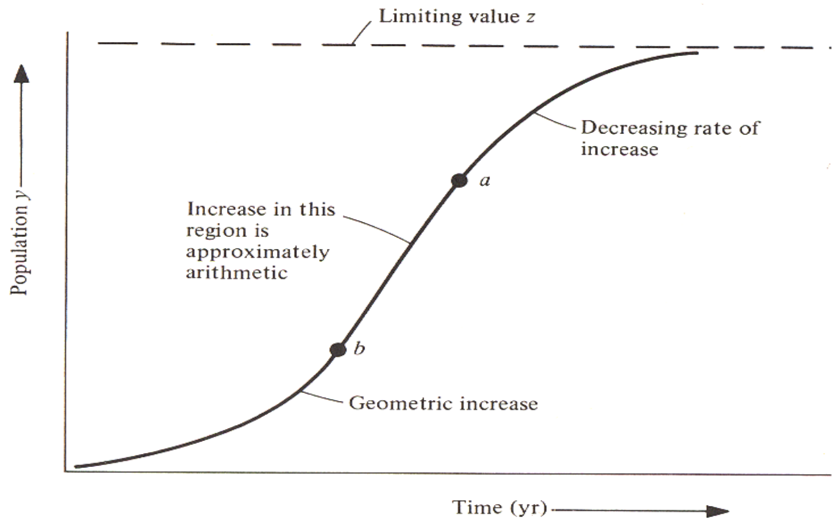
- Analysis of factors affecting water consumption (e.g. climate, economic level, population density, quality of water, etc.)



Population Estimation (forecast / projection)

- Population project periods may range from 5 to 50 years depending on the particular component of the system that is being designed (e.g. distribution system, treatment plant, pumping stations, etc.)
- Estimation of population depends on many factors such as:
    - Past census records (Bureau of census)
    - Economic conditions
    - Future growth of the economical and industrial activities in the region
- The rate of population growth can be expressed as a percent increase per year (e.g. 1.7% per year) (e.g. 17 people per year per 1000 population)
- Factors affect rate of increase of population
    - Natural growth (birth and deaths)
    - Immigration
    - Diseases
    - Economic development
    - Health care

A typical population growth curve has three segments: a geometric increase (exponential), an arithmetic increase and a decreasing rate of increase.



Methods for Estimation the future Population:

1- Arithmetic Method

- Short term population prediction, 1-10 years
- The rate of population growth (dp/dt) is constant:

  $dp/dt = K_a$ = arithmetic constant

  $P_t = P_o + K_a t$

  Where:

  P_t : the population at some time in the future

  P_o : the present population

  t : the period of projection

  K_a : the growth rate = $\Delta P/\Delta t$

Example:

Based on the previous population record, estimate the population in the year 2010.

Year	1960	1970	1980	1990	2000
Population	70000	82000	95000	105000	115000

Solution

$K_a = 1/n \ \{\sum(\Delta P/\Delta t)\} = 1/4\{(82,000 - 70,000)/10 + (95,000-82,000)/10 + (105,000 - 95,000)/10 + (115,000 – 105,000)/10\} = 1125$

$P_t = P_o + K_a t$

$P_{2010} = P_{2000} + K_a t = 115,000 + 1125 \times 10 = 126,250$ people

2- **Geometric Method** (uniform percentage method)

- Short term population prediction, 1-10 years
- The rate of population growth (dp/dt) is proportional to population (exponential growth)

  $dp/dt \propto P$

$$dp/dt = K_g P$$

Integrating both sides yields:

$$\ln P_t = \ln P_o + K_g \Delta t \quad \rightarrow \quad K_g = \ln (P_2 / P_1)/\Delta t$$

$$P_t = P_o e^{K_g \Delta t}$$

Example:

Solve the previous example using the geometric method.

Solution

K_g (1960-1970) = ln (82/70) / 10 = 0.0158

K_g (1970-1980) = ln (95/82) / 10 = 0.0147

K_g (1980-1990) = ln (105/95) / 10 = 0.0100

K_g (1990-2000) = ln (115/105) / 10 = 0.0091

Average $K_g = P_o e^{K_g \Delta t} = 115{,}000\ e^{10 \times 0.0124}$ = 130, 182 people

3- Logistic Method

- Long term population prediction, 10-50 years
- It is assumed that the population growth curve has an S shape and the city has a saturation that will not be exceeded (limiting population)
- From the available population record, we choose three values, two near the two ends of the record (P_o and P_2) and one in the middle of the record (P_1)

  $$P = P_{sat}/[1 + e^{\alpha + b \Delta t}]$$

  $$P_{sat} = [2P_o P_1 P_2 - P_1^2 (P_o + P_2)]/ (P_o P_2 - P_1^2)$$

  $$\alpha = \ln [(P_{sat} - P_2) / P_2]$$

  $$b = [1/n] \ln [P_o (P_{sat} - P_1)/ P_1 (P_{sat} - P_o)]$$

where

n = the time interval between succeeding censuses (e.g. 10, 20 years)

$\Delta t = t - t_o$

Example:

Estimate the population in 2010, If the population record of a city is:

Year	1960	1980	2000
Population	30,000	90,000	250,000

Solution:

$\Delta t = t - t_o = 2010 - 1960 = 50$ years

n = 20 years

$P_o = 30,000$ $P_1 = 90,000$ $P_2 = 250,000$

$P_{sat} = 1,530,000$ $a = 1.633$ $b = -.057$

$P_{2010} = 1,180,449 = 1,180,000$

- **Population Density**
  - Population density describes the physical distribution of population (people/km^2).
  - The total population of a city is needed to estimate the total volume of water or wastewater to be considered. But to design pipe systems for these flows, information about the population densities is required.
  - Population densities may be estimated from data collected on already developed areas or from zoning master plans for undeveloped areas.
  - The following table presents the range of population densities found in areas of different characters.

Residential zone classification	**Population density (people/km^2)**
Single-family dwellings (large lots)	1250 – 3700
Single-family dwellings (small lots)	3700 – 5500
Multiple-family dwellings	5500 – 25,000
Apartments	25,000 – 250,000

Example

Estimate the expected average water consumption rate (Lpcd) for the area shown below. Data on the expected saturation population densities and water demands are also given.

Industrial Area
30 ha
30,000 L/ha.d

Mosque
2 ha; 2000 c
50 Lpcd

High rise Buildings
50 ha
350 c/ha
450 Lpcd

Hospital
10 ha
200 beds – 700 Lpd/bed
400 employees --- 300 Lpd/employee

School
5 ha; 1500 students
200 Lpcd

Commercial Area
120 ha
200 people/ha; 30,000 L/ha.d

Park and Playground
15 ha
15,000 L/ha.d

University
60 ha
10,000 students
200 Lpcd

Single-family dwellings
200 ha
70 person/ha
450 Lpcd

Solution

Zone	**Area (ha)**	**Population**	**Consumption (Lpha.d)**	**Consumption (Lpcd)**	**Total consumption (m^3/d)**
Industrial	30	-	30,0000	-	900
Mosque	2	2000	-	50	100
High-rise Buildings	50	350x50=17,500	-	450	7,875
Hospital	10	200 beds 400 employee	-	700 300	140 120
School	5	1500	-	200	300
Commercial	120	200x120=24,000	30,000	-	3,600
Park and playground	15	-	15,000	-	225
University	60	10,000	-	200	2000
Single-family dwellings	200	70x200=14,000	-	400	6,300
Total		69,600	-		21,560

Average water consumption = 21,560 / 69,600 = 0.31 m^3/c.d = 310 Lpcd

Water uses

1- Domestic use

- Water used in houses, hotels…..for sanitary, culinary and other purposes such as:
    - Toilet flushing and bathing (sanitary): 80 % of the total domestic use.
    - Kitchen and drinking: 10 % of the total domestic use.
    - Clothes wash
    - Car washing, garden watering, house cleaning,….
- Domestic use represents 50 % of the total water consumption
- Common range : 75 – 380 Lpcd

2- Commercial and Industrial use

- Water used in industries and commercial establishments (stores, factories …..,)
- Industrial consumption depends on size of industry and whether the industry has its own water supply
- Commercial consumption represents 15% - 25% of the total water consumption

3- Public Use

- Water supplied to public building and used for public services (schools, street flushing, fire protection, prisons……)
- Common range: 50 – 75 Lpcd

4- Loss and Waste

- Water that is "uncounted for" due to:
    - Leaks from water pipes
    - Unauthorized connections
    - Errors in meter reading
    - Pump slippage
- Represents about 10% of the total water consumption

Total water consumption = 1 + 2 + 3 +4

Factors affecting Water Consumption

1- Size of the city

- Small cities might have much less water consumption that larger cities due to limited water use, absence of sewer system and water supply system and other reasons.
- In U.S., average daily per capita water consumption 130 – 1200 Lpcd.

2- Industrial and commercial activities

- Industrial water use has no direct relation to population
- Whether industry has its own water supply!
- Existing and future industrial water use must be carefully studied.
- Commercial water use is related to number of people employed.

3- Living Standard of population

In slum areas, domestic water use is low

4- Metering of water

Metering can reduce water consumption by as much as 50%

5- Climate

During hot, dry weather, water consumption is high

6- Other factors

- Water quality: water of poor quality will be used less than water of better quality
- Cost of water (water tariff)
- System pressure: high pressure will lead to greater use
- Maintenance and efficiency of the water supply system: good maintenance and high efficiency of system management means less leaks.

Variations in water consumption

- Water demand / consumption vary from hour to hour, from day to day, from month to another, from season to season, and from year to year.
- The average daily per capita consumption, Lpcd (avg. annual consumption rate) does not reflect variations in water demand.
- To evaluate variations in demand:
    - Must have complete pumping records
    - In the absence of data, you can estimate the rates of consumption:

      Max. daily rate = 1.8 × avg. daily rate

      Max hourly rate = 1.5 × max. daily rate = 2.7 × avg. rate

      Avg. daily rate (summer) = 1.2 × avg. daily rate

      Avg. daily rate (winter) = 0.8 × avg. daily rate

      Min. consumption rate = 0.2 to 0.5 × avg. daily rate

      (important for the design of pumping stations)

Goodrish formula (for moderate size cities):

$P = 180 \times t^{-0.1}$

Where P = Percentage of the annual avg. rate

t = length of the period in days

Fire Demand

- Annual volumes used are small but rate of use is high
- Fire demand can be estimated from the following formula:

$F\ (gpm) = 18\ C\ (A)^{0.5}$ where A = total building floor area excluding basement (ft^2)

$F\ (m^3/day) = 320\ C\ (A)^{0.5}$ where A in m^2

C = a coefficient related to the type of construction and existence of automatic sprinkler

C = 1.5 for wood frame construction

C = 0.8 for noncombustible construction

C = 1.0 for ordinary construction

- Fire demand calculated from above formulas should not be more than 8000 gpm (32 m^3 /min) in general, nor 6000 gpm (23 m^3 /min) for one story construction.

- Fire demand can also be estimated in terms of population:

  F (m^3/hr) = 231.6 $\sqrt{P}$ (1 – 0.01 $\sqrt{P}$) not exceed 1500 m^3/hr

  Where P is population in thousands

- Fire flow duration: The duration during the required fire flow must be available for 4 to 10 hours. National Board of Fire recommends providing for a 10 hours fire in towns exceeding 2500 in population.

- The water distribution system (the pipe network) should be designed to provide the larger of"
  - The maximum hourly demand or
  - The maximum daily demand + fire demand

Example

A community with a population of 22,000 has an average consumption of 600 Lpcd and a fire flow dictated by a building of ordinary construction with a floor area of 1000 m^2 and a height of 6 stories. Determine the required capacity of the pipe distribution system.

Solution

Avg. daily consumption = 600 × 22000 = 132 × 10^5 Lpd = 13,200 m^3 /day

Max. daily consumption = 1.8 × Avg. daily rate = 1.8×13,200 = 23, 760 m^3 /day

Max. hourly consumption = 2.7×Avg. daily rate = 2.7×13,200 = 35,640 m^3 /day

F = 320 C $\sqrt{A}$ = 320 ×1× ($\sqrt{1000 \times 6}$) = 24,787 m^3 /day (17.2 m^3 /min < 32 m^3 /min, o.k)

The fire flow duration = 10 hr

The total flow required during this day = 23, 760 m^3 /day + 24,787 (10/24) = 34,000 m^3 /day < Max. hourly rate (35,640 m^3 /day)

Then, the pipe capacity must be 35,640 m^3/day

Example:

Estimate the municipal water demands for a city of 225,000 persons, assuming the average daily consumption 600 Lpcd.

Step 1: Estimate the avg. daily demand "Q_{avg}"

Q_{avg} = 600 L/cd × 225,000 c = 135,000,000 L/d = 1.35×10^5 m^3/day

Step 2: Estimate the max. Daily demand "Q_{max}"

$Q_{max} = 1.8 \times 1.35 \times 10^5 = 2.43 \times 10^5$ m^3/day

Step 3: Calculate the fire demand

$Q\ (m^3/hr) = 231.6 \sqrt{P}\ (1 - 0.01 \sqrt{P})$

$= 231.6 \sqrt{225}\ (1 - 0.01 \sqrt{225}) = 2{,}952.9\ m^3/hr = 49.215\ m^3/min$

For 10-hr duration of daily rate:

$Q = 2{,}952.9\ m^3/hr \times (10\ h/day) = 0.3 \times 10^5\ m^3/day$

Step 4: sum of max. daily demand and fire demand

The total flow required $= 2.43 \times 10^5 + 0.3 \times 10^5 = 2.73 \times 10^5\ m^3/day$

Step 5: calculate the max. hourly demand:

Qmax. Hourly $= 2.7 \times 1.35 \times 10^5 = 3.645 \times 10^5\ m^3/day$

Step 6: compare and we take the larger which will be $3.645 \times 10^5\ m^3/day$

Design Periods for water supply components

The design period of the components of a water supply sysyem depends on:

- System life
- First cost (costly system needs long design period)
- Ease of expansion after design period
- Possibility that the system will become obsolete by technological advances

Item	Design period year	Remarks
Water source structure (dams, wells, reservoirs) Surface water Groundwater	 20 – 50 5	Design capacity of source should meet the max. daily demand during the design period (not on a continuous basis)
Pipelines from source	≥ 25	Cost of construction is much more than cost of materials. Life of pipe is very long
Water treatment plant components	10- 15	Expansion is generally simple. Most units are designed on the basis of avg. daily rate at the end of design period but the hydraulic design is based on the max. expected flow through the plant
Pumping stations	10	Expansion and modification are easy
Storage (elevated tanks)	Variable depending on cost but usually long	Its design linked to design of pumps
Distribution System	Indefinite (>100)	Replacement is very difficult and expensive. |Design is based on the max. anticipated development in the region.

Annual rate of increase

$P_n = P_o\,(1+R/100)^n$

Where R = rate of increase

n = Time period (years)

P_n = Pop. After n years

P_o = Present pop.

Example: If $R = 2\ \%$ & $P_o = 60000$ cap. It is required to estimate Pop. after 30 years

$$P_{30} = 60000\,(1+2/100)^{30} = 108680 \text{ capita}$$

Future Water Consumption

Percentage increase in W.C. = 10 % of percentage increase in population

Δ W.C. = (10/100) ΔPop.

Example: R = 2 % (% increase of Pop.)

W.Co (present W.C) = 200 L/c/d

It is required to determine W.C. after 30 years

Solution:

$W.C_{30} = W.C_o\ (1+0.1R/100)^n$

$W.C_{30} = 200\ (1+2/1000)^{30} = 212.4$ L/c/d

Example:

For a town of Pop. 60000 cap. And an average water consumption of 200 L/c/d. If the Pop. Increased at a rate of 2% per year and the increase of water consumption is 10 % of the percentage increase of Pop. Per year. Find the max. monthly, daily and hourly consumption discharge now and after 30 years.

Solution

$Q_{average}$ (now) = Pop. (now) × W.C. (now)

$= 60000 \times 200 = 12 \times 10^6$ L/d $= 12000\ m^3/d$

$Q_{max.\ Monthly} = 1.4\ (12000) = 16800\ m^3/d$

$Q_{max.\ daily} = 1.8\ (12000) = 21600\ m^3/d$

$Q_{max.\ hourly} = 2.7\ (12000) = 32400\ m^3/d$

At future $Q_{average}$ (future) = Pop. (future) × W.C. (future)

$P_{30} = 60000\ (1+2/100)^{30} = 108680$ capita

$W.C_{30} = 200\ (1+2/1000)^{30} = 212.4$ L/c/d

$Q_{max.\ monthly} = 1.4\ (108680 \times 212.4) = 32317\ m^3/d$

$Q_{max.\ daily} = 1.8\ (108680 \times 212.4) = 41550\ m^3/d$

$Q_{max.\ hourly} = 2.7\ (108680 \times 212.4) = 62325\ m^3/d$

Wastewater (Sewage)

Sources of Wastewater

<u>Domestic Wastewater</u>

Wastewater from residential area and commercial establishments

<u>Industrial Wastewater</u>

- Industrial sewage quantities vary with the industry type, size, industry development and many other factors
- Wastewater from industries must be discharged to sewer networks only after treatment

<u>Ground water Infiltration</u>

- Water enters sewers from underground water through poor joints, cracked pipes and walls of manholes
- Infiltration rate depends on:
  - Height of water table
  - Construction care of sewers
  - Properties of soils
- Infiltration rate is difficult to predict because construction conditions and soil properties very widely

<u>Storm water</u>

- Runoff from rainfall, snowmelt and street washing
- Storm water may be drained into a separate storm sewers or into a combined sewers which carry all types of sewerage

<u>Sewer</u>

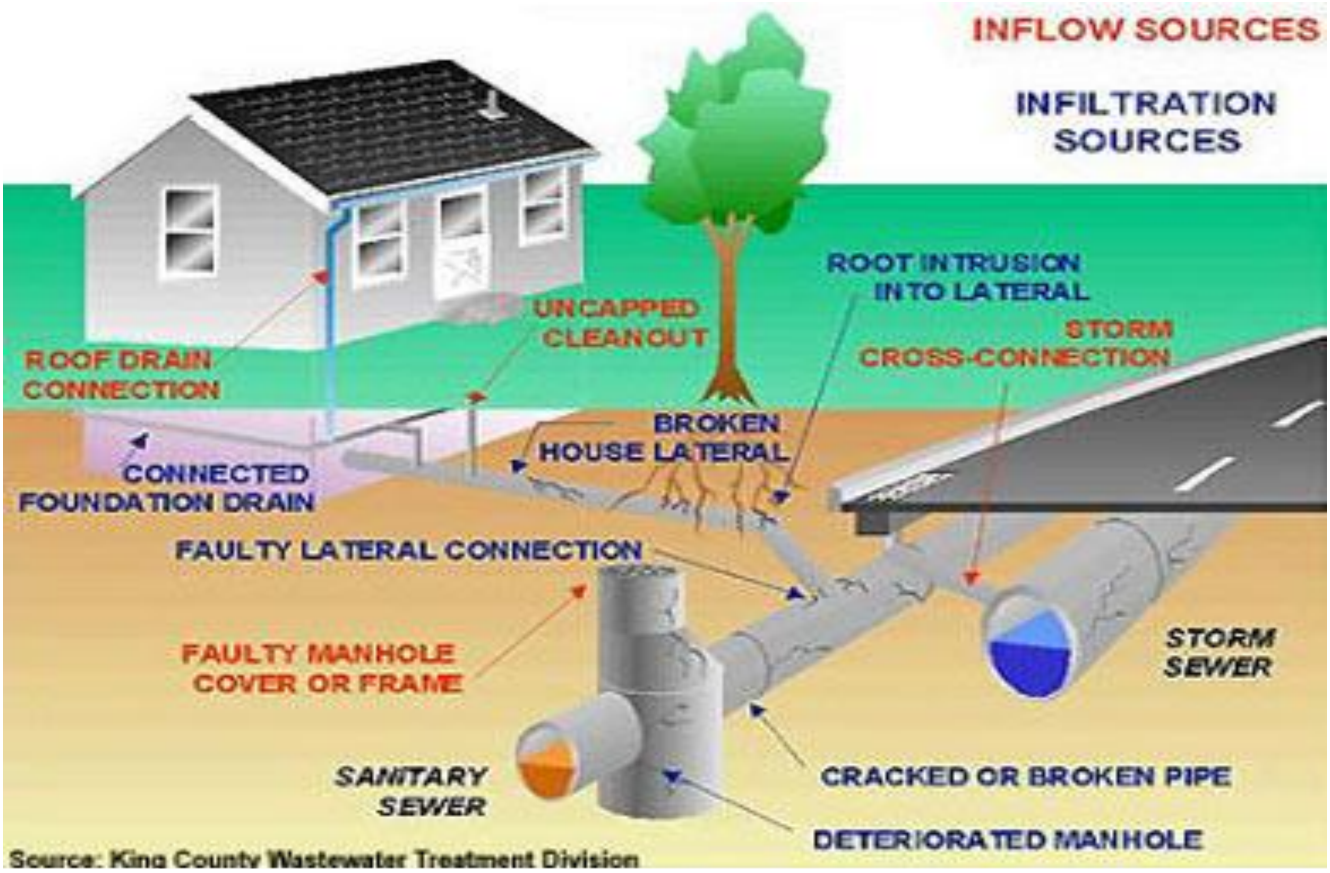
A pipe or conduit generally closed but not flowing full that carries sewage. Sewers are classified according to their use:

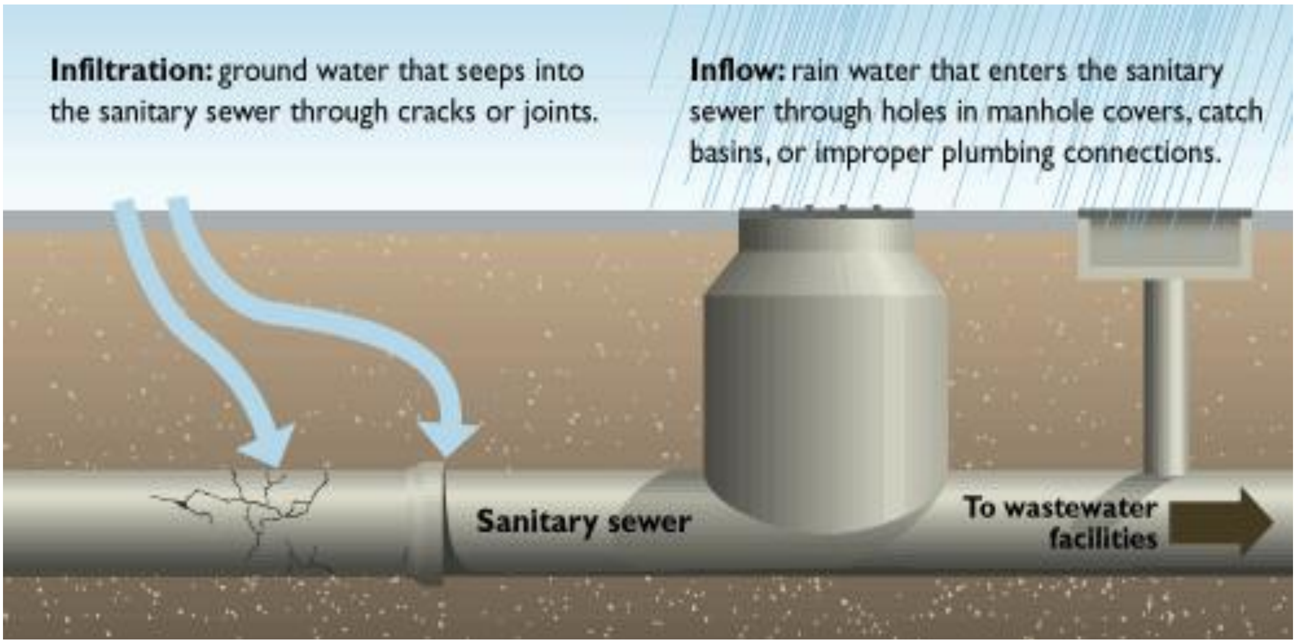
- Sanitary separate sewers: carry municipal wastewater and any subsurface water (infiltration) and surface water (inflow) that enter the sewer
- Storm Sewers: carry storm water
- Combined sewers: carry municipal wastewater and storm water

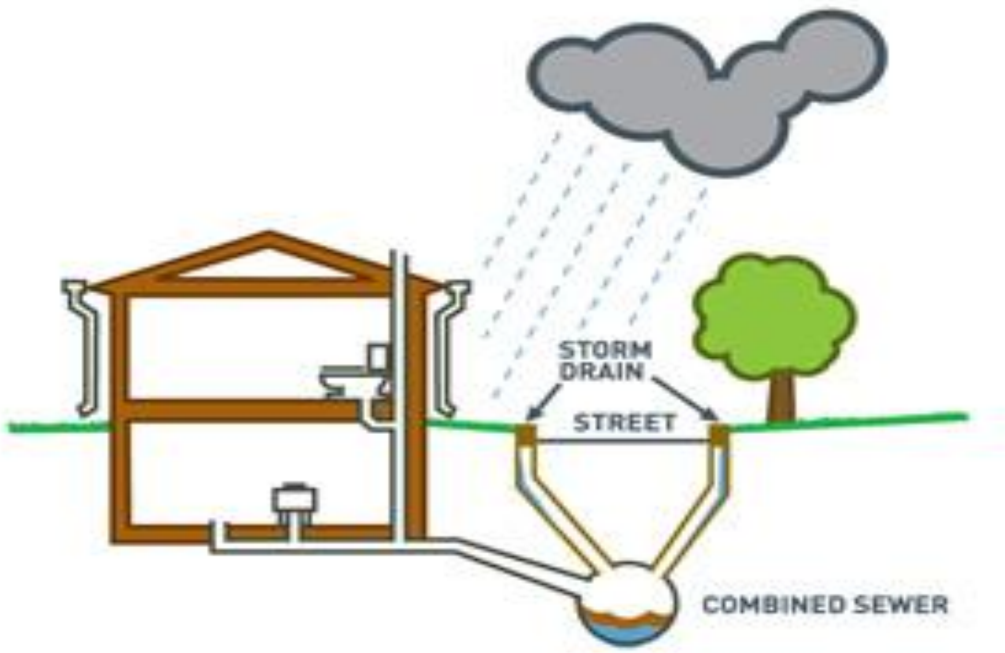
Inflow

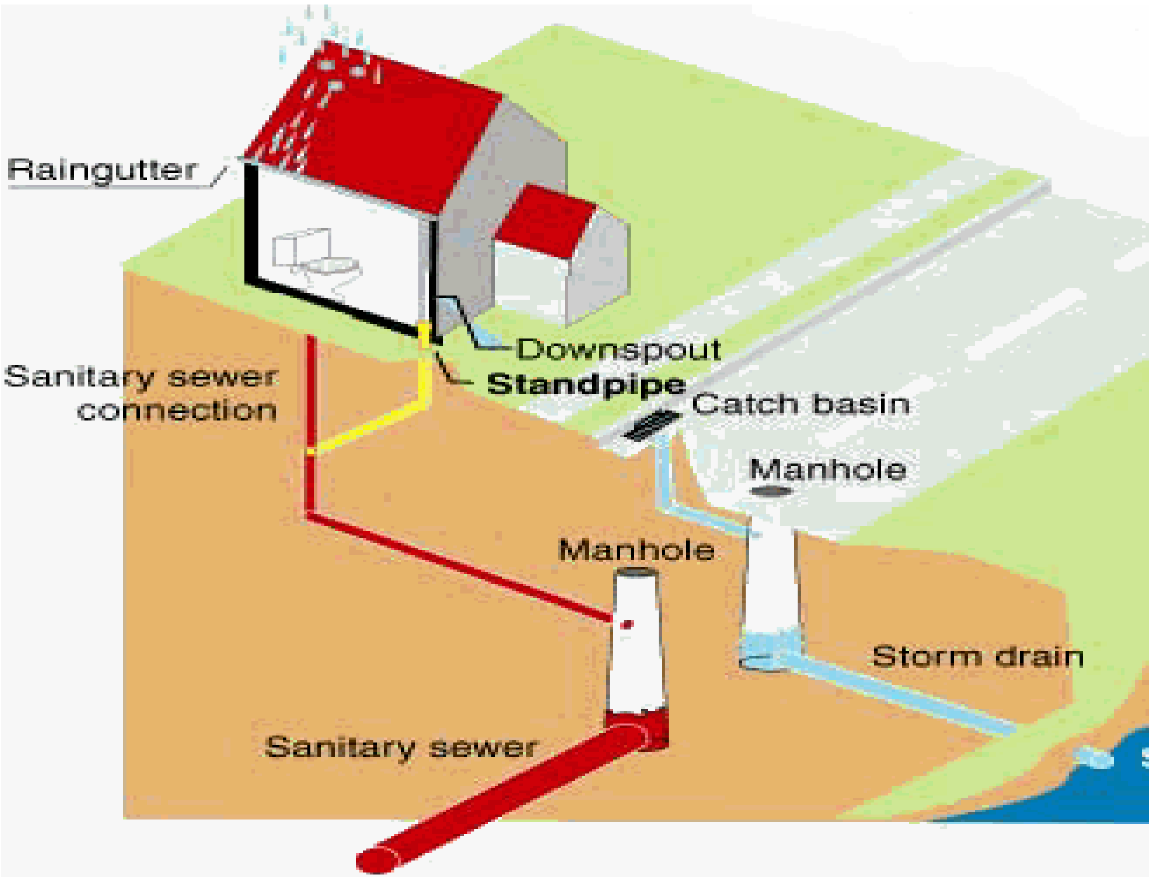
Water enters sewers from the surface during rainfall events through roof and basements drains and perforated manholes.

Wastewater Sewer System






STORM
DRAIN
STREET
COMBINED SEWER


Raingutter
Sanitary sewer
connection
Downspout
Standpipe
Catch basin
Manhole
Manhole
Storm drain
Sanitary sewer

Estimate of wastewater quantities

• Quantity of wastewater is related to water consumption

• The relationship between water demand and wastewater flow varies from city to city.

• 70% to 130% of the water consumed becomes wastewater

• Accurate wastewater determination comes from past gauging (measurements) records.

• Usually the recommended design flow rate is not less than 100 gal/cap-d (380 *l*/cap-d)

Variations of Wastewater Flow

- Like water consumption, wastewater flow varies with time of day, day of the week, season of the year, and weather conditions.
- Sewers must be designed to accommodate the maximum rate or there may be a backing of sewage into lower plumping fixture of buildings.
- Max. wastewater flow = Max. daily flow + max. infiltration
  - Max. daily flow = avg. flow × peak factor
  - Min. daily flow = 50% of the avg. flow
  - Peak factor (P.F) = max. flow/avg. flow
  - P.F. can be estimated by the following formulas:

    Harmon formula: $P.F = 1 + 14/(4 + P^{0.5})$

    Where P = Population in thousands

    Babbit formula:

    $P.F = 5/P^{0.2}$ (for residential area)

    $P.F = 4/P^{0.2}$ (for industrial areas)

Design Periods of Sewerage System Components

Item	Design period (yr)	Remarks	Design flow
Sewers	Indefinite	Expensive to replace and long lived	Q_{peak}
Pumping station	About 10	Easy to expand and short lived	Q_{avg} , Q_{min} , Q_{peak}
Treatment plant	15 - 20	Easy to expand	Q_{avg} , Q_{peak}

Example

In a city, the population growth is characterized as exponential with growth rate of 2% and the annual increase in water consumption is 2 Lpcd. Estimate the avg. and peak flow rates (m^3/d) in both the current year 2009 and the year 2019, if the Current population of a city = 150,000 and Current avg. water consumption = 100 Lpcd. Assume that 80% of the water use reaches sewers, and exclude infiltration.

Solution

Avg. W.W. flow rate = 0.80 × 100 Lpcd × 150,000 c = 12×10^6 Lpd = 12,000 m^3/d

P.F = 1 + 14/(4 + √P) = 1 + 14/(4 + √150) = 1.86

Peak W.W flow = 12,000 × 1.86 = 22,320 m^3/d

For the year 2019

Population = 150,000 × $e^{0,02 \times 10}$ = 183,210

Avg. per capita water use = 10 y × 2 Lpcd/y + 100 Lpcd = 120 Lpcd

Avg. W.W flow rate = 0.8 × 120 × 183,210 = 17,588,160 Lpd = 17,588 m^3/d

P.F = 1 + 14/(4 + √183) = 1.8

Peak W.W flow = 17,588 × 1.8 = 31,658 m^3/d

Distribution of Water

Methods of Distribution

Water may be distributed by:

1- Gravity distribution

   Used when water source is located at some elevation above the city to provide sufficient pressure

2- Pumping without storage

   - Water is pumped into water mains, and the flow must be constantly varied to match the fluctuations in demand
   - The least desirable method

3- Pumping with storage

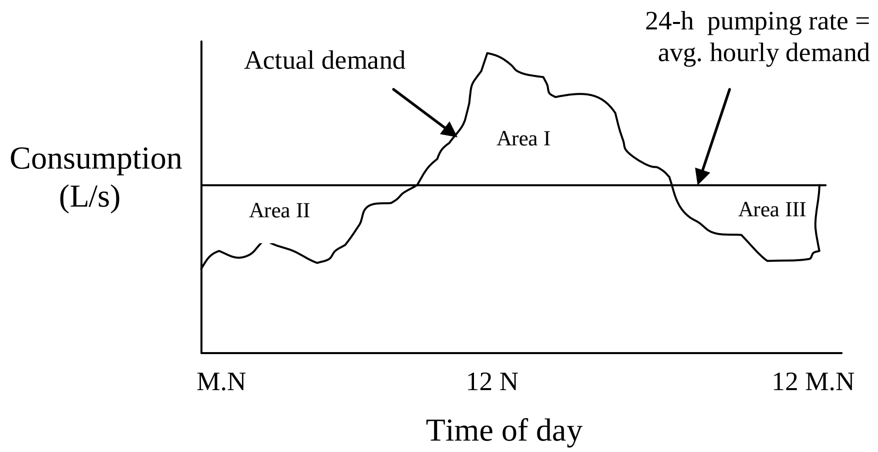
   - Water is pumped at a uniform rate for a given period with flow in excess of consumption (during periods of low water demand) is stored in elevated tanks or reservoirs. And during periods of high demands, the stored water augments the pumped water to equalize the pumping rate and to maintain more uniform pressure in the system
   - The most desirable and common method

Purpose of Storage

- To meet variable water demand while maintaining sufficient water pressure in the system (equalizing or operating storage)
- To provide storage for fire fighting and storage for emergencies such as failure of water source, repairs and maintenance
  - Storage volume = operating storage + fire storage + emergency storage
  - Emergency storage is about 25% - 30% of the operating and fire capacity requirements (normally it is 1 day average consumption)

Computation of Operating Storage

- When no information on water demand is available, equalizing storage is taken to be 15%-25% of the maximum daily consumption.
- When information on water demand is available (i.e. records of water consumption), operating storage can be calculated or found graphically:
    - **Supply-Demand Curve Method**

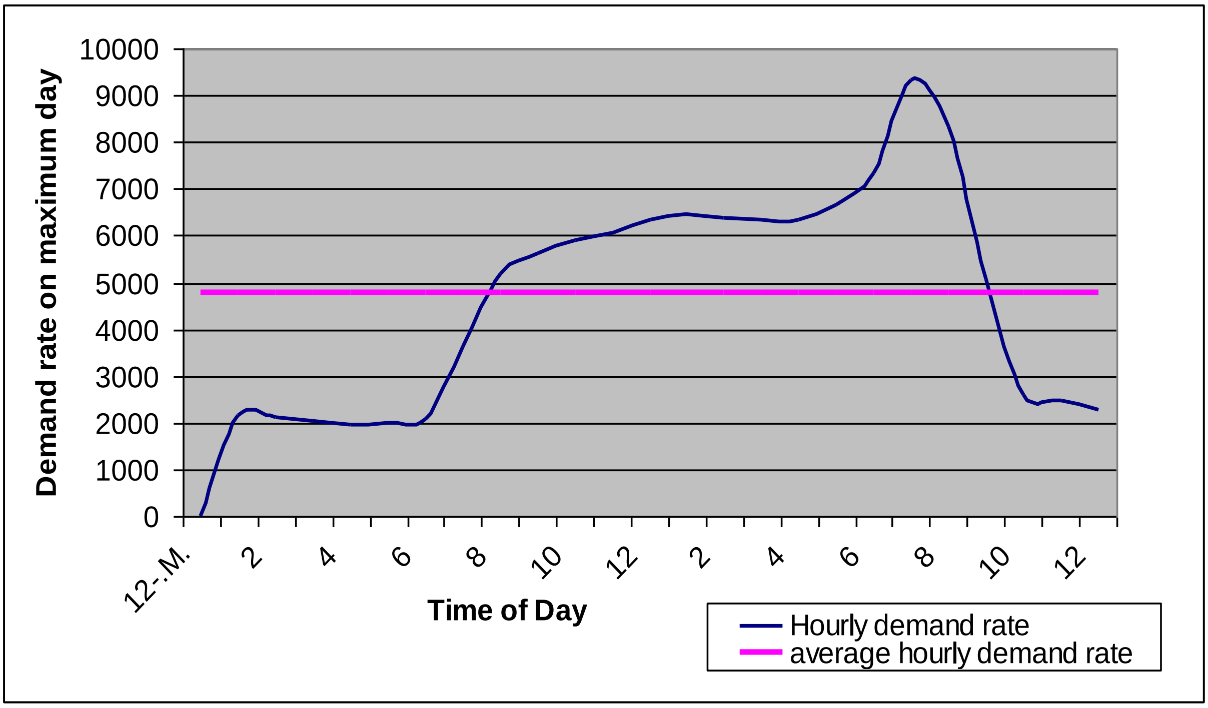


Hourly Demand for the Max. Day

Area I = Area II + Area III = Operating Storage

Time	Average Hourly Demand Rate (gpm)	Hourly Demand (gal)	Cunulative Demand (gal)	Hourly Demand as a percent of Average	Average Demand Minus Demand 286207.5-col3	Hourly Hourly
12-.M.	0	0	0	0.0		
1 A.M.	2170	130200	130200	45.5		156008
2	2100	126000	256200	44.0		160208
3	2020	121200	377400	42.3		165008
4	1970	118200	495600	41.3		168008
5	1980	118800	614400	41.5		167408
6	2080	124800	739200	43.6		161408
7	3630	217800	957000	76.1		68407.5
8	5190	311400	1268400	108.8	-25192.5	
9	5620	337200	1605600	117.8	-50992.5	
10	5900	354000	1959600	123.7	-67792.5	
11	6040	362400	2322000	126.6	-76192.5	

12	6320	379200	2701200	132.5	-92992.5	
1 P.M	6440	386400	3087600	135.0	-100192.5	
2	6370	382200	3469800	133.5	-95992.5	
3	6320	379200	3849000	132.5	-92992.5	
4	6340	380400	4229400	132.9	-94192.5	
5	6640	398400	4627800	139.2	-112192.5	
6	7320	439200	5067000	153.5	-152992.5	
7	9333	559980	5626980	195.7	-273772.5	
8	8320	499200	6126180	174.4	-212992.5	
9	5050	303000	6429180	105.9	-16792.5	
10	2570	154200	6583380	53.9		132008
11	2470	148200	6731580	51.8		138008
12	2290	137400	6868980	48.0		148808
					-1465275	1465275
		6868980				
Average hourly demand =		286208	gal			



- ## Mass-Digram Method

Plot the cumulative water consumption against time for the maximum day.

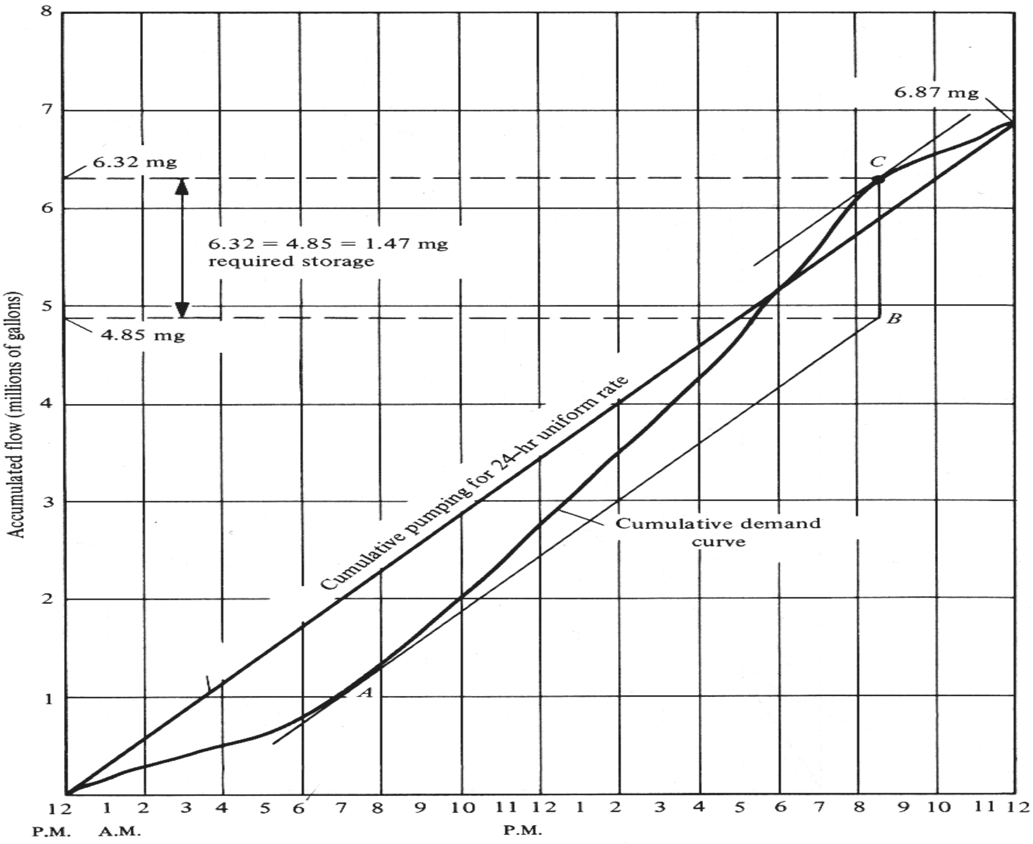


FIGURE 5-21. Operating storage for 24-hour pumping determined by use of the mass diagram.

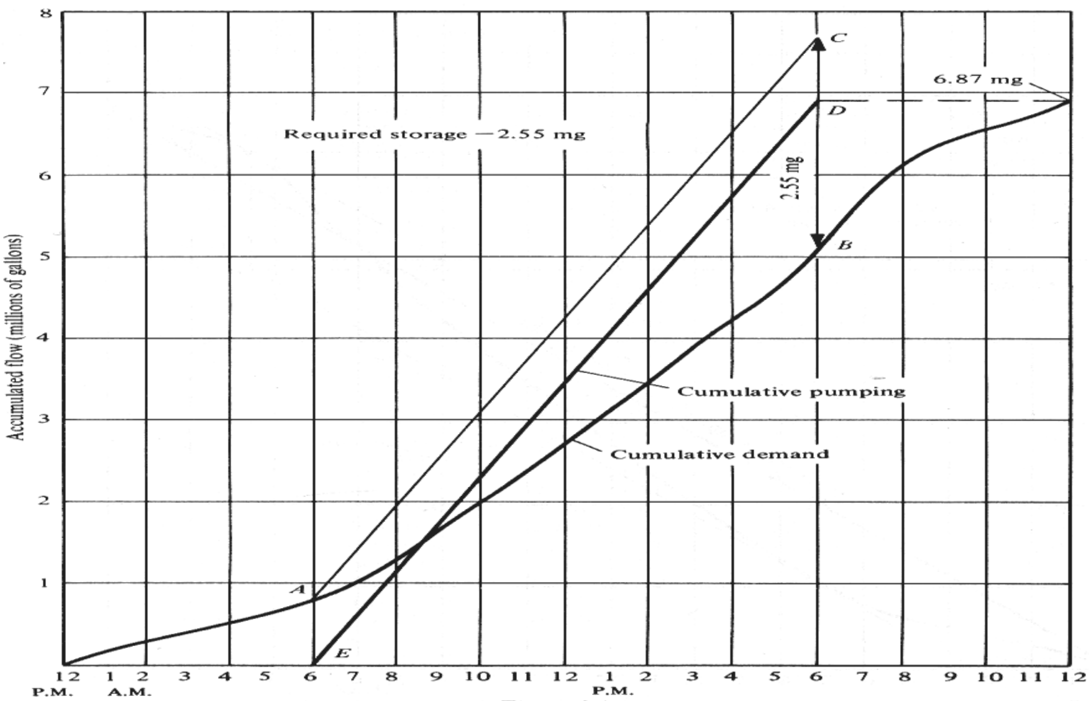


FIGURE 5-22. Mass diagram determination of equalizing storage for 12-hour pumping.

Types of Storage Reservoirs

- **Surface reservoirs**
- **Stand pipes**
    - Tall cylindrical tanks whose upper portion constitutes the useful storage and the lower portion serves to support the structure.
    - Standpipes over 15 m in height are not economical.
- **Elevated tanks**
    - Steel or concrete reservoirs.

Location of Storage Reservoirs

- The location of a reservoir affects its ability to equalize operating pressure throughout the distribution system.
- Normally, the reservoir should be located near the center of use to decrease friction losses by reducing the distance from the supply point to the area served.
- In large metropolitan areas a number of distribution reservoirs may be located at key points.

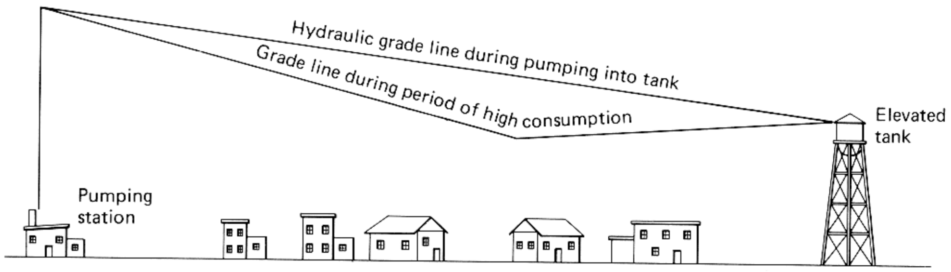


FIGURE 7-7
Effect of elevated storage on pressure.

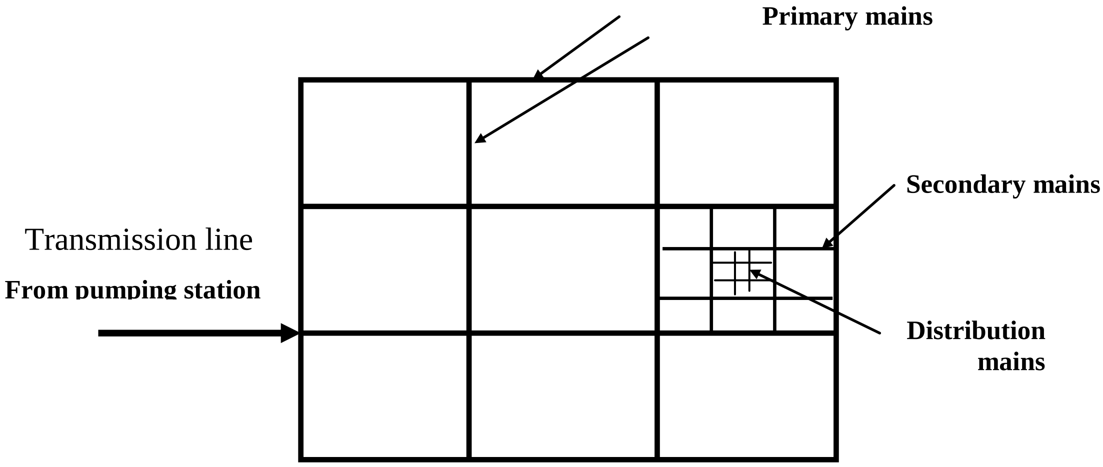
Pressure Required

- Recommended pressure rang (according to AWWA) is 400 to 500 KPa (60-70 psi or 4-5 bars) because this:
    - will supply water for buildings up to 10 stories in height
    - will supply sprinkler systems in buildings of 4-5 stories in height

- Pressure in the range of 150 to 300 kPa (20-40 psi) in residential areas of small towns may be used.

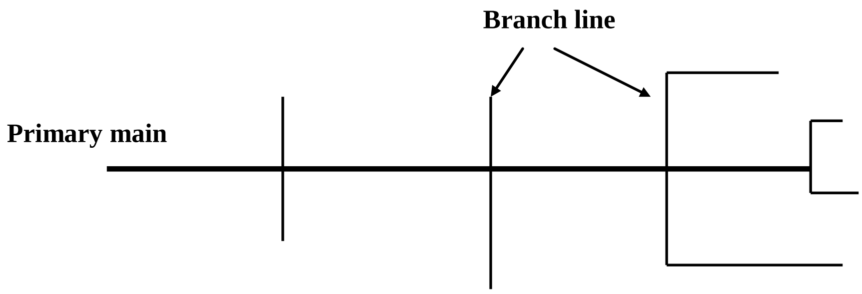
  N.B.: AWWA Standards for Water Utilities (AWWA - American Water Works Association)

The Pipe System

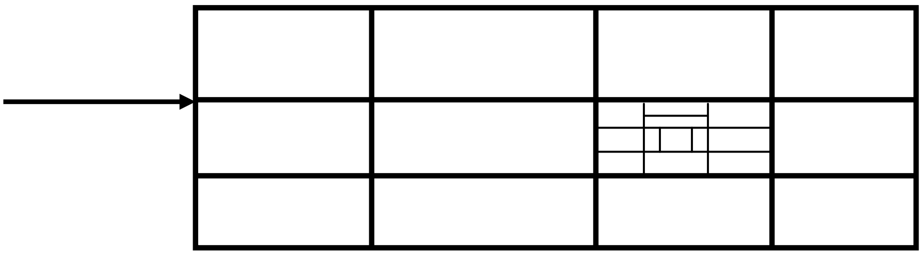


- The primary mains (arterial mains)
  - Carry flow from the pumping station to and from elevated storage tanks.
  - Laid in interlocking loops with the mains not more than 1 km.
  - Should be valved of not more than 1.5 km.
- The secondary mains
  - Form loops within the primary mains and run from one primary to another.
  - Carry water from primaries to the various areas.
- The small distribution mains
  - Supply water to every user and to fire hydrants.
  - Connected to primary, secondary or other smaller mains at both ends.

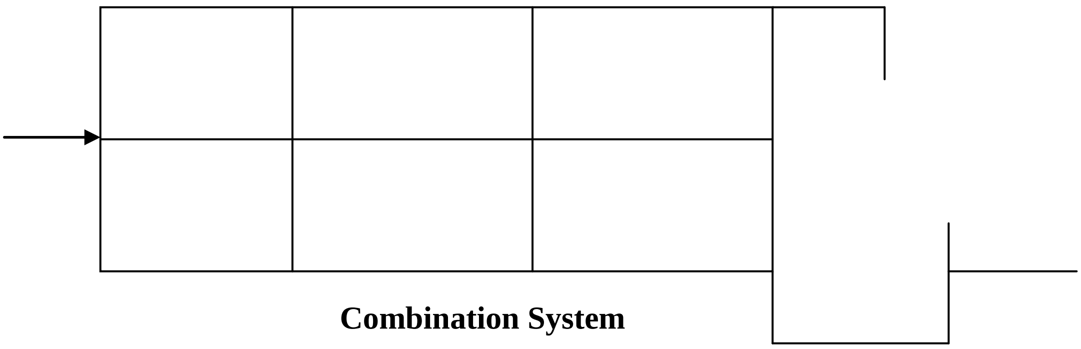
System Configurations



Branching system



Grid System



Combination System

Velocity and Pipe Sizes

Velocity range: 1 -2 m/s (at max. flow including fire flow)

Size

- Small distribution mains: ≥ 150 mm (6 in)

  ≥ 300 mm (12 in) in major streets
- For lines providing only domestic flow, the pipe diameter van be as small as 100 mm (4 in), with length not exceed 400 m if dead ended or not exceed 600 m if connected to the system at both ends
- In small communities, lines as small as 50-75 mm can be used but the length should not exceed 100 m if dead ended, and 200 if connected at both ends.

Hydraulics

Pressure (P)

- Pressure = force exerted by water mass under the influence of gravity per unit surface area

$$P = \frac{F}{A} = \frac{\text{mass x acceleration}}{A} = \frac{\text{m x g}}{A} = \frac{\text{volume x density x g}}{A}$$

$$P = \frac{A \times h \times \rho \times g}{A} = \rho \times g \times h = \gamma\ h$$

γ = specific weight of water (N/m^3)
If h = 1m , then P = 9800 (N/m^3) × 1 (m) = 9800 N/m^2 = 9800 Pa = 9.8 KPa

Units of pressure

- KPa = 0.102 m of H_2O
- Psi (Ib/in^2) = 6.9 KPa = 0.7 m of H_2O
- bar = 100 KPa = 10.2 m of H_2O
- 1 atm = 101.3 KPa = 14.7 psi = 10.33 m of H_2O

Pipe and Open-Channel Flow

- Pipe flow: e.g. pipe flowing full or under pressure
- Open-channel flow: e.g. open channel, partially filled sewers (the free-liquid surface is subject to atmospheric pressure)

Flow in Pipes

- Flow Regimes : Laminar and turbulent
  - Laminar flow: streamlines remain parallel to one another and no mixing occurs between adjacent layers
  - Turbulent flow: mixing occurs across the pipe
  - The transition from laminar to turbulent flow depends on the velocity in the pipe (V), the pipe diameter (d), the fluid density (ρ), and its viscosity (μ) [or

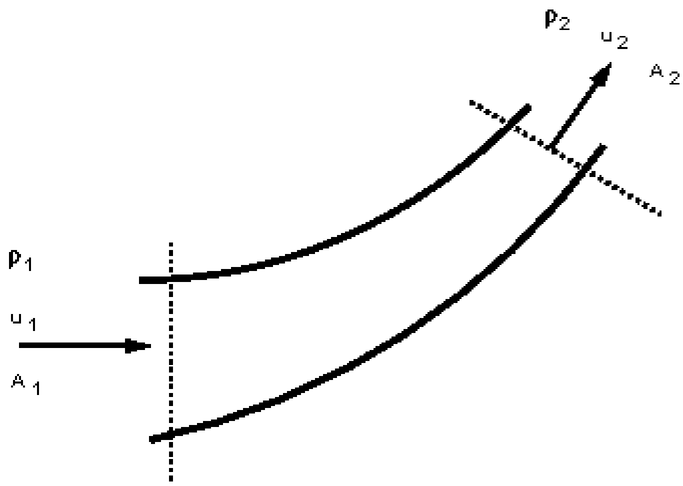
its kinematic viscosity, μ/ ρ] according to Reynolds Number (Re) (a dimensionless parameter):

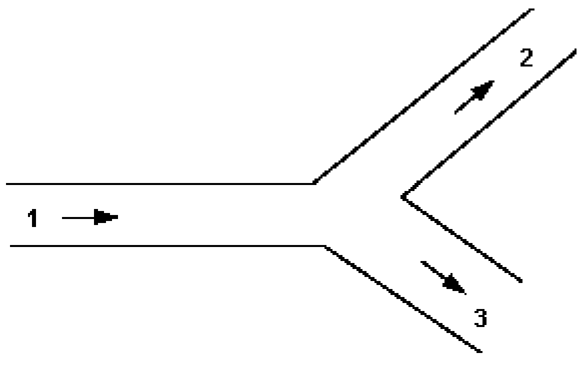
$$R_e = \frac{\mathrm{V(m/s)} \times \rho(kg/m^3) \times d(m)}{\mu(pa.s)} = \frac{V \times \rho \times d}{\mu} = \frac{V \times d}{\nu}$$

- If Re < 2100 (the flow is laminar)

Equation of Continuity

One concept which must be satisfied in all water flow problems is continuity of flow. This recognizes that no water is lost or gained and no cavities are formed or destroyed as the water passes through a pipe conduit. A proper understanding of flow continuity is essential to the design and evaluation of water supply systems. When the fluid is essentially noncompressible, such as water, continuity is expressed in the following equation:





$Q_{in}=Q_{out}$

$V_{in} \times A_{in} = V_{out} \times A_{out}$

The Bernoulli Theorem

The basic principle that Bernoulli discovered is the one most often used in hydraulics and is generally described as the law of conservation of water energy, or simply Bernoulli's Theorem. It is one of the most fundamental and far-reaching statements concerning fluid mechanics and it applies Newton's law of conservation of energy to the flow of water. It is stated in the equation:

$$\frac{P_1}{\gamma} + \frac{V_1^2}{2g} + Z_1 = \frac{P_2}{\gamma} + \frac{V_2^2}{2g} + Z_2 + h_L$$

$$\frac{V_1^2}{2g} = velocity - head$$

Where P/γ= pressure head

Z = elevation head

h_L = friction losses + minor losses= head losses due to friction and momentum changes at exits, entrances, changes in cross section

The Hazen-Williams formula has been developed specially for use with water and has been generally accepted as the formula used for pipe flow problems.

$$v = 0.849CR^{0.63}s^{0.54}$$

v = velocity of flow, m/s
R = hydraulic radius, m
S = slope of the energy gradient
C = a roughness coefficient

$$Q = 0.849CAR^{0.63}s^{0.54}$$

Pipe Material	C	n
Asbestos Cement	130–140	0.011–0.015
Cast Iron		
Plain	90–100	0.014–0.016
Cement-lined	100–130	0.012–0.015
Concrete		
New	120–130	0.011–0.015
Old	100–120	0.012–0.014
Plastic	140–150	0.009–0.010

Values of C in the Hazen-William Formula and of n in the Manning Formula.

Head loss due to Friction:

Head loss as a result of friction (hf) can be computed using the Hazen Williams Equation:

$h_f = KQ^{1.85}$

Where $$K = \frac{10.7L}{C^{1.85} d^{4.87}}$$

Q = flow through the pipe (m^3/s)
L = pipe length (m)
D = pipe diameter (m)
C = Hazen-Williams coefficient = roughness coefficient

Graphical solution of the Hazen-Williams equation for C = 100 can be made by the use of a nomograph.

$S_c = S_{100}\ (100/C)^{1.85}$
$d_c = d_{100}\ (100/C)^{0.38}$
$Q_c = Q_{100}\ (C/100)$

Where S = slope of the energy gradient line (E.G.L) = h_f/L

Example

Determine the head loss in a 1000 m pipeline with a diameter of 500 mm that is discharging 0.25 m3/s. Assume that C = 130.

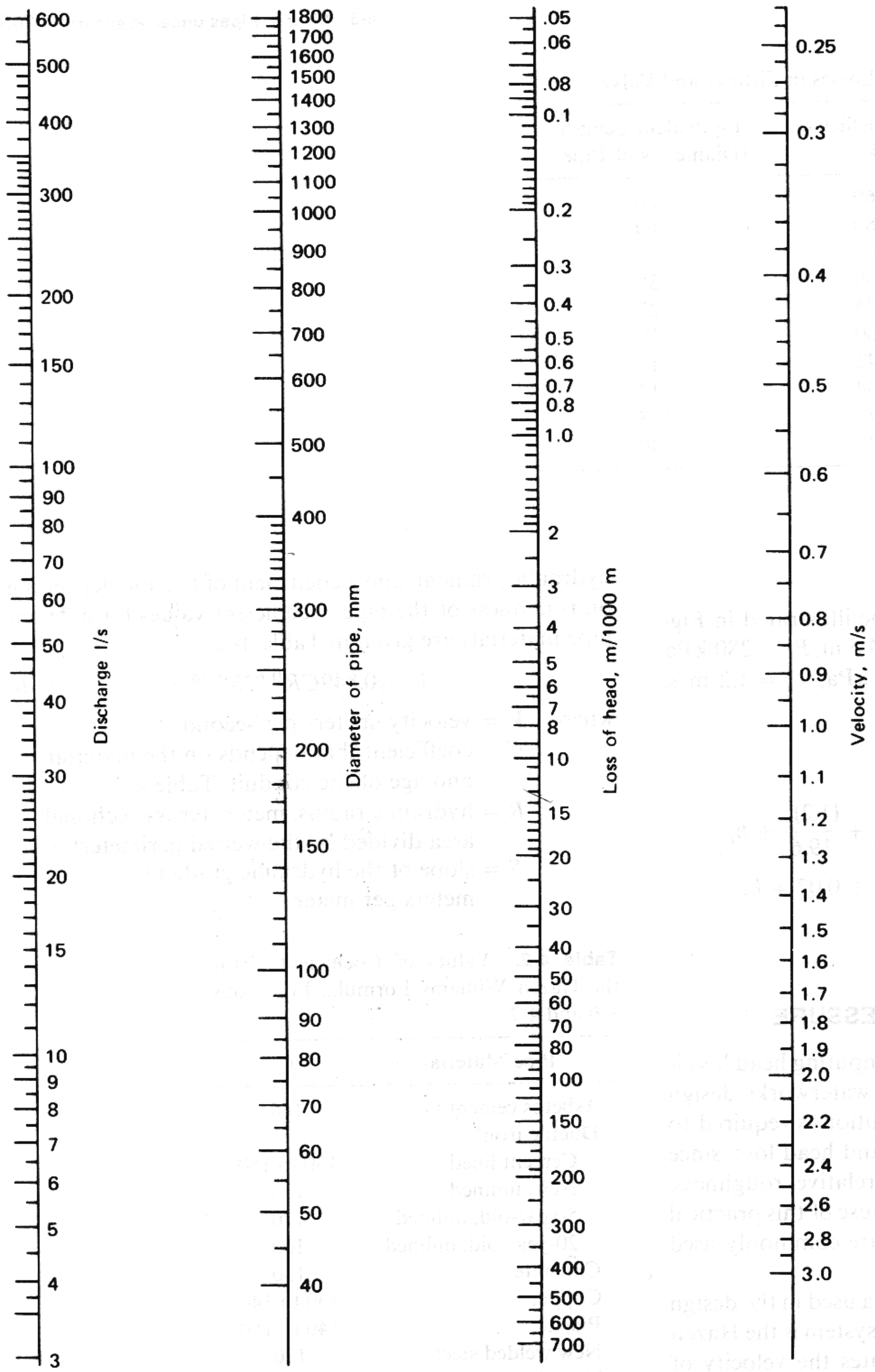
Solution
Using the Hazen-Williams equation

$$h_f = KQ^{1.85} = \frac{10.7L}{C^{1.85} d^{4.87}} Q^{1.85} = \frac{10.7 \times 1000}{130^{1.85} 0.5^{4,87}} 0.25^{1.85} = 2.96m$$

Using the nomograph for C = 100
At Q = 250 L/s and d = 500 mm $\rightarrow$ $S_{100} = h_f/L$ = 4.6 m/1000 m
$S_{130} = S_{100}\ (100/C)^{1.85} = (4.6/1000) \times (100/130)^{1.85} = 2.83/1000$

$h_f = S \times L = (2.83/1000) \times 1000 = 2.83$ m

- Graphical solution of the Hazen-Williams equation for C = 100 can be made by the use of a nomograph (Fig. 3-3, U.S system). The following nomograph is in the SI system (metric system)



Nomograph for Hazen William Equation
based on C = 100

Example:

A cast-iron water pipe, 400 mm in diameter, carries water at a rate of 0.125 cms. Determine, by means of the Hazen-Williams formula. The slope of the hydraulic gradient of this pipe and the velocity of flow.

<u>Solution</u> <u>1. Graphical solution</u>

Use the nomograph, line up the known value, d=381, the actual diameter of a nominal 400-m pipe, and Q=0.125, and find

S = 0.0045 m/m

v = 1.09 m/s

Example:

An asbestos cement water pipe (C=140) with a diameter of 300 mm has a slope of the hydraulic gradient of 0.0025 m/m. Determine, using the Hazen-Williams formula, the capacity of the pipe and the velocity of flow.

<u>Solution</u> <u>1. Graphical solution</u>

Use the nomograph, with d=305 mm, the actual diameter of a 300-mm pipe, and s=0.0025, find Q = 0.048 cms and v = 0.66 m/s

However, it must be remembered that the nomograph, as indicated was constructed for C=100, whereas pipe in question has a C=140. Consequently, the value of Q and v need to be corrected.

Design of Water Distribution System

The design includes the following steps:

- Prepare a detailed map of the area to be served showing topographic contours (or controlling elevations) and locations of present and future streets and lots.

- Mark the layout of the pipe network including main feeders, secondary feeders, distribution mains and storage reservoirs.

- Estimate the present and expected population and the spatial distribution of the population.

- Estimate the design flow.

- Disaggregate flow to various nodes of the system.

- Assume sizes of pipes based on water demands and code requirements.

- Analyze the system (for each sub-area) for flows, pressures and velocities and adjust sizes to ensure that pressures at nodes and velocities in pipes meet the criteria (i.e. pressure = 40 – 75 psi, velocity = 1 –2 m/s) under a variety of design flow conditions.
    - This can be done in a variety of ways and this what we mean by the hydraulic analysis of a water distribution system (the determination of flows, headlosses in various pipelines and the resulting residual pressure).
    - Note: Variables in the headloss equation:
        - Length (L)
        - Diameter (d)
        - Flow (Q)
        - Headloss (h_L or S)
        - Roughness (C)

      Known variables: L and C

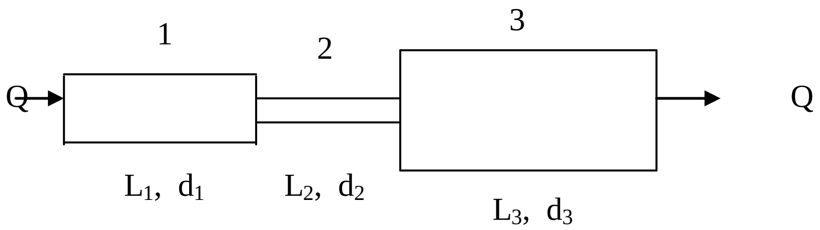
→**Method of Analysis**

- Equivalent pipe method
- Hardy cross method
- Method of sections
- Circle method
- Computer models

Equivalent Pipe Method

- An equivalent pipe is an imaginary pipe that replaces a real system of pipes such that the head losses in the two systems are identical for a quantity of flow.

- Pipes in Series



$Q = Q_1 = Q_2 = Q_3$

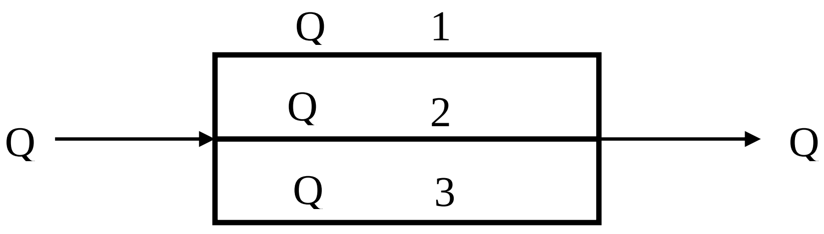
$h_f = h_{f1} + h_{f2} + h_{f3}$

$h_f = K_1\, Q^{1.85} + K_2\, Q^{1.85} + K_3\, Q^{1.85} = Q^{1.85}\,(K_1 + K_2 + K_3)$

$K_{eq}\, Q^{1.85} = Q^{1.85}\,(K_1 + K_2 + K_3)$

$K_{eq} = K_1 + K_2 + K_3$

- Pipes in Parallel



$h_f = h_{f1} = h_{f2} = h_{f3}$

$Q = Q_1 + Q_2 + Q_3$

But $h_f = K\,Q^{1.85}$ $\rightarrow$ $Q = (h_f / K)^{0.54}$

$$Q = (h_f / K_1)^{0.54} + (h_f / K_2)^{0.54} + (h_f / K_3)^{0.54}$$

$$(h_f / K_{eq})^{0.54} = (h_f / K_1)^{0.54} + (h_f / K_2)^{0.54} + (h_f / K_3)^{0.54}$$

$$(1 / K_{eq})^{0.54} = (1 / K_1)^{0.54} + (1 / K_2)^{0.54} + (1 / K_3)^{0.54}$$

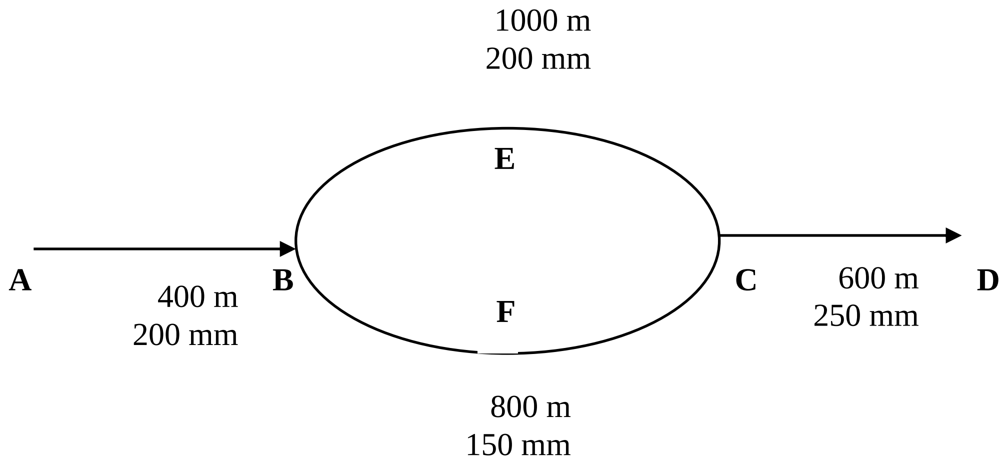
$$(1 / K_{eq}^{0.54}) = (1 / K_1^{0.54}) + (1/ K_2^{0.54}) + (1 / K_3^{0.54})$$

$$K_{eq}^{-0.54} = K_1^{-0.54} + K_2^{-0.54} + K_3^{-0.54}$$

Example

Find an equivalent p ipe 2000 m in length to replace the pipe system shown below.

Assume C = 100



Solution

Replace the parallel lines BEC and BFC with one equivalent line 1000 m in length

$$K_{eq}^{-0.54} = K_1^{-0.54} + K_2^{-0.54}$$

$$K = \frac{10.7\,L}{C^{1.85}\,d^{4.87}}$$

$$K_1 = (10.7 \text{ x } 1000) / (100^{1.85} \text{ x } 0.2^{4.87}) = 5412.1$$

$$K_2 = (10.7 \text{ x } 800) / (100^{1.85} \text{ x } 0.15^{4.87}) = 17575.6$$

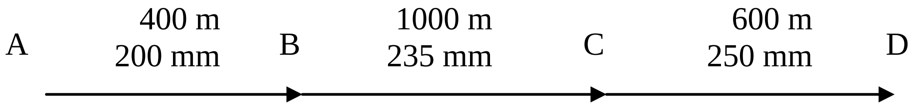
$$K_{eq} = (10.7 \text{ x } 1000) / (100^{1.85} \text{ x } d^{4.87}) = 2.13 / d^{4.87}$$

$$(2.13 / d^{4.87})^{-0.54} = 5412.1^{-0.54} + 17575.6^{-0.54} = 0.0147$$

$(2.13 / d^{4.87})^{-0.54} = 0.0147$

$0.665\ d^{2.63} = 0.0147 \rightarrow d = 0.235\text{ m} = 235\text{ mm}$

Replace the serial line AB, BC, and CD with one equivalent line 2000 m in length.



$K_{eq} = K_1 + K_2 + K_3$

$K_1 = = (10.7 \text{ x } 400) / (100^{1.85} \text{ x } 0.2^{4.87}) = 2164.8$

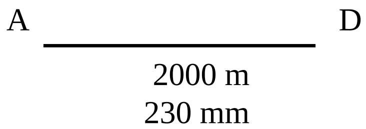
$K_2 = 2.13 / d^{4.87} = 2.13 / 0.235^{4.87} = 2461.2$

$K_3 = (10.7 \text{ x } 600) / (100^{1.85} \text{ x } 0.25^{4.87}) = 1095.4$

$K_{eq} = (10.7 \text{ x } 2000) / (100^{1.85} \text{ x } d^{4.87}) = 4.27\ d^{-4.87}$

$4.27\ d^{-4.87} = 2164.8 + 2461.2 + 1095.4 = 5721.4$

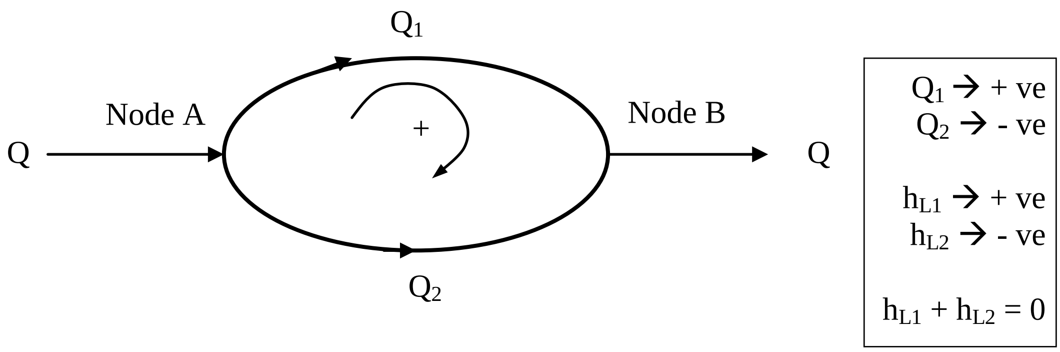
$d = 0.228 = 228\text{ mm} = 230\text{ mm}$



⇒The Hardy Cross Method

- The method of equivalent pipe cannot be applied to complex systems because crossovers results in pipes being included in more than loop and because a number of withdrawal points are normally throughout the system.

- In any network of pipes, the following conditions must be satisfied:
    - Flow into each junction (node) must equal flow out of the junction.
    - The sum of head losses around each loop must be zero.



- The method consists of :
    - Flows are assumed to each of the pipe with directions such as $Q_{in} = Q_{out}$
    - Correct the flow in each loop according to balanced head loss.

For any pipe in a loop → $Q_i = Q_{io} + \Delta Q$ ---------(1)

Q_i = actual flow in the pipe

Q_{io} = assumed flow

ΔQ = required flow correction

The headloss in the pipe is $h_{Li} = K_i\, Q_i^{x}$ --------(2)

Substitute (1) in (2):

$h_{Li} = K_i\,(Q_{io} + \Delta Q)^x$

$h_{Li} = K_i\,(Q_{io}^{x} + x\, Q_{io}^{x-1}\,\Delta Q) = K_i\, Q_{io}^{x} + x\, K_i\, Q_{io}^{x-1}\,\Delta Q$

For a balanced flow, the sum of head losses must equal zero:

$$\sum h_{Li} = 0 \qquad \text{for } i = \text{number of pipes} = 1 \text{ to } n$$

$$\sum h_{Li} = \sum [K_i Q_{io}^x + x K_i Q_{io}^{x-1} \Delta Q + \ldots\ldots\ldots] = \sum K_i Q_{io}^x + \sum x K_i Q_{io}^{x-1} \Delta Q$$

If ΔQ is small compared with Q_{io}, we may neglect the terms of the binomial series after the second one:

The above equation can be solved for ΔQ

$$\boxed{\Delta Q = \frac{-\sum h_{Li}}{x \sum (h_{Li} / Q_{io})}} \quad \text{-------(3)}$$

For Hazen-Williams equation x = 1.85

Where Q_{io} = assumed flow in pipe i

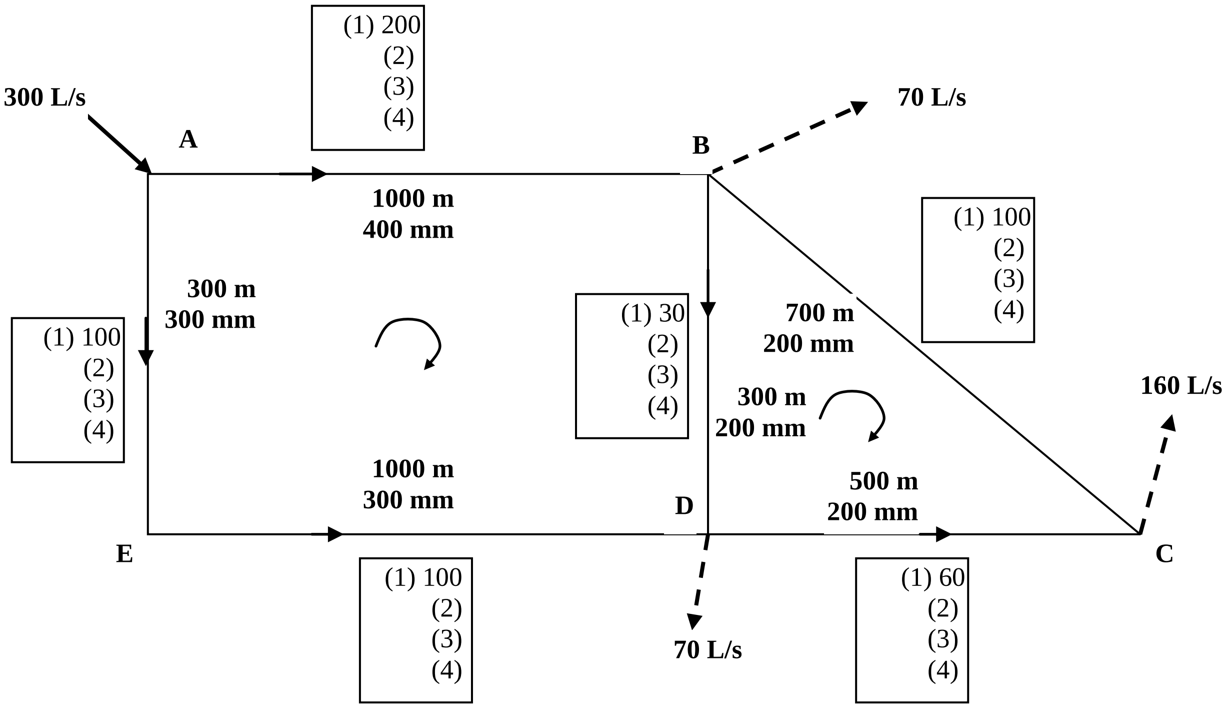
h_{Li} = headloss due assumed flow in pipe i

i = 1 to n

⇒The procedure for solving pipe network using the Hardy Cross method:

- Assume a direction and quantity of flow for each pipe in the system such that $Q_{in} = Q_{out}$
- Select one pipe loop in the system and compute the net headloss for that loop based on assumed flows (use the nomograph for Hazen-Williams).
- Compute the flow correction using Eq. 3 and correct each of the flows in the loop by this amount.
- Apply this procedure to each pipe loop in the system.
- Repeat for earlier loops until correction get significantly small (within 5% of the flow in any line).
- Adjust the pipe sizes to reduce or increase velocities and pressure and repeat the procedure until a satisfactory solution is obtained.

Example



$h_L/L = S = 0.00213\ (Q^{1.85} / d^{4.87})$ For C = 100, Q in (m^3/s), d in (m)

OR you can use the nomograph

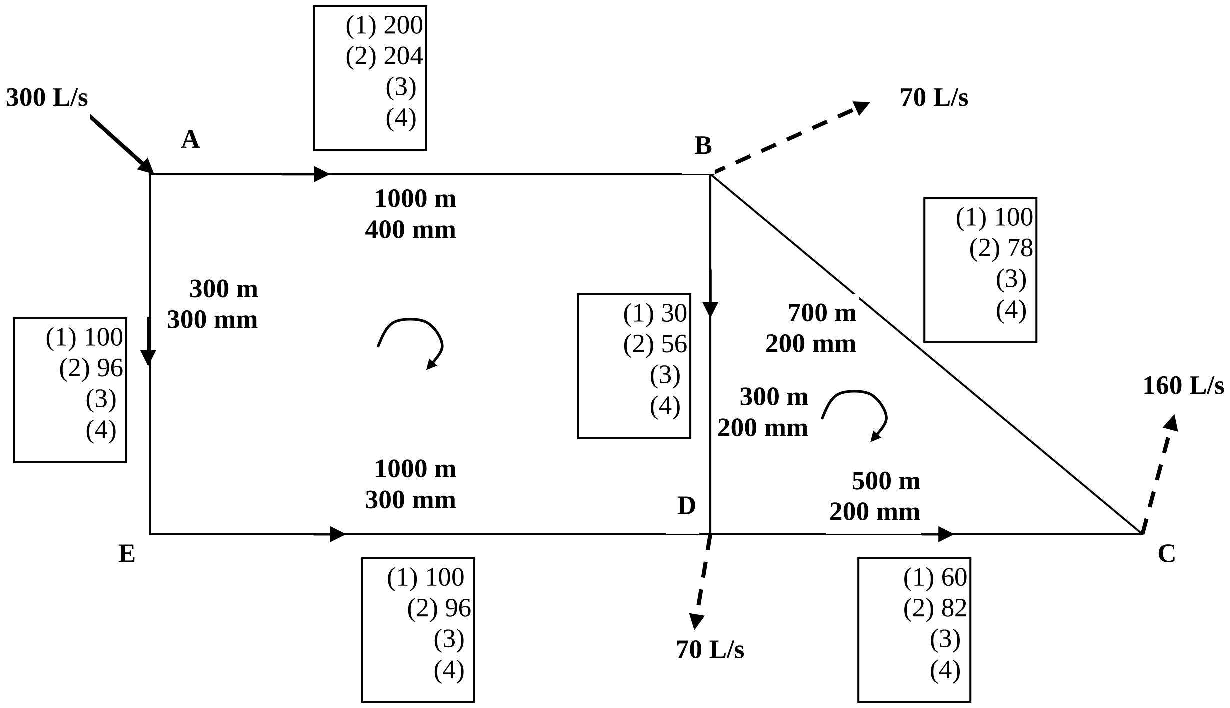
1st Trial

Line	**Q (L/s)**	**D (mm)**	**L (m)**	**S = h_L/L**	**h_L**	**h_L/Q**	**Modified Q**
AB	200	400	1000	0.00942	9.42	0.0471	204
BD	30	200	300	0.00082	2.47	0.0823	34
DE	-100	300	100	0.0106	-10.6	0.106	-96
EA	-100	300	300	0.0106	-3.18	0.038	-96
				∑	*-1.89*	*0.2672*	

ΔQ = - (-1.89)/[1.85 x 0.2672] = 3.8 = 4

Line	**Q (L/s)**	**D (mm)**	**L (m)**	**S = h_L/L**	**h_L**	**h_L/Q**	**Modified Q**
BC	100	200	700	0.076	53.51	0.5351	78
CD	-60	200	500	0.030	-15	0.25	-82
DB	-34	200	300	0.0103	-3.12	0.092	-56
				∑	*35.39*	*0.8771*	

ΔQ = - (35.39)/[1.85 x 0.8771] = - 21.8 = - 22



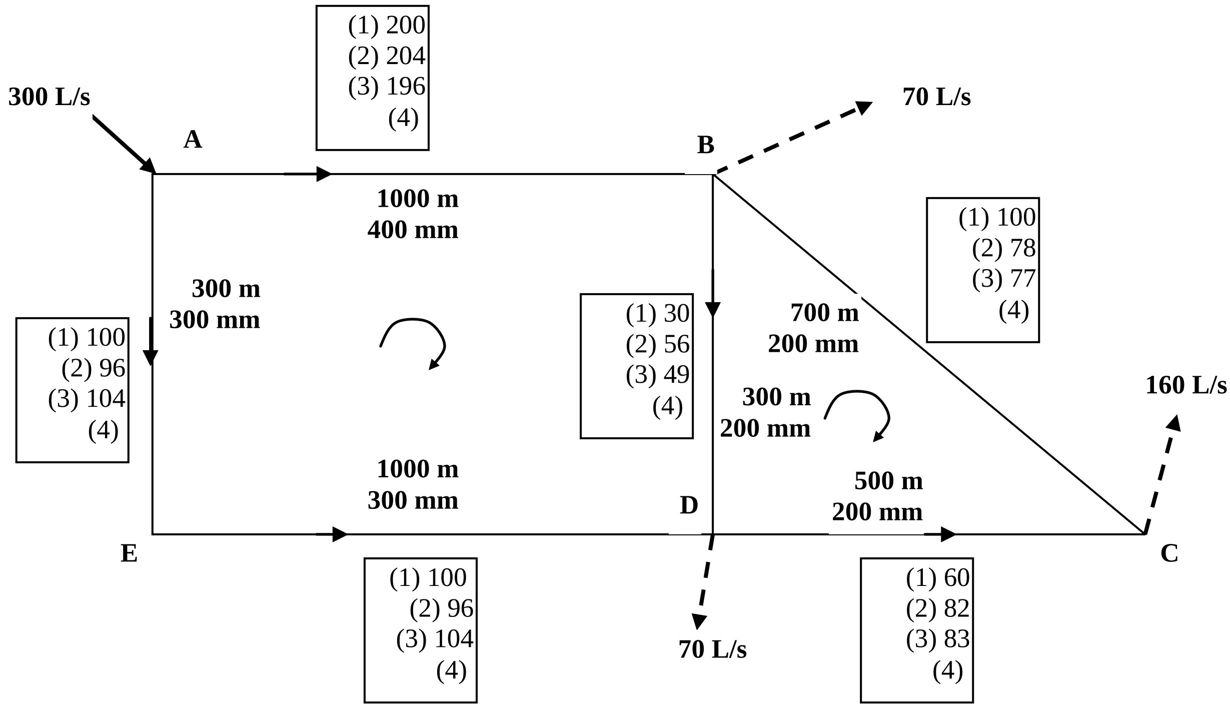
2nd Trial

Line	**Q (L/s)**	**D (mm)**	**L (m)**	**S = h_L/L**	**h_L**	**h_L/Q**	**Modified Q**
AB	204	400	1000	0.0098	9.8	0.048	196
BD	56	200	300	0.0262	7.85	0.14	48
DE	-96	300	100	0.0098	-9.84	0.1025	-104
EA	-96	300	300	0.0098	-2.95	0.0307	-104
				∑	*4.86*	*0.3212*	

ΔQ = - (4.86)/[1.85 x 0.3212] = - 8.2 = - 8

Line	**Q (L/s)**	**D (mm)**	**L (m)**	**S = h_L/L**	**h_L**	**h_L/Q**	**Modified Q**
BC	78	200	700	0.0483	33.8	0.433	77
CD	-82	200	500	0.053	-26.5	0.323	-83
DB	-48	200	300	0.0197	-5.9	0.123	-49
				∑	*1.4*	*0.879*	

ΔQ = - (1.4)/[1.85 x 0.879] = - 0.86 = - 1



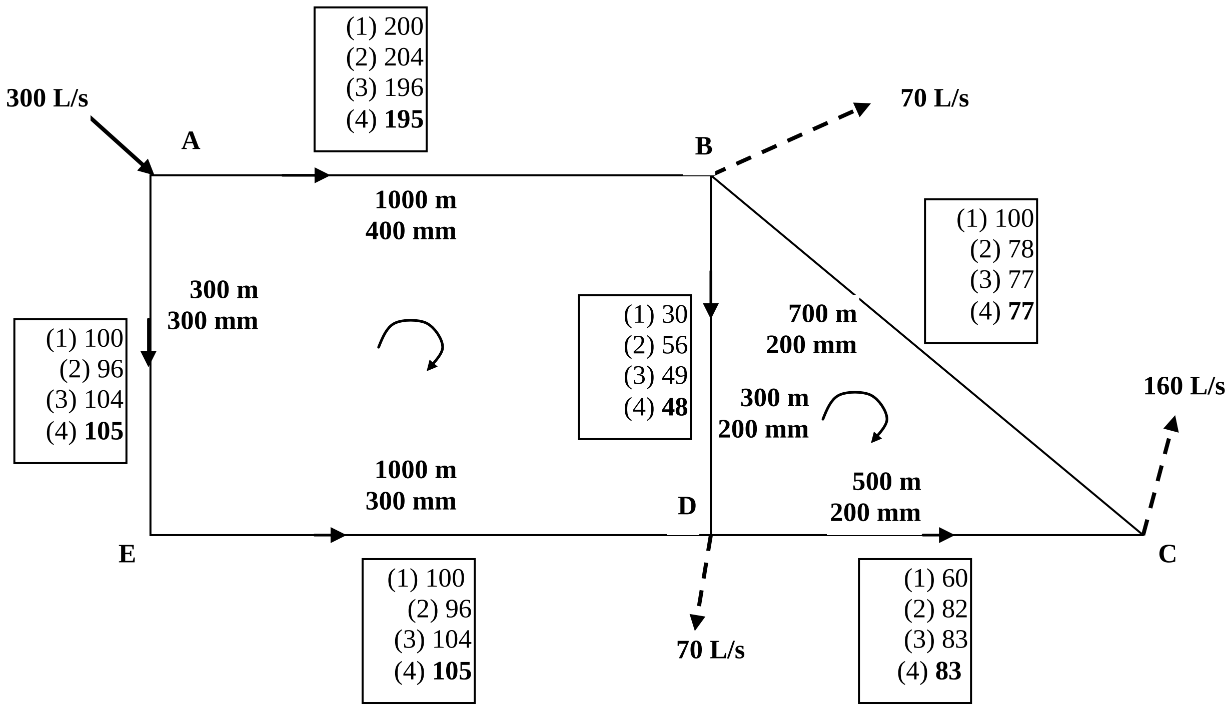
3rd Trial

Line	**Q (L/s)**	**D (mm)**	**L (m)**	**S = h_L/L**	**h_L**	**h_L/Q**	**Modified Q**
AB	196	400	1000	0.0091	9.1	0.046	195
BD	49	200	300	0.02	6.13	0.125	48
DE	-104	300	100	0.0114	-11.41	0.11	-105
EA	-104	300	300	0.0114	-3.42	0.033	-105
				∑	*0.4*	*0.314*	

$\Delta Q = -(0.4)/[1.85 \text{ x } 0.314] = -0.69 = -1$

Line	**Q (L/s)**	**D (mm)**	**L (m)**	**S = h_L/L**	**h_L**	**h_L/Q**	**Modified Q**
BC	77	200	700	0.0417	33	0.43	77
CD	-83	200	500	0.0542	-27.1	0.33	-83
DB	-48	200	300	0.0197	-5.9	0.123	-48
				∑	*0*	*0.883*	

$\Delta Q = -(10/[1.85 \text{ x } 0.883] = 0$



Notes on Water distribution Systems

- Water pipes are normally installed within the right-of-way of the streets.
- Soil cover: 0.75 m – 2.4 m depending on climate.
- Trench width:
  - Clearance of about 30 cm on either side of the pipe (i.e. for a pipe of 1220 mm diameter, the trench width = 1.220 + 0.60 = 1.82 m). Extra clearance should be provided at joints and fittings
- Backfill materials should be free from rocks, and refuse.

Pumps and Pumping Stations

- **Pumps are used for:**
  - Pumping water from source to treatment plants.
  - Pumping from reservoirs at water treatment plants to distribution systems.
  - Pumping water within distribution systems.
  - Pumping within water and wastewater treatment plants.
  - Pumping wastewater in certain wastewater collection systems.

- **Types of Pumps**

All pumps are classified as:

(1) Kinetic–energy pumps (turbo-machine).

- Centrifugal pumps (most widely used pumps in water & wastewater applications):
  - Radial-flow pumps
  - Axial-flow pumps
  - Mixed flow pumps
- Others

(2) Positive displacement pumps.

- Reciprocating pumps (piston or diaphragm pumps)
- Rotary pumps (e.g. screw pumps)
- Others

Centrifugal Pumps

- *As the head (H) increases, then the discharge (Q) decreases.*
- *Principle Components of Centrifugal Pumps*

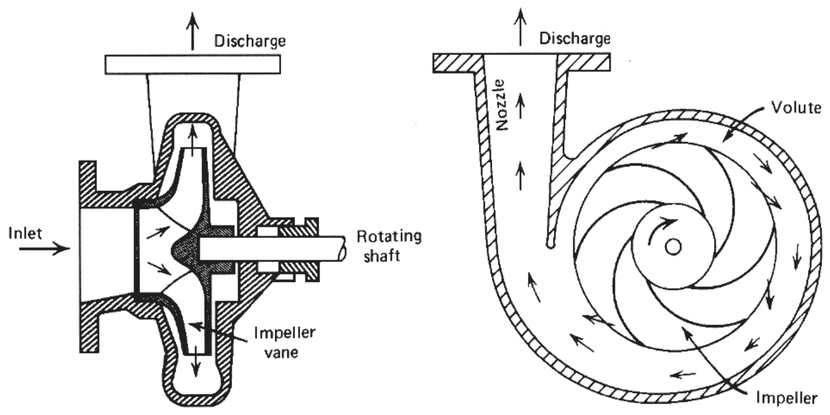
(1) The rotating element (the impeller): It forces the liquid being pumped into a rotary motion.

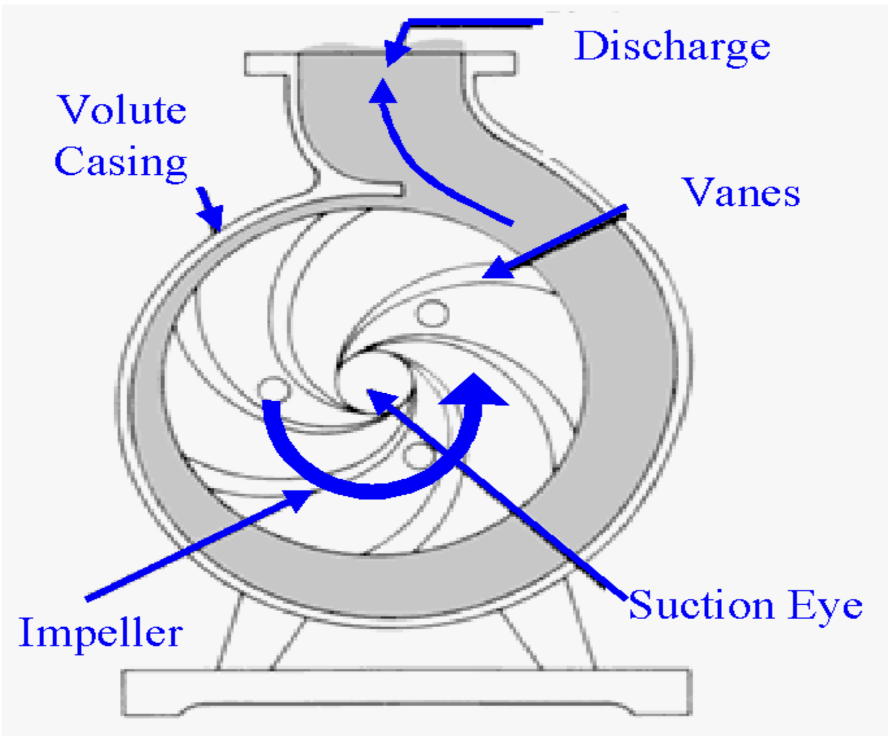
(2) The shaft on which the impeller is mounted: it could be horizontal or vertical

(3) The pump casing: to direct the liquid to the impeller and lead it away.

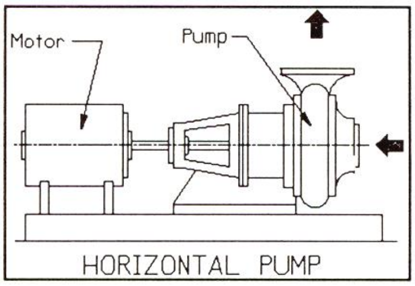
(4) The frame: to support the pump casing.

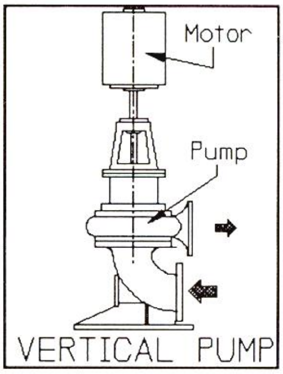
Figure 4-9 Cross-sectional diagrams showing the features of a centrifugal pump.





Liquid flow path inside a centrifugal pump





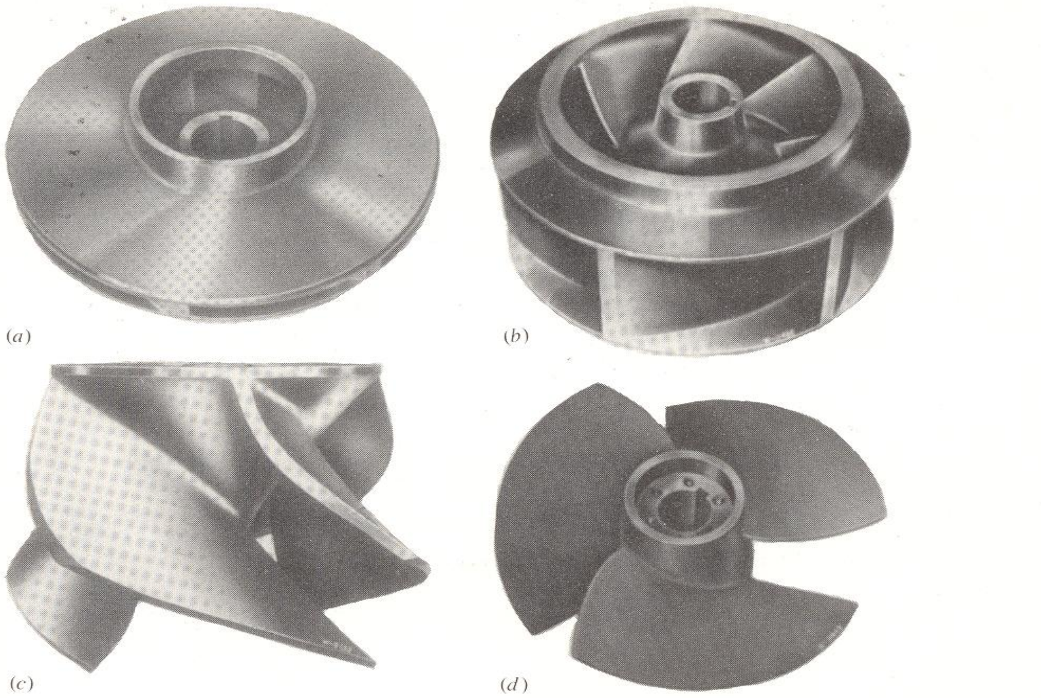


Figure 8-6 Typical impellers used in centrifugal pumps. (*a*) Closed single-suction, (*b*) Closed Francis-vane double-suction, (*c*) Open mixed-flow, (*d*) Axial-flow (propeller). (*Courtesy Worthington Pump, Inc.*)

Screw Pumps

- The pump capacity (Q) is independent of the head.
- Pump capacity (Q) depends on the depth of the liquid entering the screw; the lower the level, the less the discharge
- Screw pumps are sort of automatic variable speed pumps.

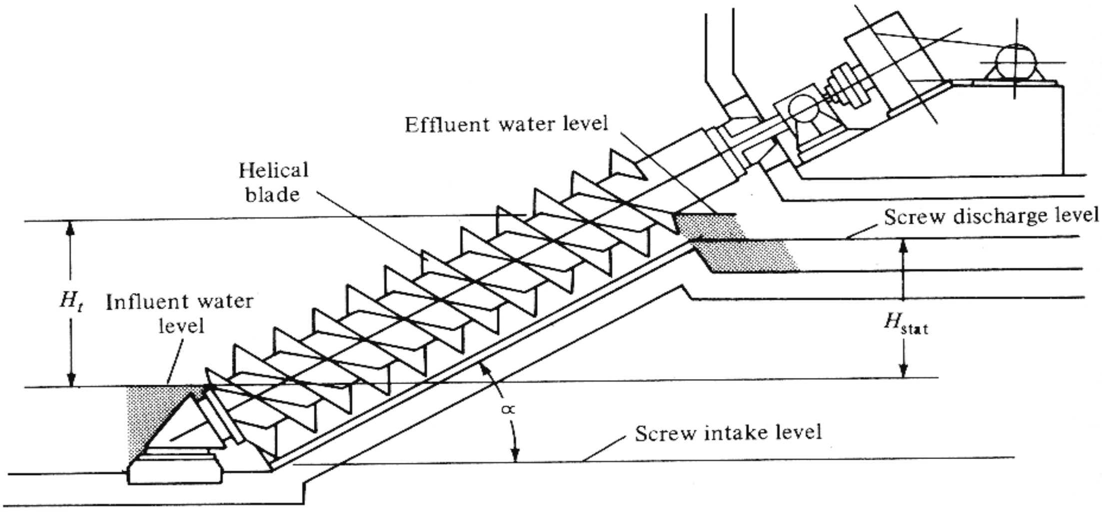


Figure 8-12 Cross section of typical screw pump.

- Advantage of screw pumps over centrifugal pumps
  - They can pump water containing large solids without clogging.
  - They operate at constant speed over a wide range of flows with relatively good efficiency.

Pump Drive units

- Electric motors: constant-speed motors and variable-speed motors
- Internal-combustion engines (usually as a source of standby power): diesel engines, natural gas engines.

Pumps Application terminology

- **Capacity (Q):** volume of water pumped per unit of time [m^3/s, Mgal/d, gpm].

- **Head (H):** the head against which pump must works when water is being pumped, [m, ft].

- **Power**: the power the pump consumes when it pumps up [KN.m/s = Kilowatt = KW].

Output Power, $p_{out} = P\ Q$ where P = pressure the pump can deliver

Output Power, $p_{out} = \gamma H Q$

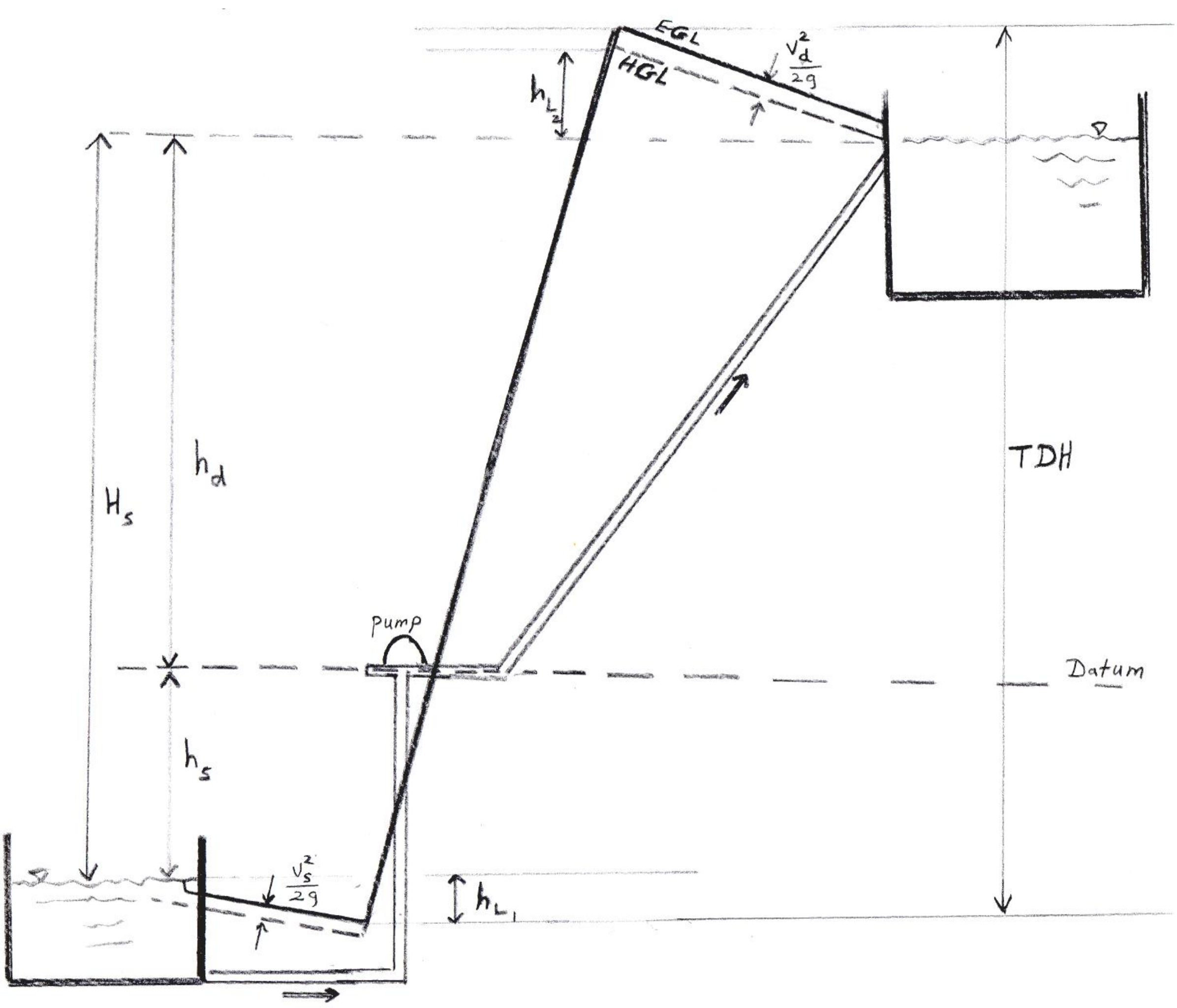
Input Power, p_{in} = Output Power, p_{out} / Efficiency, e_p

$p_{in} = p_{out} / e_p$

(i.e. Power consumed = Power Delivered / Efficiency)

Notes:

- An efficient pump is one that consumes little as compared to what it delivers.
- e_p range from 60% to 85% due to energy losses in the pump.



h_L = head losses = friction losses + minor losses = system head

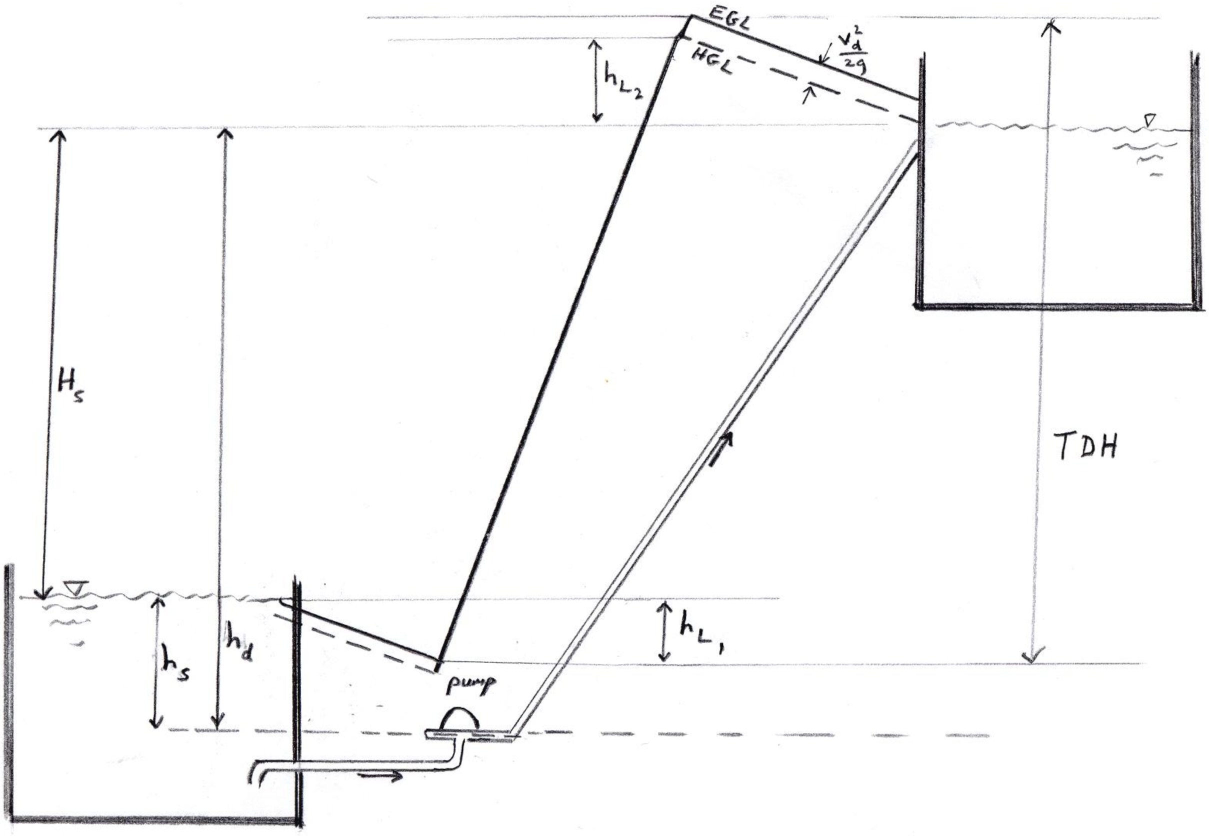
h_s = static suction lift (between pump datum and suction water level)

h_d = static discharge head (between pump datum and discharge water level)

H_s = static head (between the two water levels) = $h_s + h_d$

TDH = total dynamic head = the head against which the pump must work when water is being pumped.

TDH = $H_s + h_{L1} + h_{L2} + (V_d^2/2g)$



h_L = head losses = friction losses + minor losses = system head

h_s = static suction head (between pump datum and suction water level)

h_d = static discharge head (between pump datum and discharge water level)

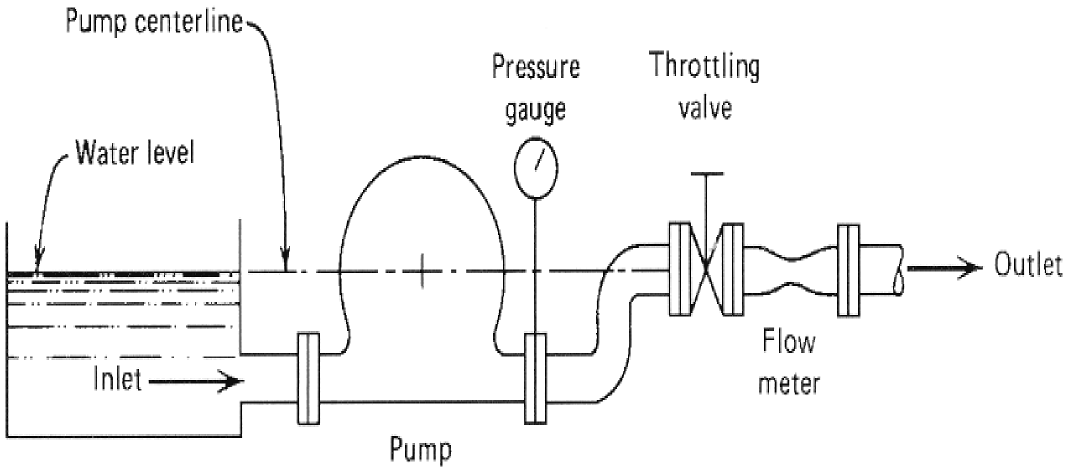
H_s = static head (between the two water levels) = $h_d - h_s$

TDH = total dynamic head = the head against which the pump must work when water is being pumped.

TDH = $H_s + h_{L1} + h_{L2} + (V_d^2/2g)$

Pump Head-Capacity Curve (Pump Characteristics Curve)

- The head (TDH) that a pump can produce at various flowrates (Q) and constant speed is established in pump tests conducted by the pump manufacturer.



- During the tests, the capacity is varied and the corresponding head is measured. Also, the efficiency and the input power are measured.

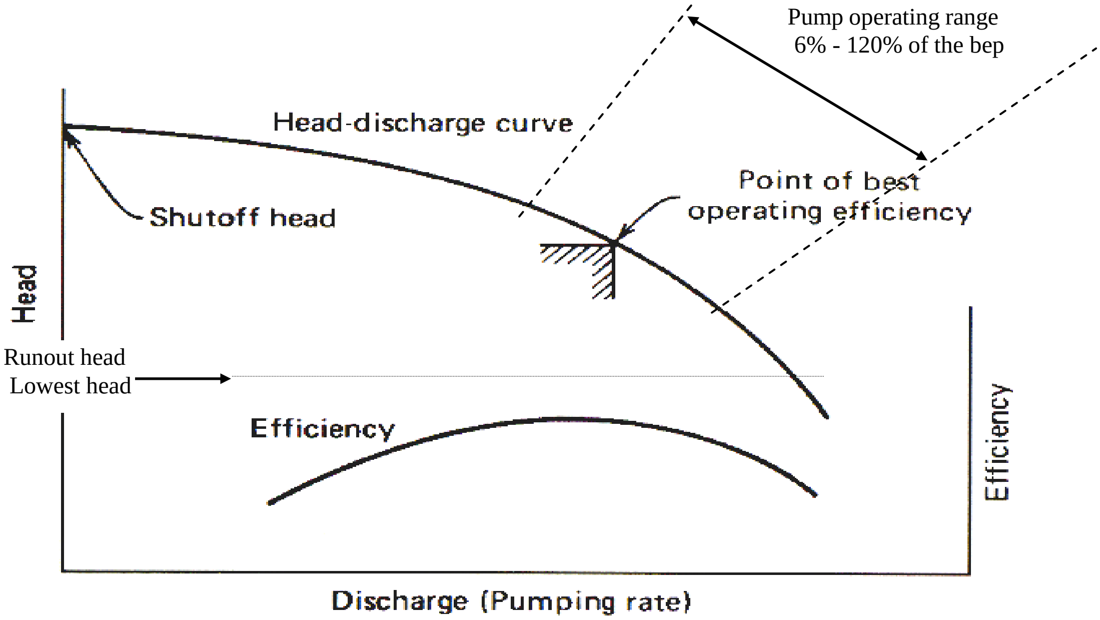
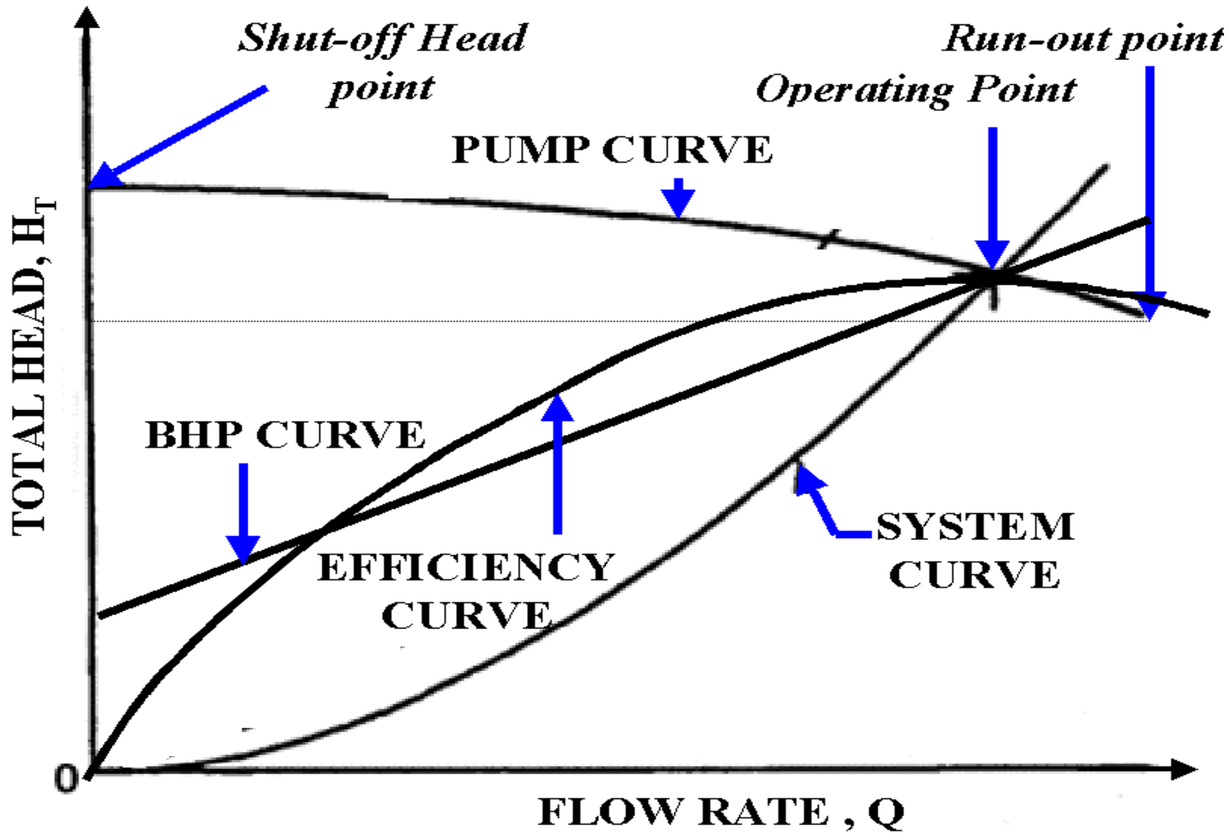


Figure 4-11 Characteristic curves for a centrifugal pump operating at a constant speed.

- At maximum efficiency, the discharge is known as the normal or rated discharge of the pump, and the corresponding head is known as the rated head. The corresponding point on the head-capacity curve is known as the best operating point (bep).



System Head-Capacity Curve

- To develop a system head-capacity curve for your system, calculate the headloss at different Q, add the headloss to the static head to get TDH or H, and plot TDH against Q.

Note: your system: the arrangement of pipes, fittings, and equipment through which the water will flow

- TDH = H_s + friction and minor losses + velocity head (small)

  TDH = $H_s + h_L$ (function of Q)

- If we plot the pump H-Q curve, it will intersect with the system H-Q curve, and the point of intersection is the point at which the pump will operate

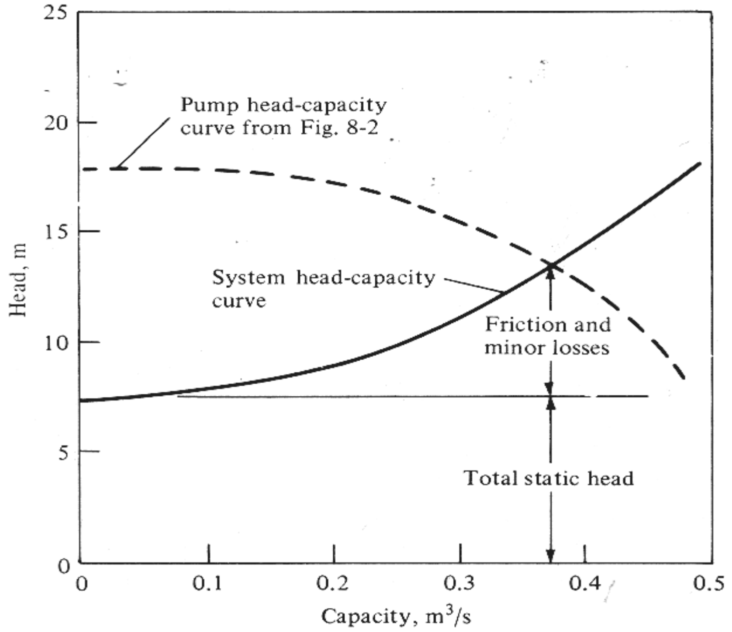
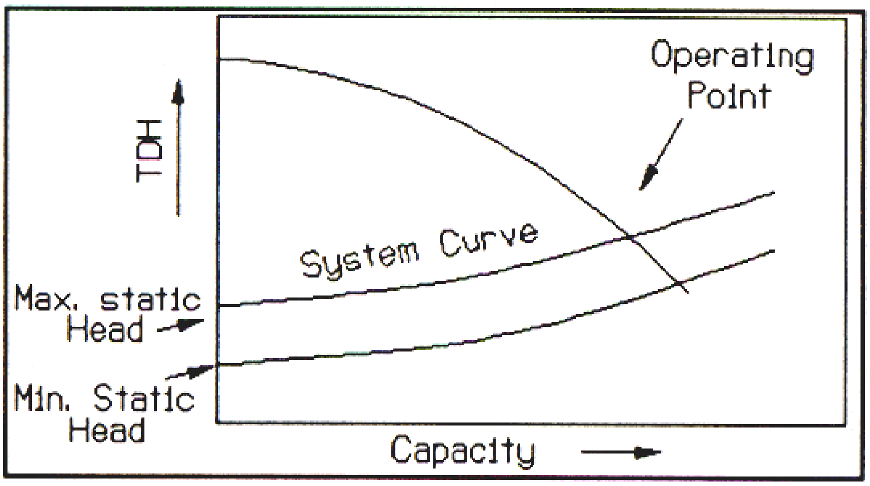


Figure 8-3 System head-capacity curve for typical pump installation.

- How to select a pump that matches our conditions of our system:
  - Mark the H-Q point at which we desire our pump to operate on the system H-Q curve.
  - Inspect several possible pump curves (representing several pumps) and superimpose each onto our system curve, and the select the pump that matches our desired point (at or near the desired point).
  - Note that the static head may vary depending on tank drawdowns and filling. Thus, both maximum and minimum static heads should be used to develop the system curve. The distance between them will be the actual operating range of the pump you choose.



Flow, Head, and Power Relationships for Pumps

- Pump discharge changes with impeller diameter and operating speed.
- For a given impeller diameter (D is constant) operating at two or more speeds, we have:

$Q \propto N$ --> $Q_1/Q_2 = N_1/N_2$ --> $Q_2 = Q_1 (N_2/N_1)$

$H \propto N^2$ --> $H_1/H_2 = N_1^2/N_2^2$ --> $H_2 = H_1 (N_2/N_1)^2$

$p \propto N^3$ --> $p_1/p_2 = N_1^3/N_2^3$ --> $p_2 = p_1 (N_2/N_1)^3$

Q = discharge

H = Head

p = power input

N = pump speed (rpm)

Note: If $N_2 = 2 N_1$ then $Q_2 = 2 Q_1$, $H_2 = 4 H_1$, and $p_2 = 8 p_1$

(Doubling the speed will increase the discharge by a factor of 2, the head by a factor of 4, and the power by a factor of 8)

- For a pump operating at the same speed (N is constant), a change in impeller diameter (D) affects discharge, head, and power input as follows:

$Q \propto D$ --> $Q_1/Q_2 = D_1/D_2$ ---> $Q_2 = Q_1 (D_2/D_1)$

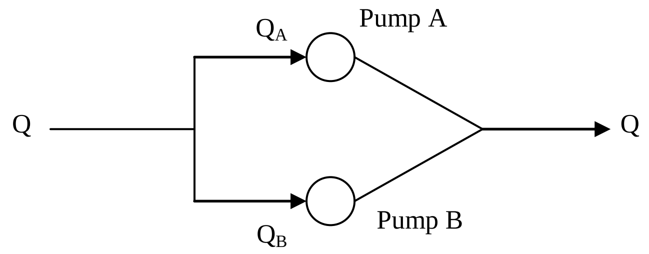
$H \propto D^2$ --> $H_1/H_2 = D_1^2/D_2^2$ ---> $H_2 = H_1 (D_2/D_1)^2$

$p \propto D^3$ --> $p_1/p_2 = D_1^3/D_2^3$ ---> $p_2 = p_1 (D_2/D_1)^3$

Note: If $D_2 = 2 D_1$, then $Q_2 = 2 Q_1$, $H_2 = 4 H_1$, and $p_2 = 8 p_1$

Multiple-Pump Operation (for constant speed pumps)

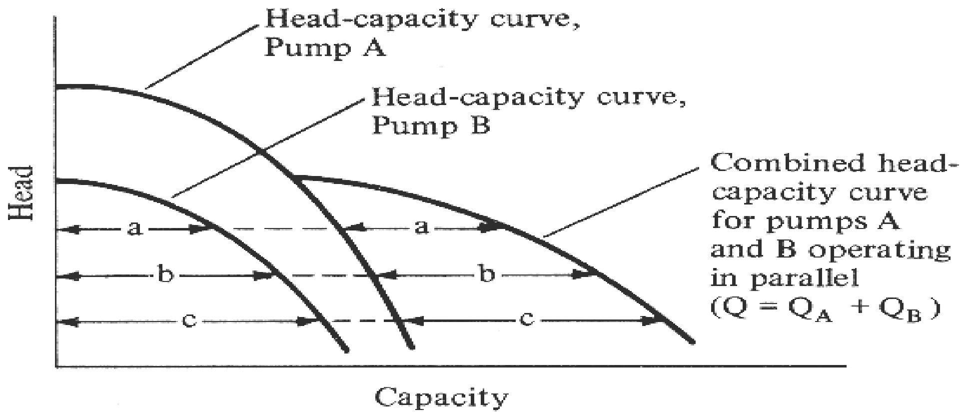
- Parallel Operation
    - For two or more pumps operating in parallel, the combined H-Q curve is found by adding the Qs of the pumps at the same head.
        - For two pumps:



- $Q = Q_A + Q_B$ at a given head
- Power input, $p = p_A + p_B$

$$\gamma\, Q\, H/e \;=\; \gamma\, Q_A\, H_A/e_A \;+\; \gamma\, Q_B\, H_B/e_B$$

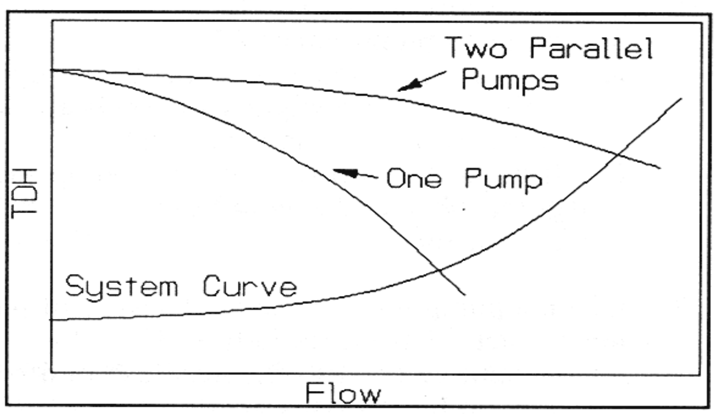
$$Q\, H/e \;=\; Q_A\, H_A/e_A \;+\; Q_B\, H_B/e_B$$



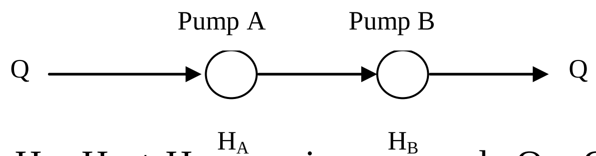
A. Parallel operation

If the two pumps are identical, $H = H_A = H_B$ *and* $Q_A = Q_B$

$Q = 2\,Q_A = 2\,Q_B$ & $Q/e = Q_A/e_A + Q_B/e_B$



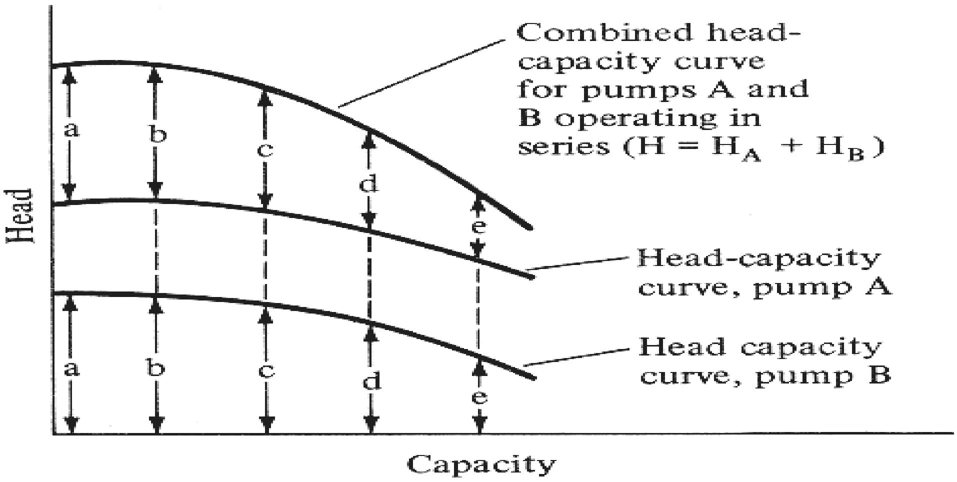
- Series Operation
  - For two or more pumps operating in series, the combined H-Q curve is found by adding the Hs of the pumps at the same Q.
    - For two pumps:



- $H = H_A + H_B$ at a given Q and $Q = Q_A = Q_B$
- Power input, $p = p_A + p_B$

$$\gamma Q H/e = \gamma Q_A H_A/e_A + \gamma Q_B H_B/e_B$$

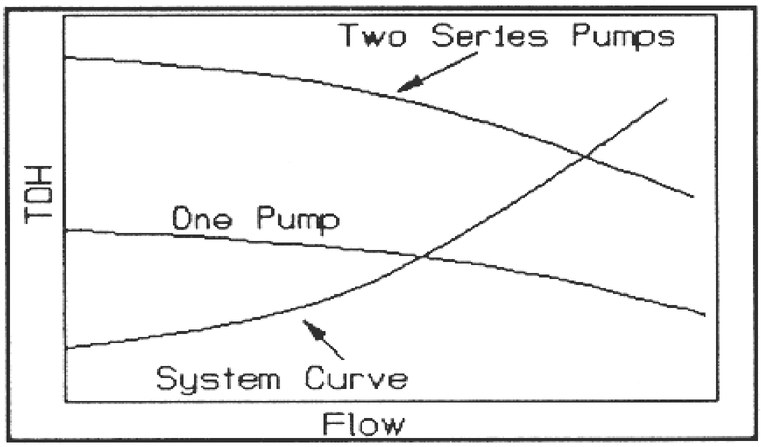
$$H/e = H_A/e_A + H_B/e_B$$



B. Series operation

If the pumps are identical, $H_A = H_B$, *and* $Q = Q_A = Q_B$

$H = 2 H_A = 2 H_B$ & $H/e = H_A/e_1 + H_B/e_2$

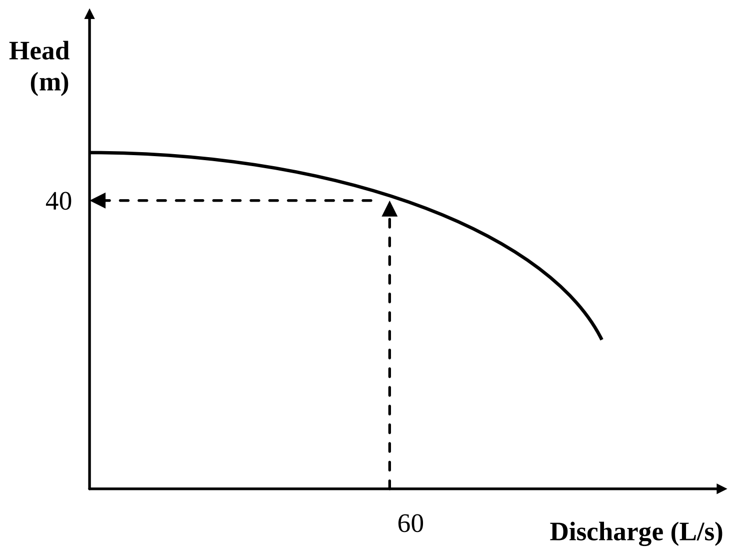


Example

A pump with a head-capacity characteristic curve shown below, is used to transfer 60 L/s of water through a 1000-m long pipe from a reservoir at an elevation of 105 m to another reservoir at an elevation of 130 m

(1) Find the diameter of the pipe (assume that C = 100 and neglect minor losses)

(2) Determine the pump power required assuming that the pump efficiency is 80%.



Solution

At Q = 60 L/s → TDH = 40 m

TDH = $H_s + h_f$ = (130 – 105) + h_f

TDH = 25 + h_f

But $h_f = [10.7 \times L \times Q^{1.85}] / [C^{1.85} \times d^{4.87}]$

$h_f = [10.7 \times 1000 \times 0.06^{1.85}] / [100^{1.85} \times d^{4.87}]$

$d^{4.87} = 0.000781$ → d = 0.230 m = 230 mm

Power input = γ Q H/e = [9.8 KN/m^3 x 0.06 m^3/s x 40 m] / 0.80 = 29.4 KW

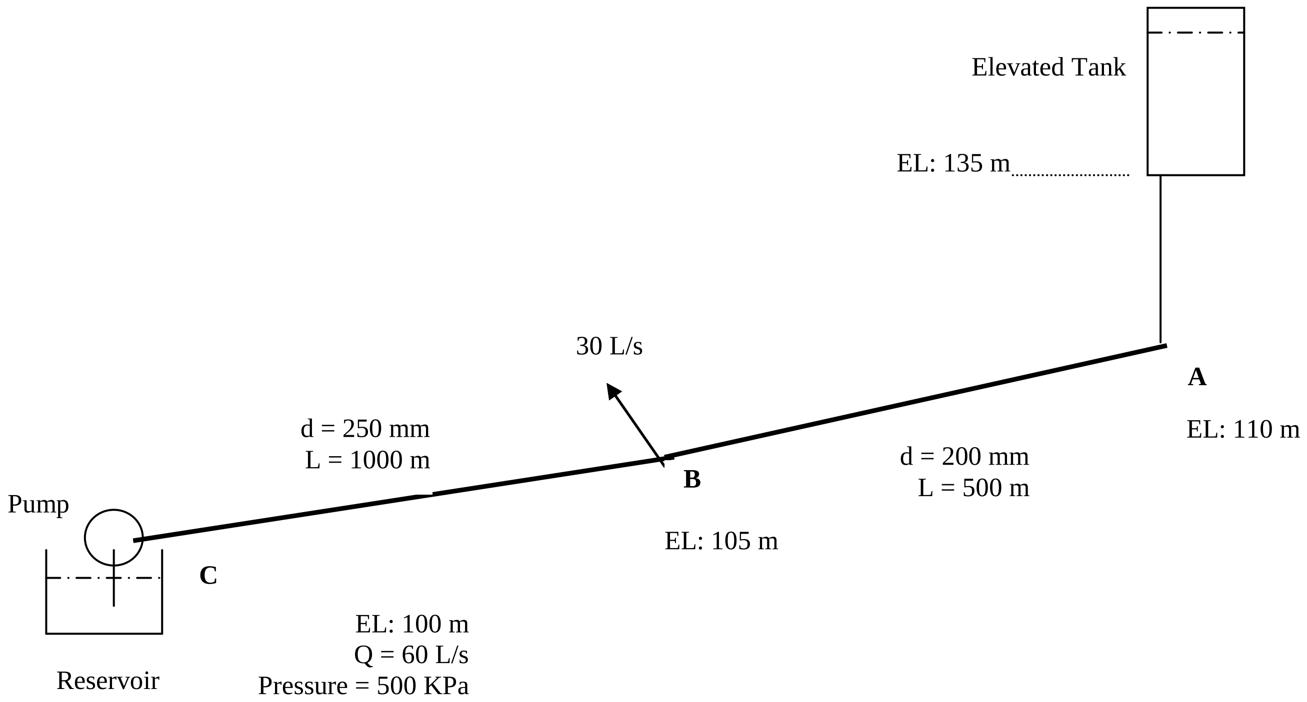
Example

At night, water is pumped from a treatment plant reservoir through distribution piping to an elevated storage tank as shown below. The pumping station operating at a flow capacity of 60 L/s and discharge pressure of 500 KPa. Determine:

(a) the pressure at A and B

(b) the maximum depth of water that can be stored in the elevated tank.

[Assume C = 100 and neglect minor losses and velocity head]



Solution

Pressure available at C = 500 KPa = 500 KPa / 9.8 KPa/m = 51 m

Pressure at B = 51 – (105 – 100) – h_{fCB}

From Hazen-Williams nomograph, at Q = 60 L/s and d = 250 mm

then S = 9 m / 1000 m

S = h_f/L --> h_f = (9/1000) x 1000 m = 9 m

Pressure at B = 51 – 5 – 9 = 37 m [or 37 x 9.8 = 363 KPa]

Pressure at A = 51 – (110 – 100) – h_{fCB} – h_{fBA} = 51 – 10 – 9 – h_{fBA} = 32 – h_{fBA}

From Hazen-Williams nomograph, at Q = 30 L/s and d = 200 mm

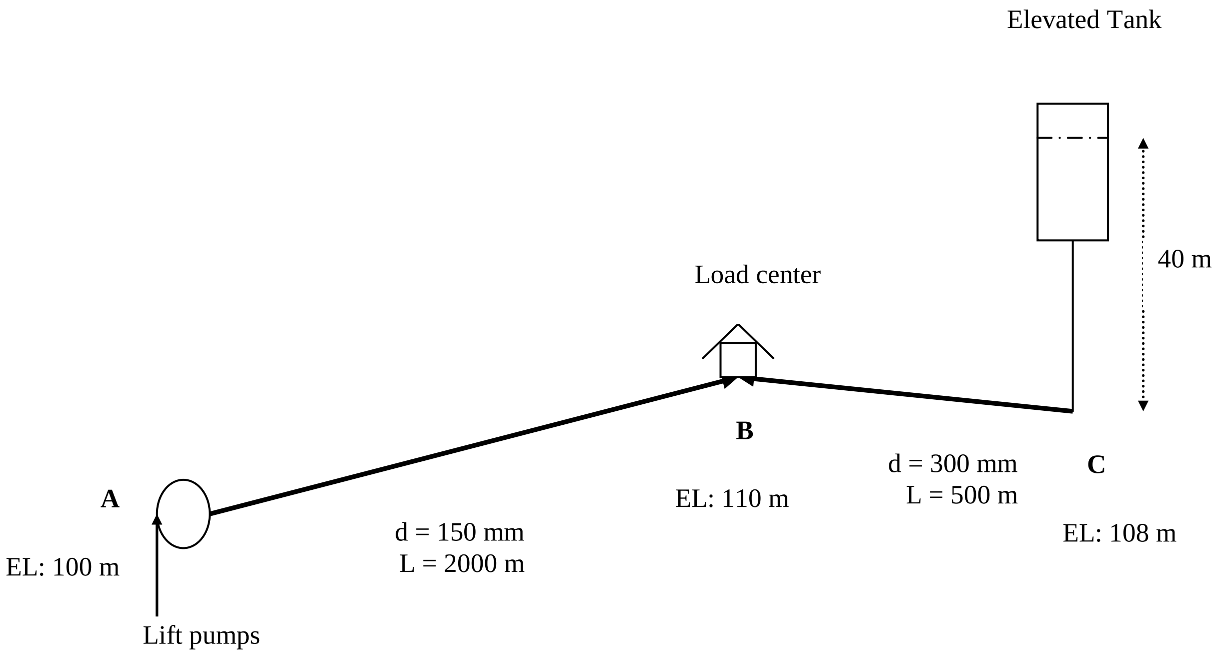
$S = 8\ m / 1000\ m$, then $h_{fBA} = (8/1000) \times 500\ m = 4\ m$

Pressure at A = 32 – 4 = 28 m [or 28 x 9.8 = 274 KPa]

Maximum depth of water in the tank = (110 + 28) - 135 = 3 m

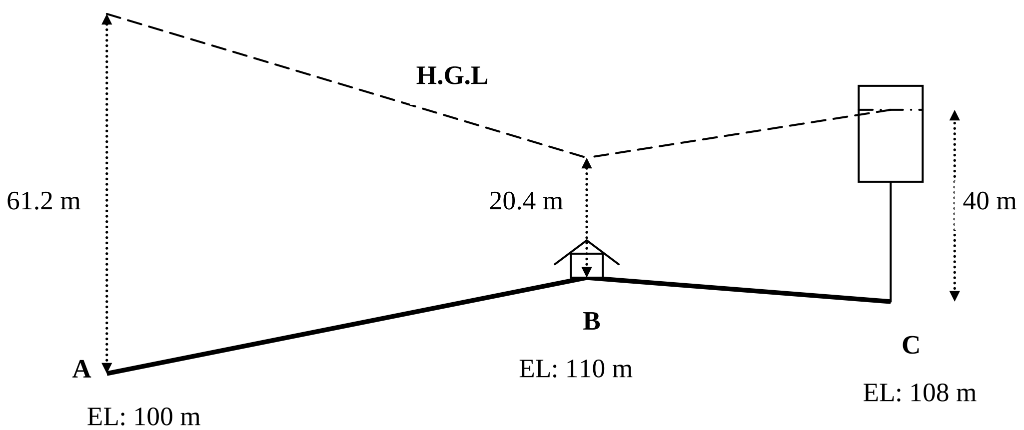
Example

A simplified water supply system consisting of a reservoir with lift pumps, an elevated storage, piping, and a load center (i.e. withdrawal point), is shown in the Figure below. Draw the hydraulic gradient for the system and calculate the quantity of flow available at point B from both the reservoir and the supply pumps if the pumps provide a pressure of 600 KPa, and the residual pressure at the load center is 200 KPa. [Assume C = 100 and neglect minor losses and velocity head]



Pressure at A = 600 KPa = 600 KPa / 9.8 KPa/m = 61.2 m

Pressure at B = 200 KPa = 200 KPa / 9.8 KPa/m = 20.4 m



h_{fAB}:
Pressure at B = Pressure at A – Elevation difference – h_{fAB}

20.4 = 61.2 – (110 – 100) – h_{fAB}

20.4 = 61.2 – 10 – h_{fAB}

20.4 = 51.2 – h_{fAB}

h_{fAB} = 51.2 – 20.4 = 30.8 m ---> S = h_f/L = 30.8 m / 2000 m = 15.4 m / 1000 m

From nomograph at S = 15.4/1000, and d = 150 mm ----> Q_{AB} = 20 L/s

h_{fCB}:

Pressure at B = Pressure at C – Elevation difference – h_{fCB}

20.4 = 40 – (110 –108) – h_{fCB}

20.4 = 40 – 2 – h_{fCB}

20.4 = 38 – h_{fCB}

h_{fCB} = 38 – 20.4 = 17.6 m ---> S = 17.6 m / 500 m = 35.2 m / 1000 m

From nomograph at S = 35.2/1000 and d = 300 mm ---> Q_{CB} = 200 L/s

Total Q available at B = 20 + 200 = 220 L/s

Cavitation

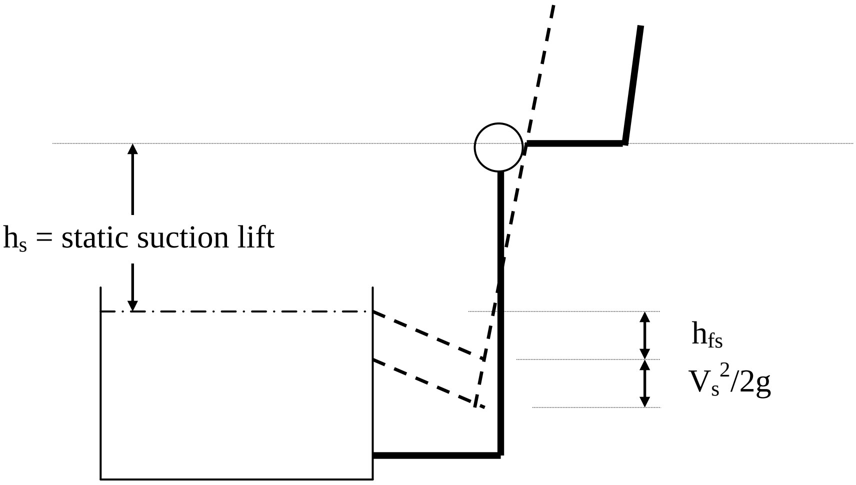
- When a liquid flows into a region where its pressure is reduced to vapor pressure of the liquid, the liquid boils and vapor pockets develop in it. The vapor bubbles are carried along with the liquid until a region of higher pressure is reached, where the bubbles suddenly collapse and the liquid rushes to fill the voids creating very high localized pressures that cause pitting of the surrounding surfaces.

- It occurs in pumps when the absolute pressure of the pump inlet drops below the vapor pressure of the water being pumped. The bubbles are formed and when the water reaches the impeller, the bubbles collapse.

- Damages caused by cavitation:
  - Reduce pump efficiency and capacity.
  - Damage the impeller (pitting).
  - Damage the valves and fittings.
  - Cause shaft deflection and break.

- How to determine if cavitation is a problem?

  - If the available “net positive suction head” ($NPSH_A$) which is the head available in the system at the eye of the impeller to push the water into the pump to replace water discharge by the pump, is more than “net positive suction head” required by the pump ($NPSH_R$) to prevent cavitation, then no cavitation will occur.
  - $NPSH_R$ is a characteristic of the pump (determined by the manufacturer).
  - $NPSH_A$ is a characteristic of the system.



$NPSH_A = [-h_s - h_{fs} - V_s^2/2g] + [p_{atm}/\gamma - p_{vapor}/\gamma]$

$NPSH_A = [-h_s - h_{fs}] + [p_{atm}/\gamma - p_{vapor}/\gamma]$

$V_s^2/2g$ cancels out because it is also included in the $NPSH_R$.

Example:

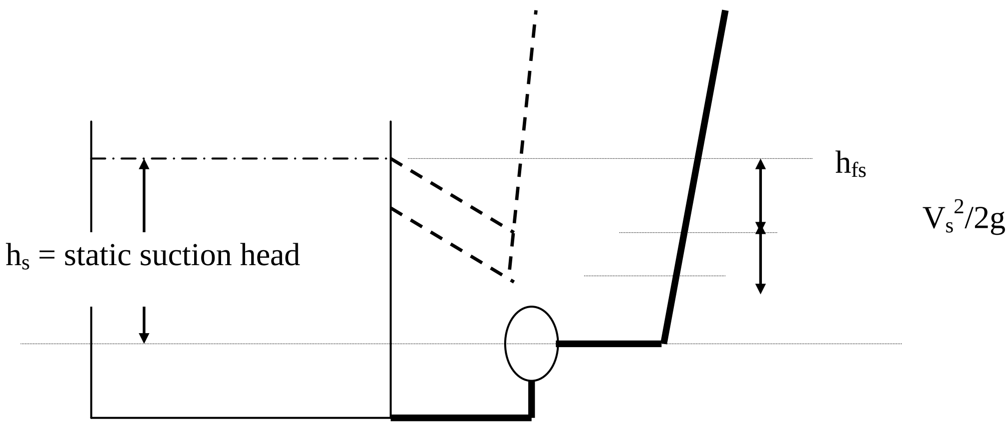
If $h_s = 2$ m and $h_{fs} = 0.5$ m

Water temperature = 20 ^oC → p_{vapor} = 2.34 KPa = 2.34 KN/m^2

p_{atm} = 101.325 KPa. Determine the $NPSH_A$.

$NPSH_A = [-h_s - h_{fs}] + [p_{atm}/\gamma - p_{vapor}/\gamma]$

$NPSH_A = [-2 - 0.5] + [101.325/9.8 - 2.34/9.8] = 7.59$ m



$NPSH_A = [+h_s - h_{fs} - V_s^2/2g] + [p_{atm}/\gamma - p_{vapor}/\gamma]$

$NPSH_A = [+h_s - h_{fs}] + [p_{atm}/\gamma - p_{vapor}/\gamma]$

- Causes of Cavitation;
  - Operating the pump at high speeds (impeller runs too fast for water to keep up).
  - Operating the pump at a capacity greater than bep (toward the runout head). This will increase the losses.
  - Operating the pump at a capacity lower than the bep (toward the shutoff head). This re-circulates the water within the impeller causing vibration and hydraulic losses.

  For these reasons, operating a pump at a rate of discharge within a range between 60% and 120% of the bep is a good practice.
  ($Q = 0.6\ Q_{bep}$ to $1.2\ Q_{bep}$)

  - Excessive suction lift.
  - Excessive friction and minor losses in suction piping.
  - High temperature of water.
  - Insufficient submersion of the suction pipe in the suction tank (air will draw right in with water).
  - Entrained air (outside air coming into the system on the suction side through an opening, suction valves and joints).
  - The discharge head is too high and the pump will operate outside the operating range.

Water Pipes

- Selection of pipe type depends on:
  - Characteristics of water
  - Internal water pressure
  - External pressure / loads
  - First cost and maintenance cost
  - Pipe durability (resistance to corrosion, bending and tensile stress)

Types of Pipes

- Iron Pipe
  - Durable and has a service life of > 100 years.
  - Cast iron and ductile iron.
  - Ductile iron is produced by adding magnesium alloy to an iron of very low phosphorus and sulfur content.
  - Ductile iron is lighter, less brittle, and more stronger than cast iron.
  - Subject to corrosion: the inside of the pipe may be coated with scales of rust (tuberculation), reducing the pipe diameter and increasing its roughness.
  - Therefore, it is common to line iron pipes with cement or bituminous material.
  - Pipes can also be encased in polyethylene tubes to protect against external corrosion.
  - Size: 2 to 79 inch (50 – 2000 mm) with several thickness classes in each size.
  - Length: up to 5.5 m.

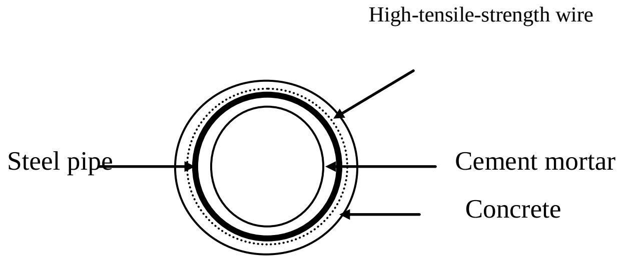
- Steel Pipe
  - Made of iron with small amount of carbon added to it (0.15% – 1.5%).

- Stronger, lighter, and cheaper than iron pipes.
- More likely to be damaged by corrosion than iron because of its thin walls (needs coating with cement or bituminous material)
- Service life: about 50 years.
- Usually used in transmission lines.

- Concrete Pipe
  - Three types

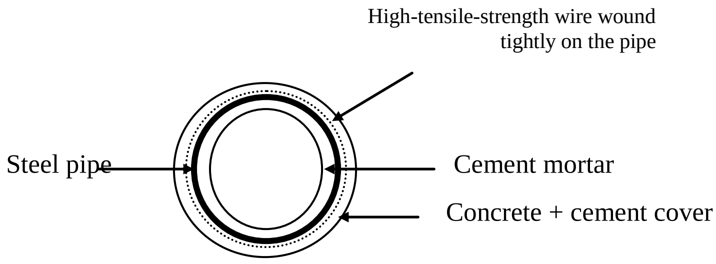
    (1) Concrete cylinder pipe

    - *High-tensile-strength wire is wrapped about a steel pipe and then covered with a layer of concrete. The inside of the pipe is lined with cement.*
    - *Used for high internal pressure (up to 1800 KPa)*



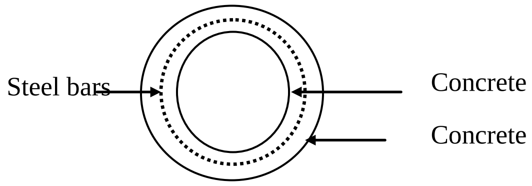
(2) Pre-stressed concrete cylinder pipe

- *The steel pipe is pre-stressed by winding the wire tightly to pre-stress the pipe that is covered with concrete. The exterior is then finished with a coating of cement covered.*
- *Used for higher internal pressure (up to 2400 KPa).*



(2) Reinforced concrete pipe (without steel cylinder)

- *Used for low pressure ($\ngtr$ 310 KPa)*



- Properties of concrete pipes:
    - High durability and low maintenance cost.
    - No tuberculation.
    - Smooth pipes
    - Service life ≈ 75 years.

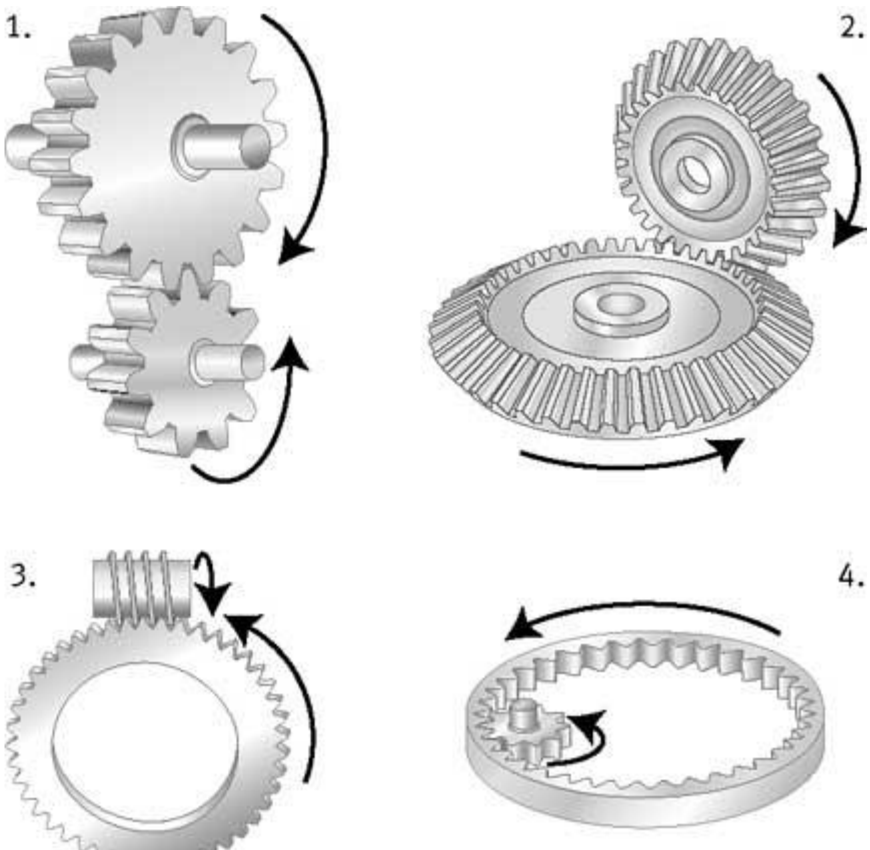
- Asbestos Cement Pipe
    - Composed of a mixture of asbestos fiber, cement, and silica sand. Pipes are formed from this mixture on a rotating steel mandrel and then compacted with steel pressure rollers, heat-dried and cured.
    - Smooth pipes.
    - Light and cheap.
    - Resist corrosion.
    - Do not conduct electricity.
    - Not as strong as cast iron pipes.
    - Asbestos is carcinogenic when fibers are inhaled.

- Plastic Pipe (Thermoplastic Pipe)
    - Types:
        - Polyvinyl chloride (PVC).
        - Polyethylene (PE).
        - Polybutylene (PB).

        ▪ Acrylonitrile-Butadiene-Styrene (ABS).
    - Resist corrosion, chemicals and biological attack.
    - Light and exceptionally smooth.
    - Cheaper than iron and concrete pipes
    - Used mainly for service connections and household plumbing systems.
    - Affected by water temperature (expansion and compression).
    - Service life ≈ 25 – 30 years.

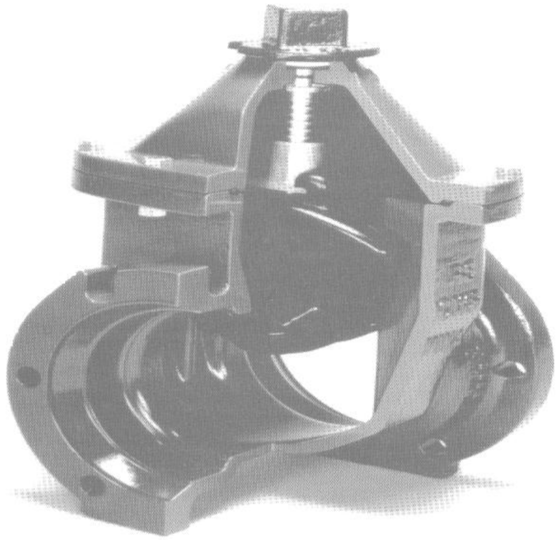
Valves

- Valves are used to control the magnitude or the direction of water flow.
- All valves have a movable part that extends into the pipe for opening or closing the passage of water.
- The means of operating the movable element of a valve are by screws, gears or water pressure.



UG/GGS Information Services

- **Gate Valves**
    - Shutoff valves: to stop the flow of water through pipes.
    - Located at regular intervals (150–350 m) throughout distribution systems so that breaks in the system can be isolated.



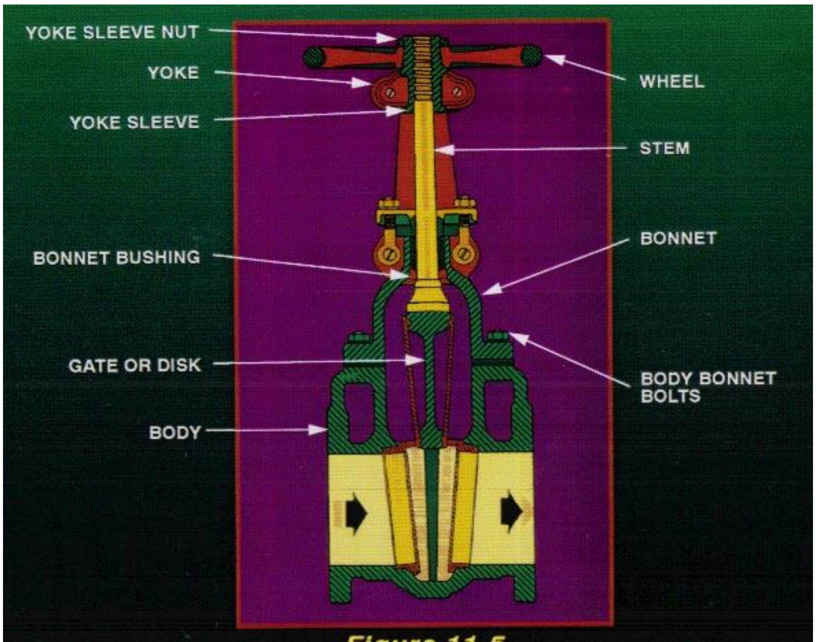
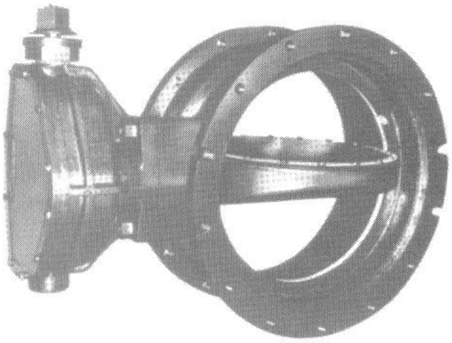


Figure 11-5
CUTAWAY VIEW OF A GATE STOP VALVE (rising stem type)

- **Butterfly Valves**
    - Shutoff valves.
    - Have a movable disk that rotates on axle.

- Cheaper, more compact, easier to operate, and less subject to wear than gate valves.
- Not suitable for water containing solids.



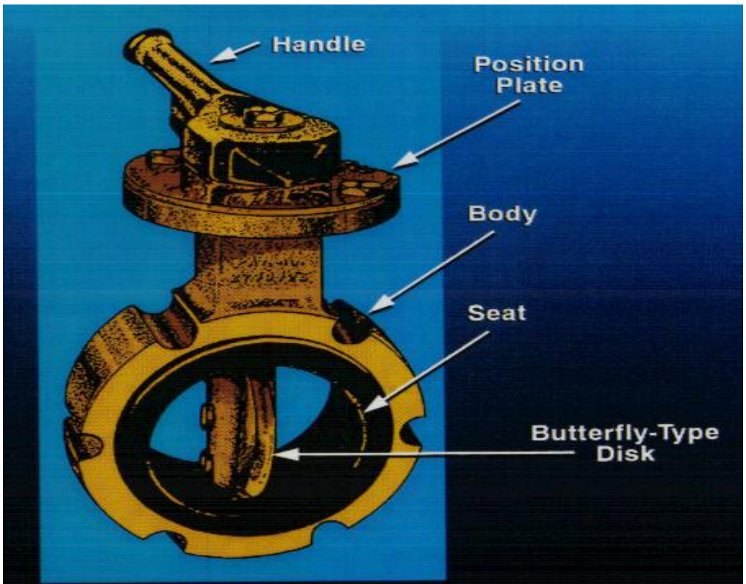
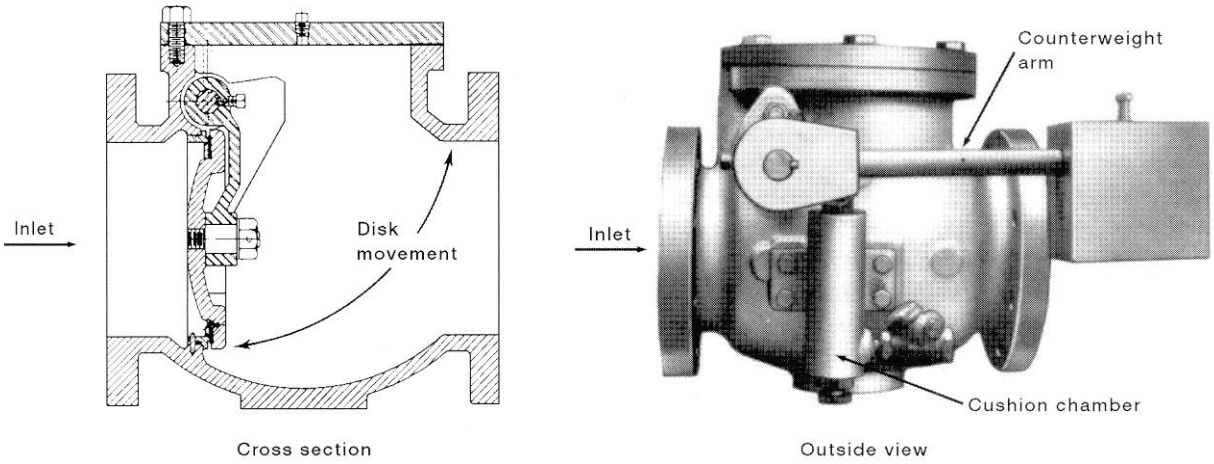


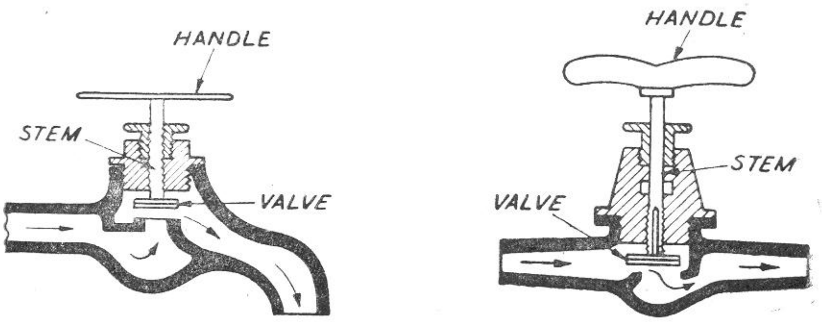
Figure 18-14
Butterfly valve

- **Check Valves**
  - Allow water flow in only one direction (backflow preventer).
  - Open under the influence of water pressure and closes automatically when flow ceases.



- Installed at the end of the a suction line to prevent draining of the suction line and loss of prime when the pump is shut down (foot valves)
- Also, installed on the discharge side of pump to reduce hammer forces on the pumps.

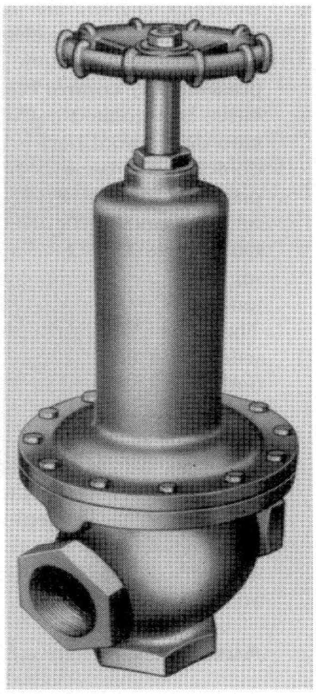
- **Globe Valves**
  - Shutoff valves.
  - Used in household plumbing.
  - Cheap but create high headloss.



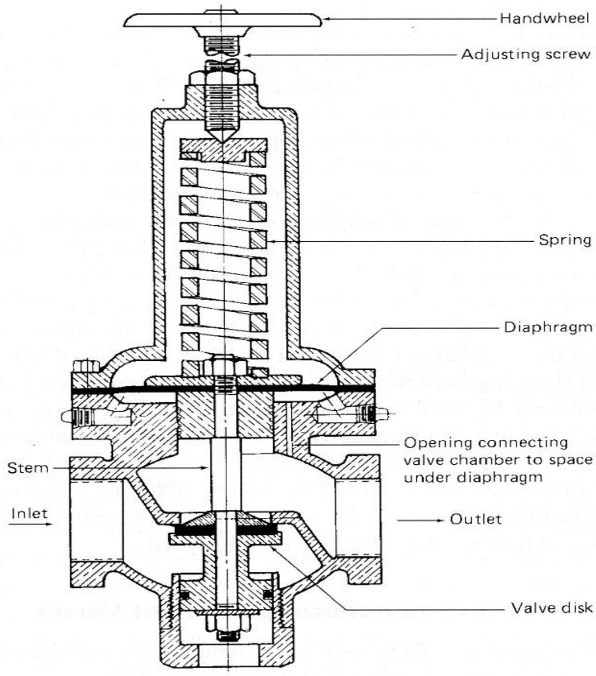
- **Air Relief Valves**
  - Air can enter pipe network (from a pump drawing air into the suction pipe, through leaking joints, and by entrained or dissolved gases being released from the water) and thus increase resistance to water flow by accumulating in high points, in valve domes and fitting, and in discharge lines from pumps.

- Air relief valves are used to discharge trapped air.
- The valve contains a floating ball at the top of a cylinder, sealing a small opening. When air accumulates in the valve chamber, the ball drops away from the outlet opening, allowing the air to escape.

- **Pressure Reducing Valves**
  - Automatically reduce high inlet pressure to a predetermined lower outlet pressure, to prevent excessive pressure in the pipe network at lower elevation.
  - Regulating the flow through the valve controls outlet pressure. Less flow means more headloss is developed and thus the outlet pressure is reduced.



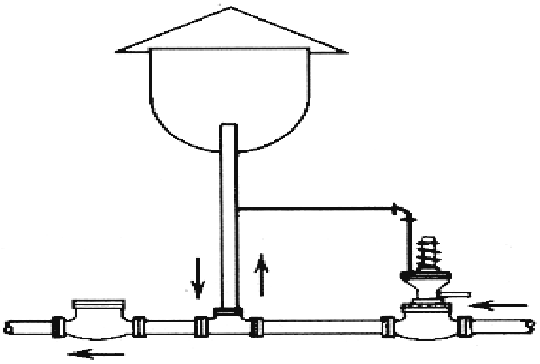
Outside view

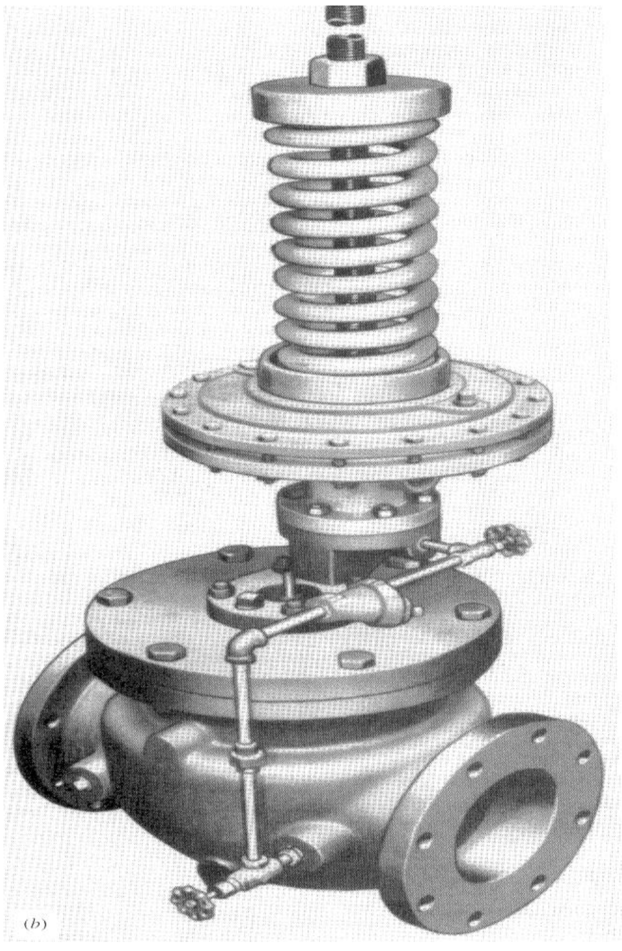


Cross section

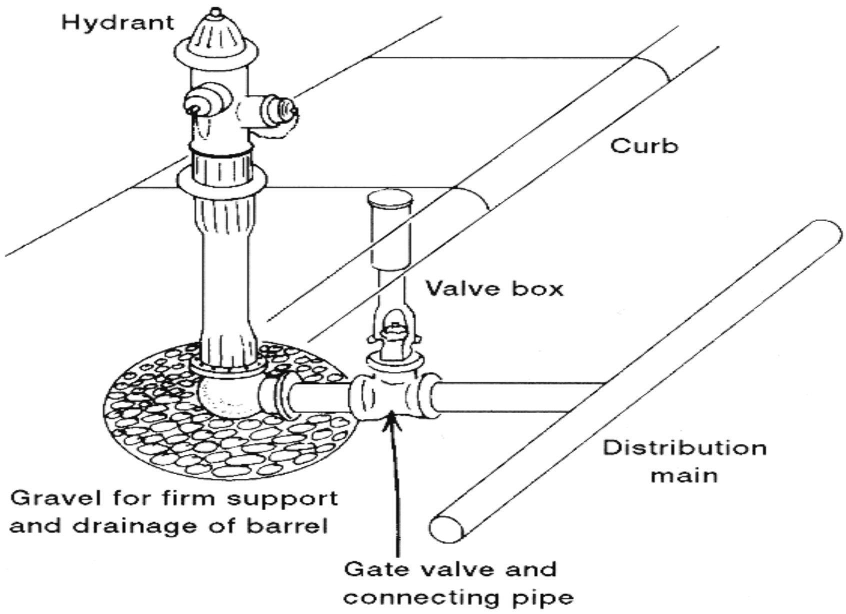
  - The desired outlet pressure is preset by turning the hand wheel to adjust the position of the spring. If the inlet pressure is high the valve will close (the disk is moved upward) to reduce the amount of flow through the valve and thus reduce the pressure.

- **Altitude Valves**
    - To control the flow into and out of an elevated storage tank to maintain desired water-level elevation.
    - The valve closes when the tank is full and remains closed until the water level drops to a predetermined level before it open to refill the tank.
    - Refilling occurs when the system pressure exceeds the tank pressure.





Fire Hydrants



- Hydrants are installed along streets behind the curb line at a distance of 0.6 m (to avoid damage from overhang vehicles).

- Function of fire hydrants:
    - Extinguish fires
    - Wash down streets
    - Flush out water mains

- The pipe connecting the hydrant to a distribution main is normally $\nless$ 150 mm in diameter.

Storm-Water Flow

- When rain falls on a certain area,
  - a portion will be lost by evaporation,
  - a portion will be lost by infiltration into ground,
  - a portion will be intercepted by vegetation,
  - a portion will be held in depression, and
  - a portion will flow overland (run off).

- Storm sewers should be designed to carry the maximum (peak) rain runoff produced from a certain drainage area (catchment basin).

- The peak rate of runoff depends up:
  - Type of precipitation
  - Intensity and duration of rainfall
  - Distribution of rain fall
  - Direction of prevailing storm
  - Climatic condition
  - Type and size of catchment area
  - Characteristics of soil / surface of drainage area

- Methods to determine the peak runoff
  - Empirical formula
  - Rainfall runoff correlation studies
  - The inlet method
  - Digital computer models
  - The hydrograph method
  - The rational method

The Rational Method

$Q = C\ i\ A$

Q = rate of runoff (m^3/h)

i = rainfall intensity (m/h)

A = drainage area (m^2)

C = runoff coefficient (fraction of the rain that appears as runoff)

- **Runoff Coefficient (C)**
    - C relates the combined effects of infiltration, evaporation and surface storage.
    - It increases as the rainfall continues (depends on characteristics of drainage area: slope, moisture, soil type, ground cover, etc.).
    - For impervious surfaces:

$$C = 0.175\ t^{1/3} \quad \text{or} \quad C = t / (8 + t)$$

Where t = duration of storm in minutes

- For improved pervious surfaces:

$$C = 0.3\ t / (20 + t)$$

- Table 13-2 presents C values for various surfaces

TABLE 13-2
Runoff coefficients for various surfaces

Type of surface	*C*
Watertight roofs	0.70–0.95
Asphaltic cement streets	0.85–0.90
Portland cement streets	0.80–0.95
Paved driveways and walks	0.75–0.85
Gravel driveways and walks	0.15–0.30
Lawns, sandy soil	
2% slope	0.05–0.10
2–7% slope	0.10–0.15
> 7% slope	0.15–0.20
Lawns, heavy soil	
2% slope	0.13–0.17
2–7% slope	0.18–0.22
> 7% slope	0.25–0.35

- For a composite drainage area (e.g. residential area and business area) an effective runoff coefficient can be obtained by estimating the percentage of the total area that is covered by roofs, lawn, etc., multiplying each fraction by the corresponding C and then summing the products:

  $C_{effective} = C_1 (A_1/A_{total}) + C_2 (A_2/A_{total}) + \ldots\ldots\ldots$

  $C_{effective} = (C_1 A_1 + C_2 A_2 + \ldots\ldots) / A_{total}$

Example

Determine the run off coefficient for an area of 0.2 Km^2, of which 3000 m^2 is covered by buildings, 5000 m^2 by paved driveways and walk, and 2000 m^2 by Portland cement streets. The remaining area is flat, heavy soil covered by grass.

Solution

Surface Type	**Area (m^2)**	**C (Table 13-2)**	**$C_i (A_i/A_{total})$**
Roofs	3000	0.7 – 0.95	0.7 (3000/200,000) – 0.95 (3000/200,000) = 0.0105 – 0.0143
Paved driveway	5000	0.75 – 0.85	0.75 (5000/200,000) – 0.85 (5000/200,000) = 0.0188 – 0.0213
Cement streets	2000	0.8 – 0.95	0.8 (2000/200,000) – 0.95 (2000/200,000) = 0.008 – 0.0095
Lawn (Slope = 2% ≈ flat)	190,000	0.13 – 0.17	0.13 (190,000/200,000) – 0.17 (190,000/200,000) = 0.124 – 0.162
		$C_{effective} = \sum$	0.16 – 0.21

The effective C = 0.16 – 0.21

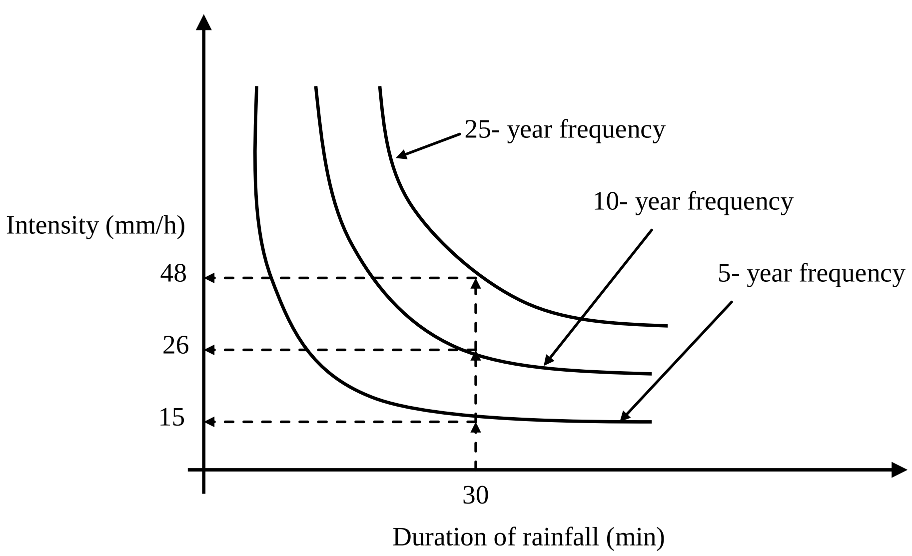
The higher C value can be used as a conservative practice.

- ○ Table 13-3 presents values for composite areas of specific characteristics.

TABLE 13-3
Runoff coefficients for different areas[3]

Description of area	C
Business	
Downtown area	0.70–0.95
Neighborhood area	0.50–0.70
Residential (urban)	
Single-family area	0.30–0.50
Multiunits, detached	0.40–0.60
Multiunits, attached	0.60–0.75
Residential (surburban)	0.25–0.40
Apartment areas	0.50–0.70
Industrial	
Light	0.50–0.80
Heavy	0.60–0.90
Parks, cemeteries	0.10–0.25
Playgrounds	0.20–0.35
Railroad yards	0.20–0.40
Unimproved areas	0.10–0.30

- **Rainfall Intensity – Duration – Frequency Relationships (IDF Curves)**
  - ○ Intensity: the average amount of rain falling per unit time (mm/min).
  - ○ Duration: the time interval the rain falls with a given intensity (min).
  - ○ Frequency (recurrence interval, or return period): the average period in years between rainfalls of a given intensity for a given duration.



- For storms with duration of 30 minutes, the maximum average intensity anticipated once every 5 years is about 15 mm/h, and
- And the maximum average intensity anticipated once every 10 years is 26 mm/h.
- General observations:
  - The shorter the duration, the greater the intensity.
  - A high-intensity storm will have lower frequency (i.e. it will be repeated after long duration of time interval).

- **Frequency (return period)**
  - It establishes the frequency with which the sewer system will be overloaded on an average.

  - The frequency chosen is a matter of economics.
    - It is too expensive to design a sewer to carry largest flow which could ever occur
    - But, flood damage to properties should be avoided when possible.

  - 5-year frequency is used for residential areas.
  - 10-years frequency is used for business areas.
  - 15-year frequency is used for high-value districts.

- **Duration (time of concentration, t_c)**
  - Time of concentration: the time required for the maximum runoff to develop (the time required by the water to reach the outlet or sewer from the most remote point of the drainage area).

  - If the rainfall duration is $< t_c$, the runoff will not be maximum, as the entire area will not contribute to runoff.

- If the rainfall duration is > t_c , the runoff will not be maximum, as the rain intensity reduces with the increase in its duration.

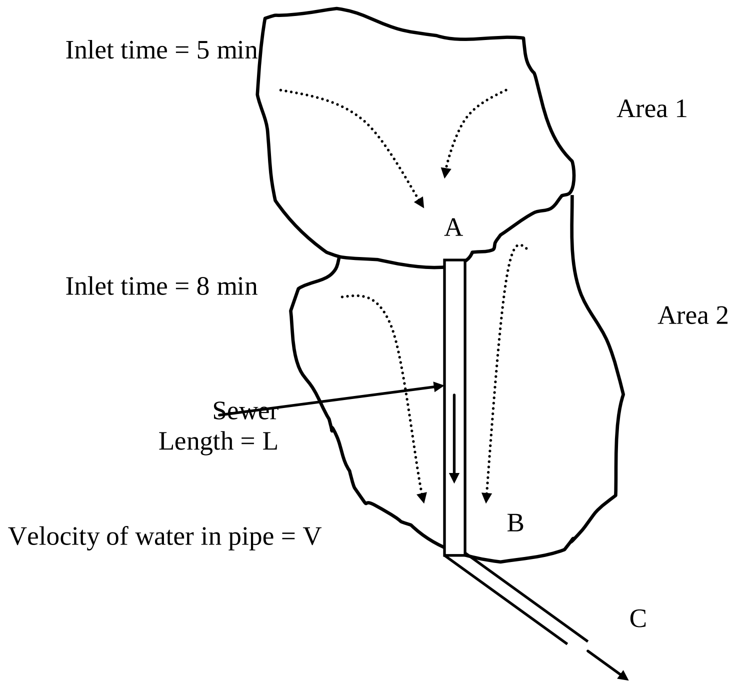
- The maximum runoff will be obtained for the rain having a duration equal to the time of concentration (which is called the critical rainfall duration).

- After t_c is determined, the intensity can be obtained from the intensity-duration-frequency curve.

- Estimation of t_c:

  t_c = Flow time from the most remote point in the drainage area to the point in question.

  = Overland flow time (inlet flow time) + channel flow time.



- If point of interest is A (i.e to find the flow in pipe AB)

  t_c = 5 min

- If the point of interest is B (to find the flow in pipe BC)

  t_c = [5 min + L/V] or t_c = 8 min

  Choose the bigger t_c

- The inlet time
  - Varies depending on the characteristics of the drainage area including amount of lawn, slope of street gutters or ground, spacing of street inlets, etc.
  - For high-density area with closely spaced street inlet, t_c = 5 – 10 minutes.
  - For highly-density areas with relatively flat slopes, t_c = 10 – 20 minutes.
  - For areas with widely spaced street inlets, t_c = 20 – 30 minutes.
  - Figure 13-3 can be used to estimate t_c (the Figure neglects the effect of rainfall intensity).

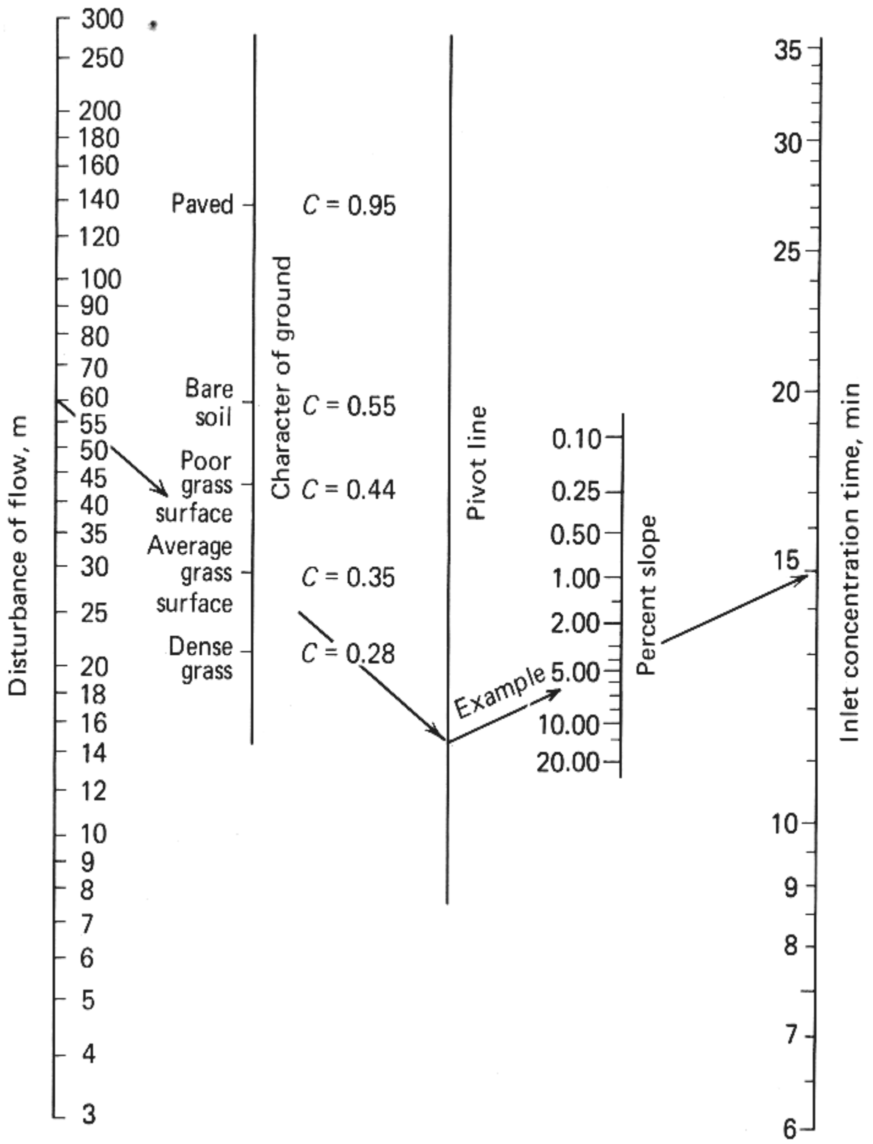


FIGURE 13-3
Overland flow time. (*Modified from a figure in Data Book for Civil Engineering, Vol. I, Design, 3d ed., by Elwyn E. Seelye. Copyright 1960. With permission of John Wiley and Sons, Inc.*)

Example

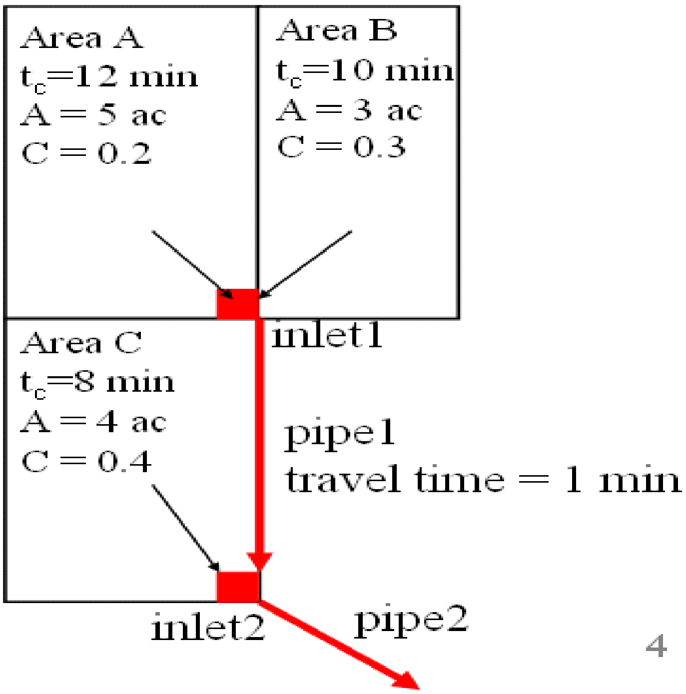
A sewer drains a single-family residential area (A= 10,000 m^2) with C = 0.35. The distance from the most remote point is 60 m over ordinary grass with a slope of 4%. Determine the design flow.

Solution

- From Figure 13-3, inlet time = 15 min. = t_c
- From a curve of the intensity-duration-frequency relationship at t = 15 min for a certain return period (say 10 year return period), the intensity can be found. i = 156 mm/h.
- Q = C i A = 0.35 x 0.156 m/h x 10,000 m^2 = 546 m^3/h.

Example

A storm drain system consisting of two inlets and pipe is to be designed using rational method. A schematic of the system is shown. Determine the peak flow rates to be used in sizing the two pipes and inlets.



1 acre = 0.4047 ha = 4047 m^2 = 43560 ft^2

Rainfall intensity (in/hr) as a function of t is:

$$i = \frac{30}{(t+5)^{0.7}}$$

Size Inlet 1 and pipe 1:

Area A and B contribute, take largest t_c = 12 min

A = 5+3 = 8 acre
C = (5*0.2+3*0.3)/8 = 0.24
I = 30/(12+5)0.7 = 4.13 in/hr
Q = CIA = 0.24*4.13*8 = 7.9 cfs

Size Inlet 2:

Flow from area C contributes, take t_c = 8 min

A = 4 acre
C = 0.4
I = $30/(8+5)^{0.7}$ = 4.98 in/hr
Q = CIA = 0.4*4.98*4 = 8.0 cfs

Size pipe 2:

Flow from all areas
Take tc = 12+1 = 13 min

A = 5+4+3 = 12 acre
C = (5*0.2+4*0.4+3*0.3)/12
= 0.29
I = $30/(13+5)^{0.7}$ = 3.97 in/hr
Q = CIA = 0.29*3.97*12 = 13.8 cfs

Note how t_c is taken as the largest value (12 min) plus travel trough pipe1.

Sewer Materials

- Pipes used to transport water can be used to collect wastewater.
- However, it is better to used less expensive pipes as flow in sewers is by gravity (no pressure)
- Iron and steel pipes can be used if sewage is pressurized.

- **Vitrified Clay Pipe**
  - Made of clay that has been ground, wetted, molded, dried, and burned in a kiln (Vitrification of the clay).
  - High weight, and rigid.
  - Resist biological and chemical attacks, and abrasion.
  - Used mostly in gravity sanitary sewers.

- **Concrete Pipe**
  - High weight, and rigid.
  - Non-reinforced concrete pipe.
  - If size is more than 24 in (610 mm), the pipe should be reinforced.
  - Good for storm sewer because of its large size, strength, and abrasion resistance.
  - Subject to corrosion where acids are present.

- **Thermoplastic pipe**
  - Polyvinyl chloride (PVC), polyethylene (PE) pipes are the most common plastic pipes used in sewer systems.
  - Light weight.
  - Ease in field cutting, and high impact strength.
  - Subject to changes by long-term UV exposure.
  - Subject to environmental stress cracking.
  - Can be used for both gravity and pressure sanitary sewers.
  - PVC pipes are used for building connections and branch sewers.

- PE pipes are used for long pipelines (under adverse conditions: swamp or underwater crossings).

- **Asbestos Cement pipe**
  - Rigid pipe.
  - Subject to corrosion where acids are present.
  - Used for both gravity and pressure sanitary sewers.
  - Available in nominal diameters from 100 to 1000 mm (4 – 42 in).
  - Available in wide range of strength classifications.

Types of Sewers

Table 6-4 Types of sewers in a typical collection system

Type of sewer	Purpose
Building	Building sewers, sometimes called *building connections*, connect to the building plumbing and are used to convey wastewater from the buildings to lateral or branch sewers, or any other sewer except another building sewer. Building sewers normally begin outside the building foundation. The distance from the foundation wall to where the sewer begins depends on the local building regulations.
Lateral or branch	Lateral sewers form the first element of a wastewater collection system and are usually in streets or special easements. They are used to collect wastewater from one or more building sewers and convey it to a main sewer.
Main	Main sewers are used to convey wastewater from one or more lateral sewers to trunk sewers or to intercepting sewers.
Trunk	Trunk sewers are large sewers that are used to convey wastewater from main sewers to treatment or other disposal facilities or to large intercepting sewers.
Intercepting	Intercepting sewers are larger sewers that are used to intercept a number of main or trunk sewers and convey the wastewater to treatment or other disposal facilities.

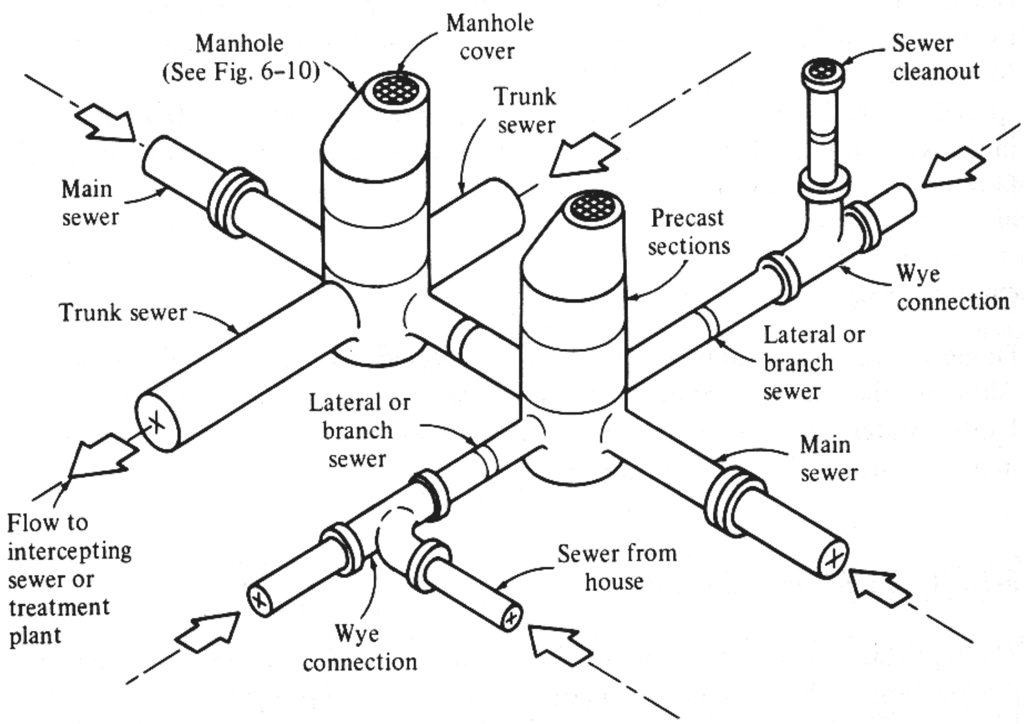


Figure 6-6 Definition sketch for types of sewers used in collection systems.

Sewer Manholes

- **Purpose**
  - to interconnect two or more sewers
  - to provide entry for sewer cleaning and inspection
- **Location**
  - at changes in direction
  - at changes in size
  - at substantial changes in grade
  - at intervals of 90-150 m in straight lines (not more than 150 m except for sewers which can be walked through)

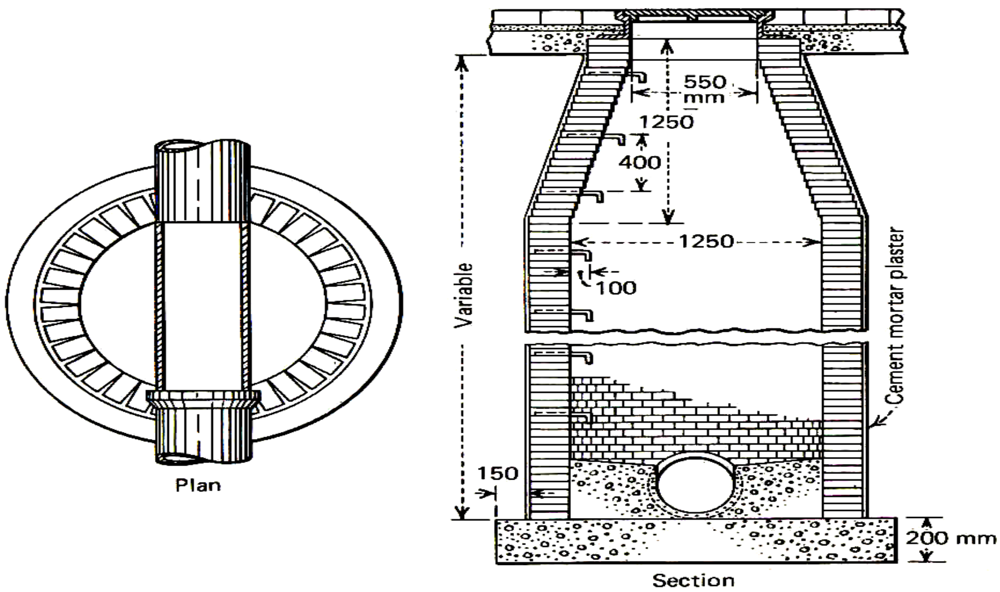


FIGURE 15-1
Brick manhole.

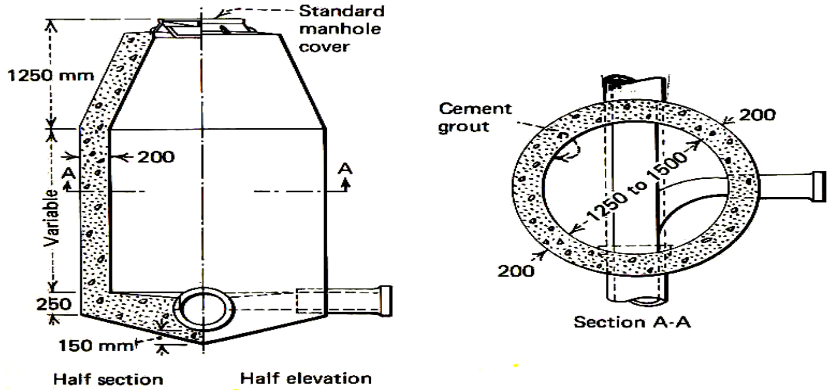


FIGURE 15-2

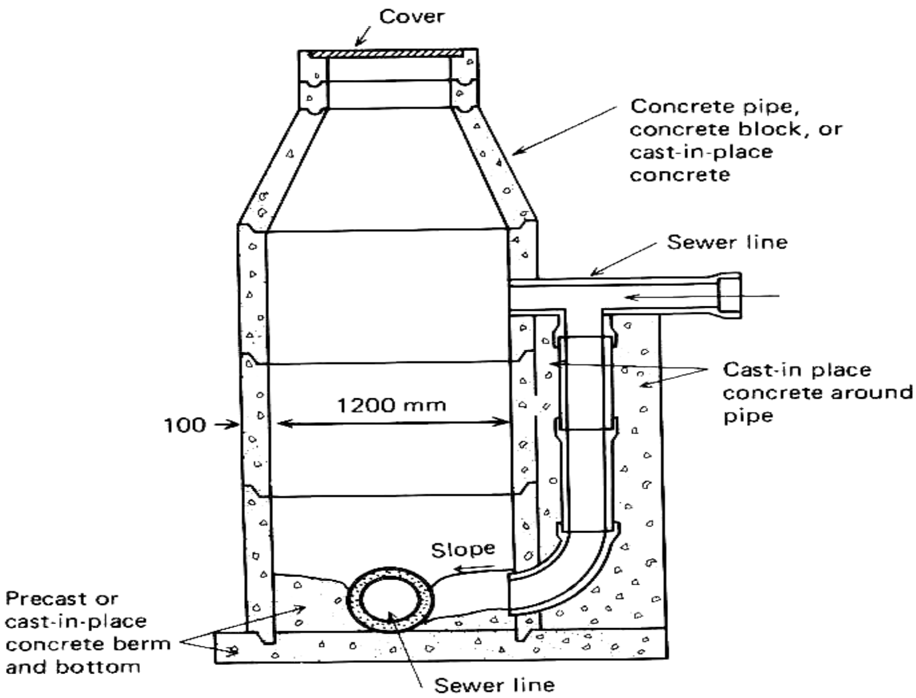


FIGURE 15-3
Precast concrete manhole with drop inlet.

- Drop manhole is usually provided when the difference in elevation between the high and low sewers exceeds 60 cm.

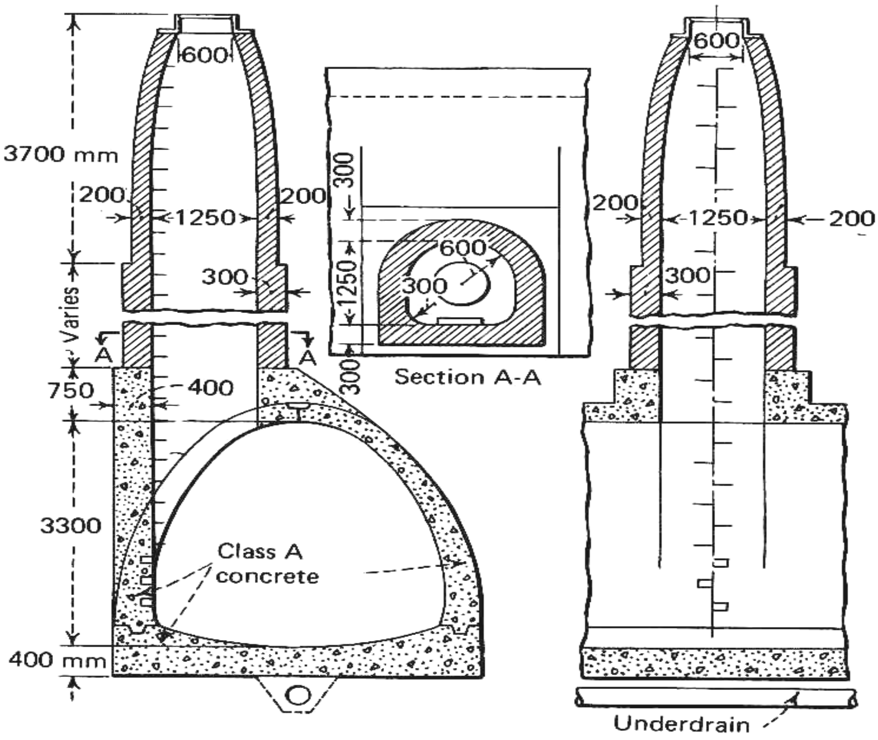


FIGURE 15-4
Manhole access to large sewer.

Sewer Corrosion

- Sulfates SO_4^{--} present in wastewater are reduced to hydrogen sulfide H_2S
- H_2S is released into the air space of the sewer, where it may re-dissolve in condensed moisture accumulated at the crown. Dissolved H_2S is then oxidized to sulfuric acid (H_2SO_4) causing the corrosion of the crown of concrete, iron, or steel sewers.

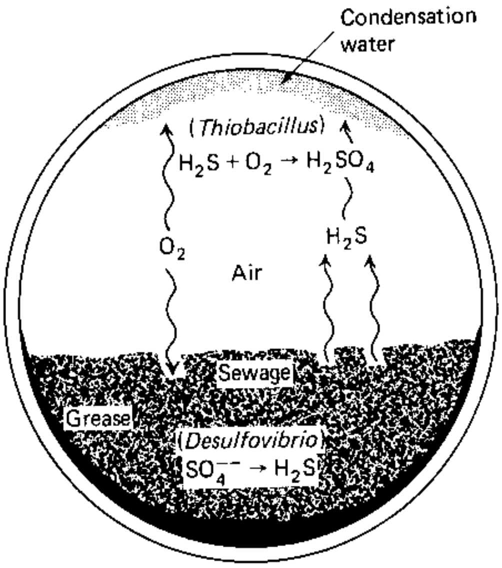


FIGURE 14-8
Schematic diagram of sewer corrosion.

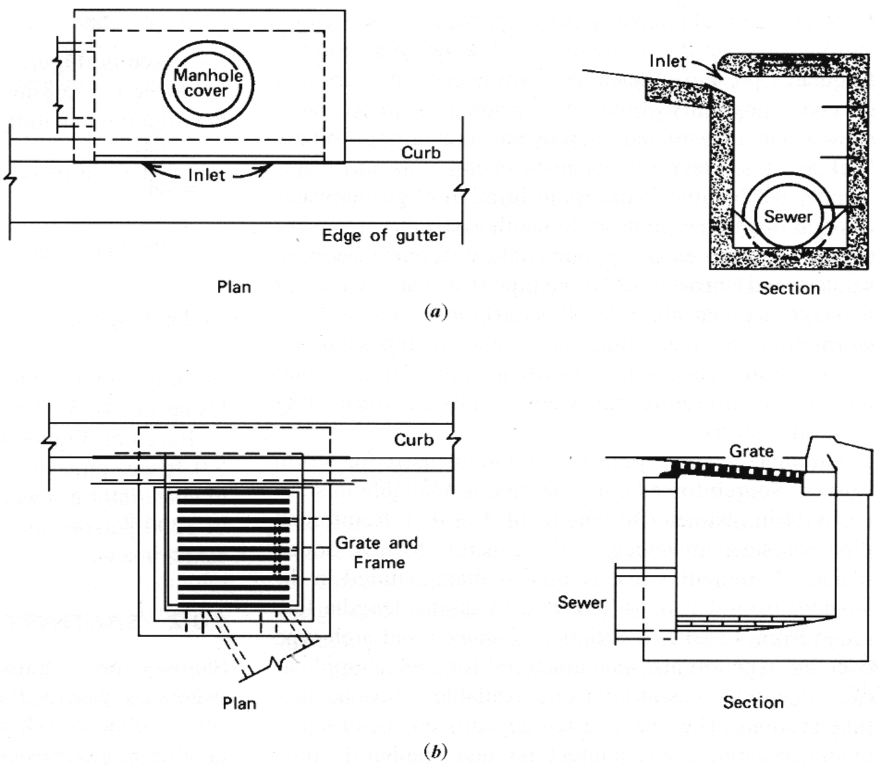
- Control of sewer corrosion
  - By chlorination to halt biological activity (at least temporarily).
  - By forced ventilation to reduce crown condensation, strip H_2S from the atmosphere of the sewer, and provide sufficient oxygen to prevent sulfate reduction.
  - By lining sewer with inert materials (plastics, clay tiles, or asphaltic compounds).

Storm-Water Inlets

- Surface waters enter a storm drainage system through inlets located in street gutters or depressed areas that collect natural drainage.
- Catch basins under street inlets are connected by short pipeline to the main sewer.

- Types of storm-water inlets:
  - Curb Inlets (vertical inlets): vertical openings in the curb.
  - Gutter Inlets (horizontal inlets): depressed or un-depressed openings in the gutter section of the street.
  - Combination Inlets: composed of both curb and gutter openings.

Figure 10-1 Storm-water inlets for street drainage. (*a*) Curb inlet. (*b*) Gutter inlet.



- Locations of inlets
  - Placed at gutters usually at street intersections.
  - At mid points of the blocks if they are more than 150 m long.

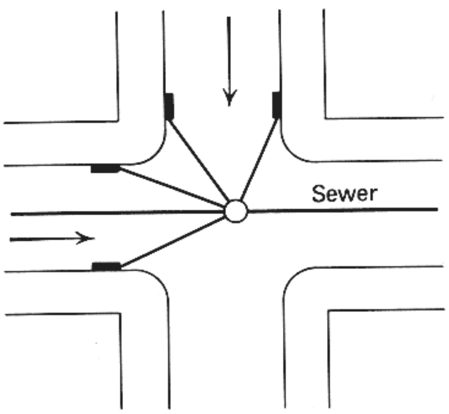


FIGURE 15-7
Street intersection showing inlets and branch lines from inlets to main sewer. Arrows show direction of surface flow.

Hydraulics of Sewers

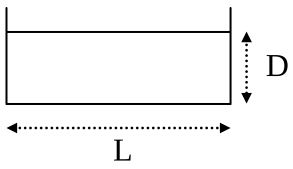
- Sanitary and storm sewers are usually designed to flow as open channels (gravity flow), partially full, and not under pressure.
- Manning equation is used in sewer design:

$$\mathbf{Q = (1 / n)\ A\ R^{2/3}\ S^{0.5}}$$

Q = flow rate (m^3/s)
A= cross sectional area of flow (m^2)
R = hydraulic radius (m/m) = cross-sectional area of flow / wetted perimeter

$$R = \frac{\pi D^4 /4}{\pi D} = \frac{D}{4}$$ For circular pipes flowing full

$$R = \frac{D L}{L + 2 D}$$ For



S = slope of energy grade line.

S = pipe invert slope (slope of pipe bottom), For uniform flow (i.e. $V_1 = V_2$ and $P_1/ \gamma = P_2/ \gamma$).

n = coefficient of roughness (Table 3-5)

n = 0.013 is used to analyze well constructed existing sewers and to design new sewers.

n = 0.015 is used to analyze most old existing sewers.

- Nomograms shown in Figure 16-2 through 16-4 can be used for circular pipes flowing full (n = 0.013):
    - Given any two parameters (Q, D, S, or V), the remaining two can be determined.

TABLE 3-5
Values of Manning's roughness coefficient

Material	Value
Glass, plastic, machined metal	0.010
Dressed timber, joints flush	0.011
Sawn timber, joints uneven	0.014
Cement plaster	0.011
Concrete, steel-troweled	0.012
Concrete, timber forms, unfinished	0.014
Untreated Gunite	0.016
Brickwork or dressed masonry	0.014
Rubble set in cement	0.017
Earth, smooth, no weeds	0.020
Earth, some stones and weeds	0.025

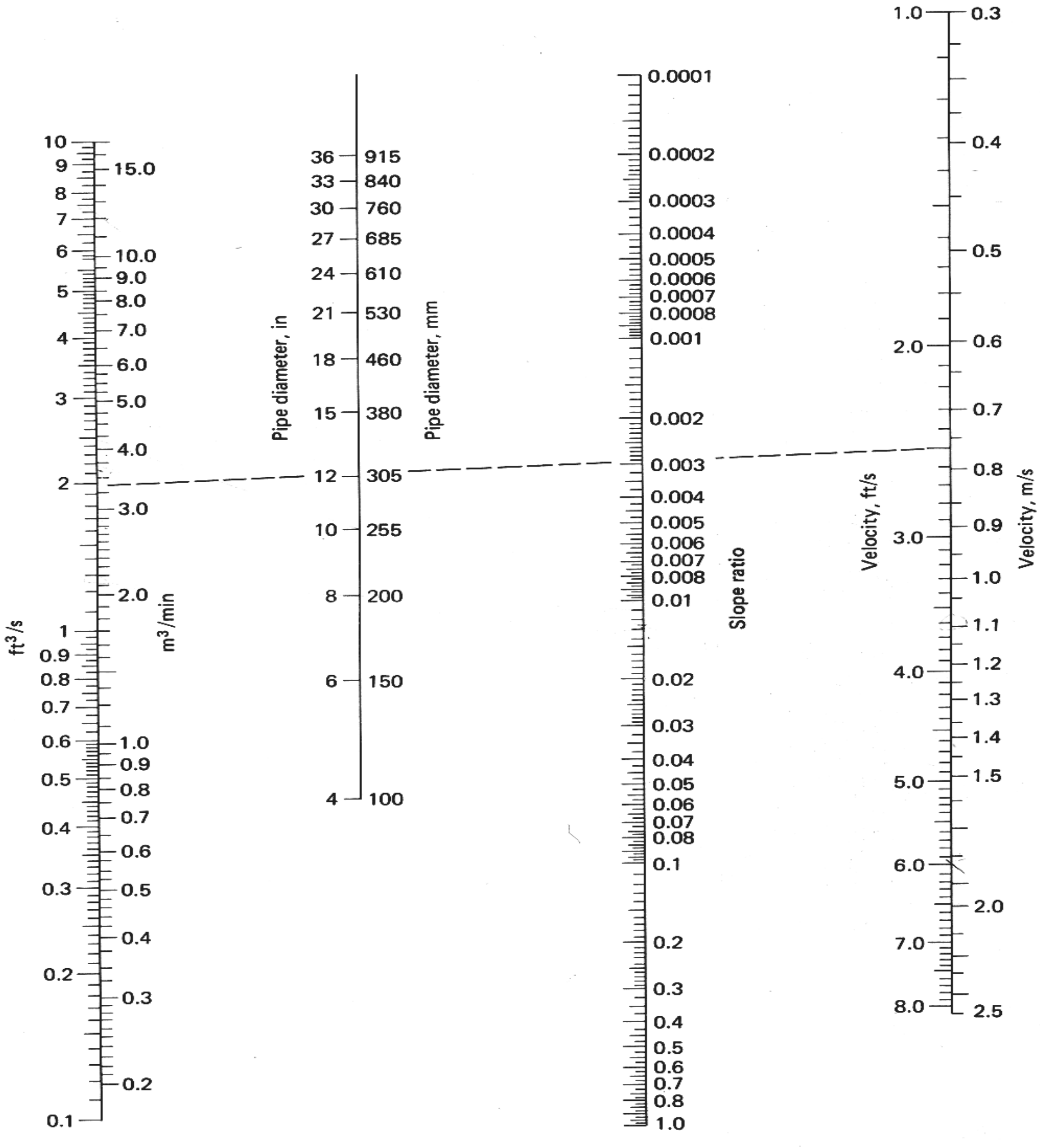


FIGURE 16-2
Nomogram for solution of Manning's equation for circular pipes flowing full (n = 0.013).

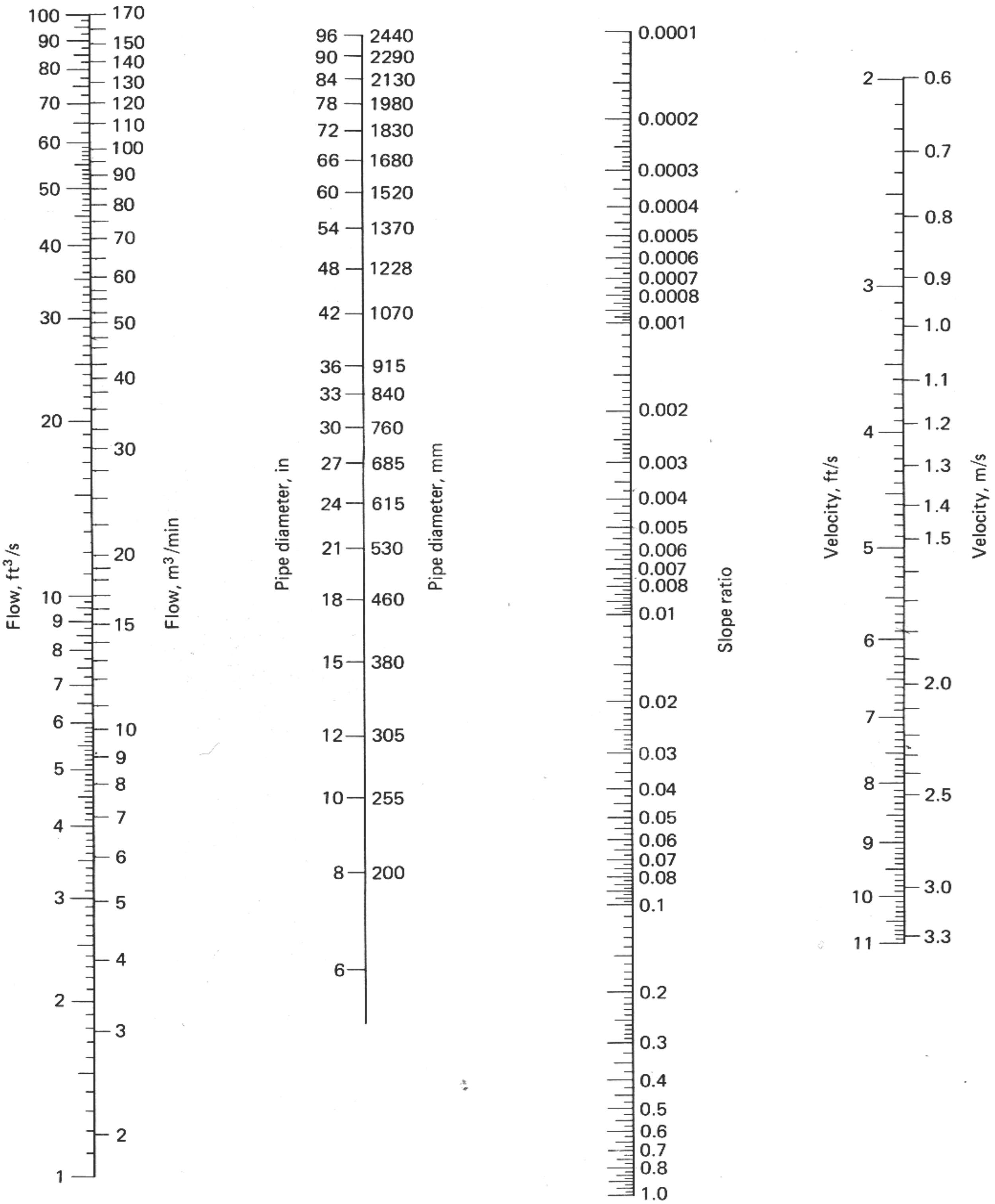


FIGURE 16-3
Nomogram for solution of Manning's equation for circular pipes flowing full (n = 0.013).

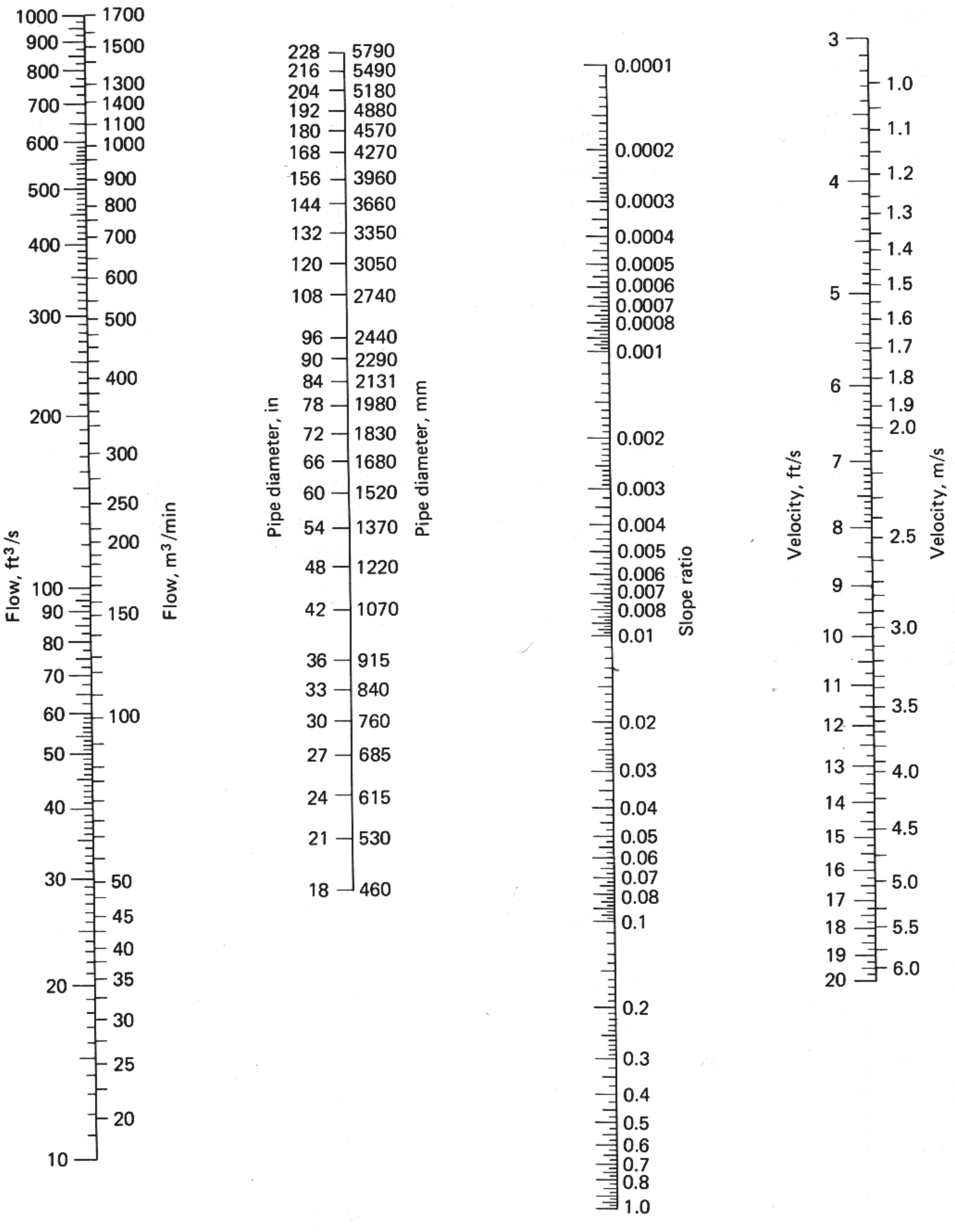


FIGURE 16-4
Nomogram for solution of Manning's equation for circular pipes flowing full (n = 0.013).

Example

D = 200 mm, and S = 0.02 (i.e. 2%)

Then, from Fig. 16-2 we have Q = 2.9 m^3/min and V = 1.48 m/s

Example

If a 255-mm sewer is placed on a slope of 0.01, what is the flowing full quantity, velocity of flow at n = 0.013 and n = 0.015.

From Fig. 16-2 at D = 255 mm and S = 0.01 for n = 0.013

$\rightarrow$ Q = 3.75 m^3/min and V = 1.25 m/s

$$\frac{Q_{n2}}{Q_{n1}} = \frac{(1/n_2)\, A\, R^{2/3}\, S^{0.5}}{(1/n_1)\, A\, R^{2/3}\, S^{0.5}} \rightarrow \frac{Q_{n2}}{Q_{n1}} = \frac{n_1}{n_2}$$

$$\text{Similarly } \frac{V_{n2}}{V_{n1}} = \frac{n_1}{n_2}$$

$Q_{n = 0.015} = Q_{n = 0.013} \times (n_1/n_2) = 3.75 \times (0.013/0.015) = 3.25\ m^3/min$

$V_{n = 0.015} = V_{n = 0.013} \times (n_1/n_2) = 1.25 \times (0.013/0.015) = 1.1$ m/s

- Figure 16-6 is useful in estimating partial flow values (q, d, v) from full-flow conditions (Q, D, V).

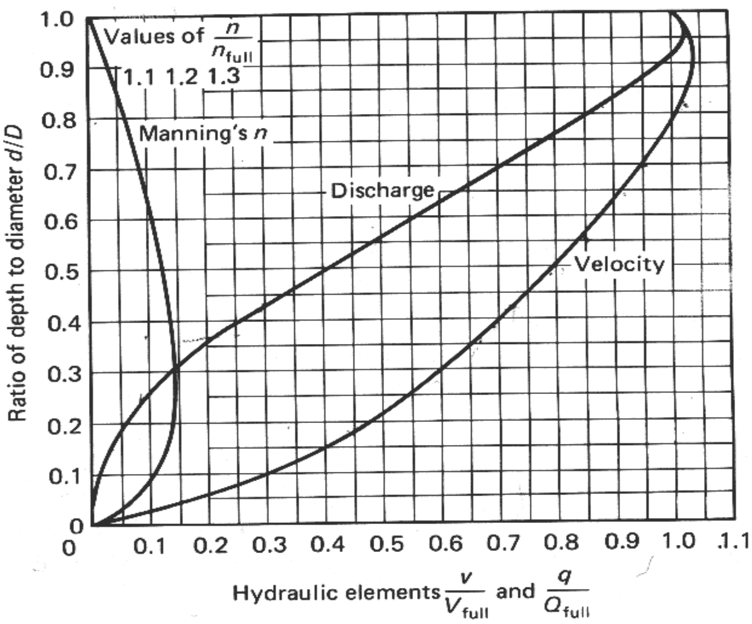


FIGURE 16-6
Variation of flow and velocity with depth in circular pipes.

Example

The measured depth of flow in 1228-mm concrete storm sewer on a grade of 0.002 m/m is 737 mm. What are the quantity and velocity of flow?

From Fig. 16-3 at D = 1228 mm and S = 0.002 (i.e. 0.2%)

$Q = 110\ m^3/min$ and $V = 1.55\ m/s$

d/D = 737/1228 = 0.6 then from Fig. 16-6 q/Q = 0.55 and v/V = 0.87

$q = 0.55 \times 110 = 60.5\ m^3/min$

$v = 0.87 \times 1.55 = 1.35\ m/s$

Example

If the flow in a pipe is at a depth of 50% of the pipe diameter,

Then d/D = 0.5 and from Fig. 16-6 → q/Q = 0.4 and v/V = 0.8

i.e. Q = q/0.4 → Q = 2.5 q

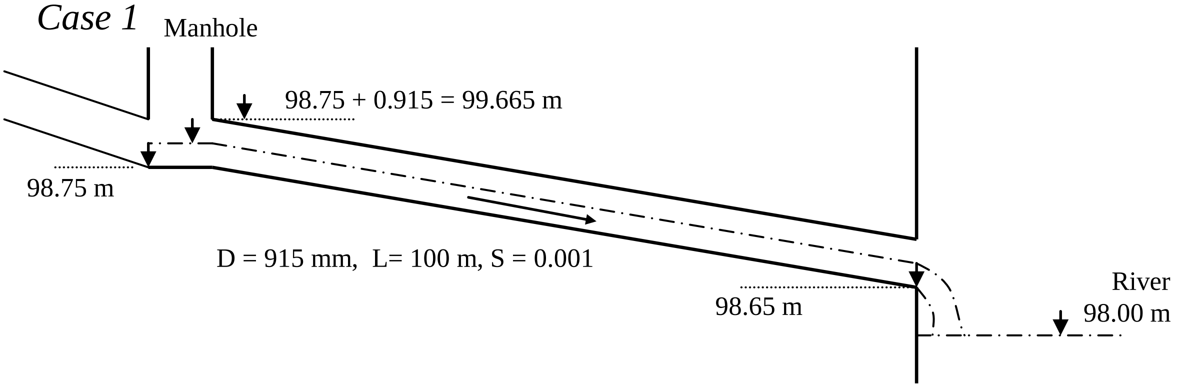
A sewer selected to be full at 2.5 times the design flow will be half full at the actual flow (q).

Example

A 915-m sewer is installed on a slope of 0.001. The sewer is 100 m long and runs from a manhole at which its invert is at 98.75 to a river discharge at which its invert is at 98.65. The pipe is to carry a flow of $0.28\ m^3/s$.

What is the depth of the water at the upstream manhole when the downstream water surface at 98 m? and at 100 m ?

Case 1



D = 915 mm and S = 0.001

Then from Fig. 16-3 → Q = 38 m^3/min = 0.63 m^3/s

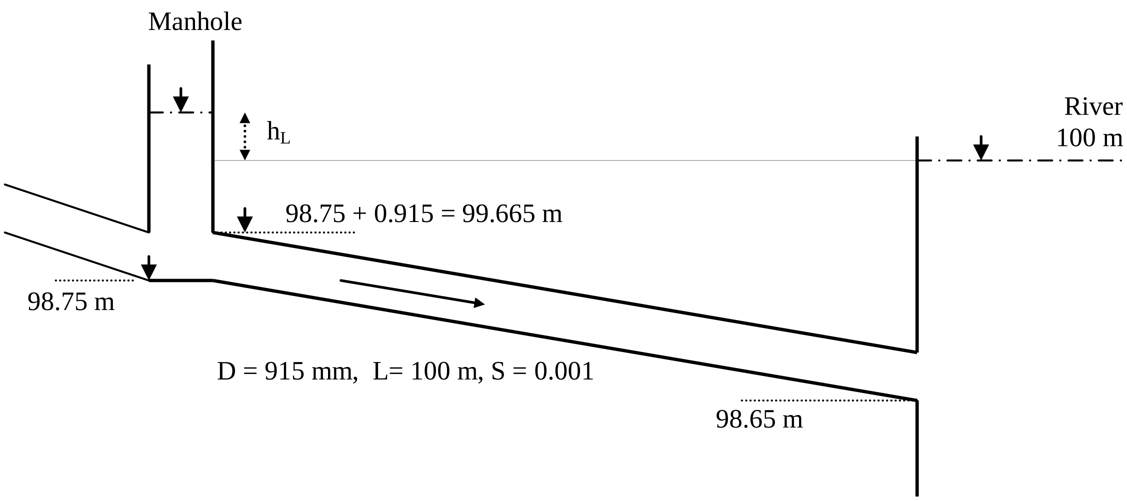
q/Q = 0.28 / 0.63 = 0.44

Then from Fig 16-6 at q/Q = 0.44 → d/D = 0.52

d = 0.52 x 915 = 475.8 mm

Elevation of water surface at upstream end = 98.75 + 0.4758 = 99.23 m

Case 2



As the water surface at the lower end (river) is above the crown at the upper end, then the pipe is flowing full throughout its length.

For the water to flow through the pipe at a rate of 0.28 m^3/s, the water level in the upstream manhole has to be higher than the water level at the lower end (river).

By how much? → By h_L.

At Q = 0.28 m^3/s and D = 915 mm → S = 0.00023 m/m

h_L = headloss = losses due friction + minor losses (exit and entrance)

h_L = 100 x 0.00023 + 0.014 (assumed) = 0.037 m

Elevation of water surface at upstream end = 100 + 0.037 = 100.037 m

Design of Sewer Systems

First: Preliminary Design

- To evaluate the feasibility of the project and estimate costs.
- Prepare a map(s) of the city showing locations of streets, alleys, public parks, buildings, streams, railways, elevations of streets, surface slopes, high and low points (ground contours), and other features and structures which may influence or may be influenced by the sewer system.
- Estimate the anticipated population, its density, and its waste production at the end of the design periods, and predict future commercial and industrial development (design period is indefinite but > 20 years).
- Select sites for treatment plants and or disposal.
- Based on the previous information, estimate the quantity of pips of various sizes (Q and slope are known then size can be estimated), the quantity of excavation, the quantity of pavement repairs and other appurtenances (manholes, drop inlets, building connections, ...).
- Estimate costs for the different alternatives identified as physically practicable and environmentally acceptable.

Second: Detailed Design

(1) Underground Survey

- Locate any underground obstacles such as existing sewers, water mains, telephone and electrical cables, tunnels, and other underground structures.
- Establishes soil properties and locations of rock and other difficult sub-surface conditions in the area of the project (may require soil borings and soundings).

- Establish the location of underground water table.

(2) **Preparation of maps:** Maps (scale: 1:500 to 1:1000) should include:

- Locations of all underground structures.
- Locations and basement elevations of all buildings.
- Locations and elevations of all streams (wadis), ditches, and max. and min. levels therein.
- Street centerline elevations every 15 m and at any abrupt changes in surface slope.
- Elevations of street intersections.
- Types of street pavement.
- Ground contours (at 0.5 to 3 m intervals).
- Permanents benchmarks on each block of every street in which a sewer is to be laid.
- Profiles of all streets in which sewers are to be placed.

(3) Layout of the System

A tentative layout of the proposed system is made on the map by:

- Locating sewer lines along the streets with arrows showing the direction of flow, which is normally in the direction of the ground slope.
    - Location: the most common location of a sanitary sewer is at or near the center of the street serving both sides of the street. In wide streets, it may be more economical to install a sewer on each side.
    - Depth of sewer: limited by the desirability of minimum excavation, service to basement sanitary facilities, and the need to provide

minimum cover to prevent freezing in cold climate, and backflow through connections.

- Sewers should be located at depths such as that it can pass all other utilities with the possible exception of storm sewers.

- Minimum cover: 3 m in cold climate; ≥ 0.75 m depending on pipe size and the anticipated loads.

- Locating manholes at sewer intersections, abrupt changes in horizontal direction or slope, changes in size, and at regular intervals along straight runs (spacing should not exceed 150 m except in sewers which can be walked through). Manholes should be numbered for identification.

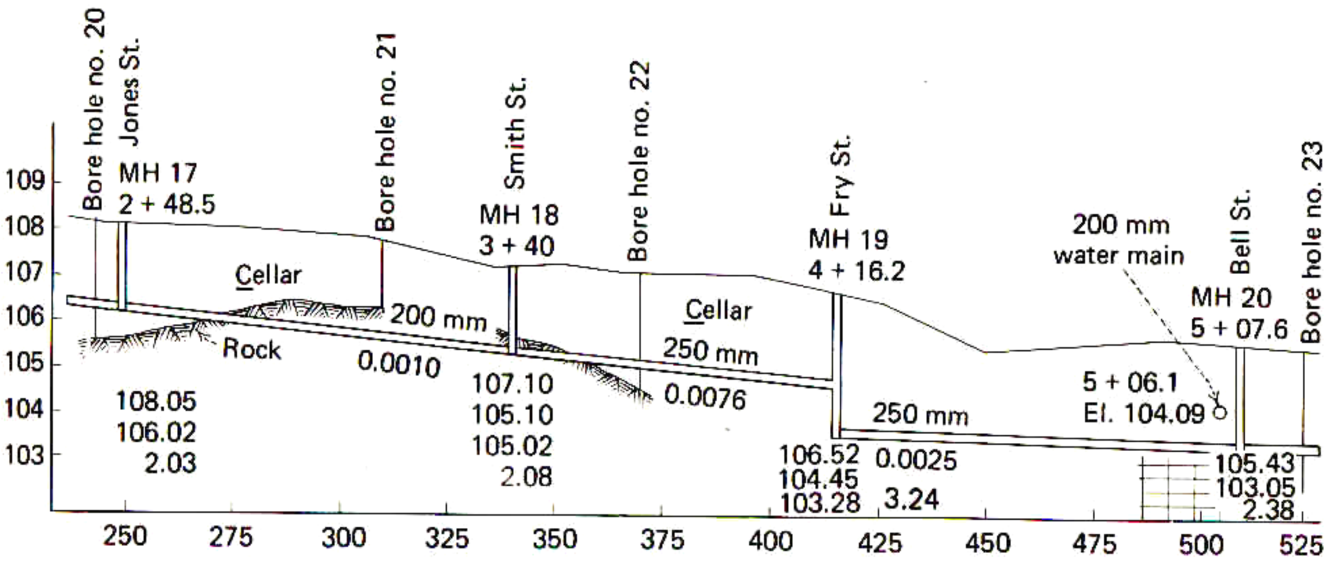
(4) The Vertical Profile

- Prepare a vertical profile for each sewer line (horizontal scale 1:500 to 1:1000; vertical scale of about 10 times greater).
    - The profile shows:
        - The ground surface.
        - Tentative manhole locations
        - Elevations of important subsurface structures/strata (rock, basements, telephone line, water mains, ….).
        - Locations of bore holes.
        - Cross streets.

    - The profiles are used to assist in design and serve as basis for construction drawings. A plan of the sewer line and relevant other structure is usually shown on the same sheet.

- When design is completed, pipe sizes and slopes, elevations at changes in size and grade are noted on these profiles. See Figure 16-1. Note that the values listed at each manhole represent:

  The ground elevation,
  Entering invert elevation,
  Leaving invert elevation, and
  The cut to the leaving invert.



(5) Principles of Design

- Design using Manning equation.
- Velocity:
  - For sanitary sewers

    V_{min} = 0.6 m/s – 0.75 m/s
    V_{max} = 3 m/s
    V_{min} for depressed sewers = 1.0 m/s

  - For storm sewers

    V_{min} = 0.9 m/s
    V_{max} = 3 m/s

- The usual practice is to design the sewer slopes to ensure the minimum velocity with flow at one-half full flow / full depth.

- Minimum size of pipe:
    - 200 mm.
    - Use 100 – 150 mm pipes for house connection.
- Minimum slopes:
    - Sewers with low slopes are often recommended to avoid excessive excavation.
- Change in sewer direction without change in size:
    - A drop of about 30 mm is to be provided in the manhole to account for the loss in head resulting from the change in direction.
- Change in sewer size with or without change in direction:
    - The crowns of the smaller and the larger sewer are matched to ensure that the smaller sewer will not be caused to flow full by backflow from the larger sewer unless the larger is also full.

Sewer Design Example

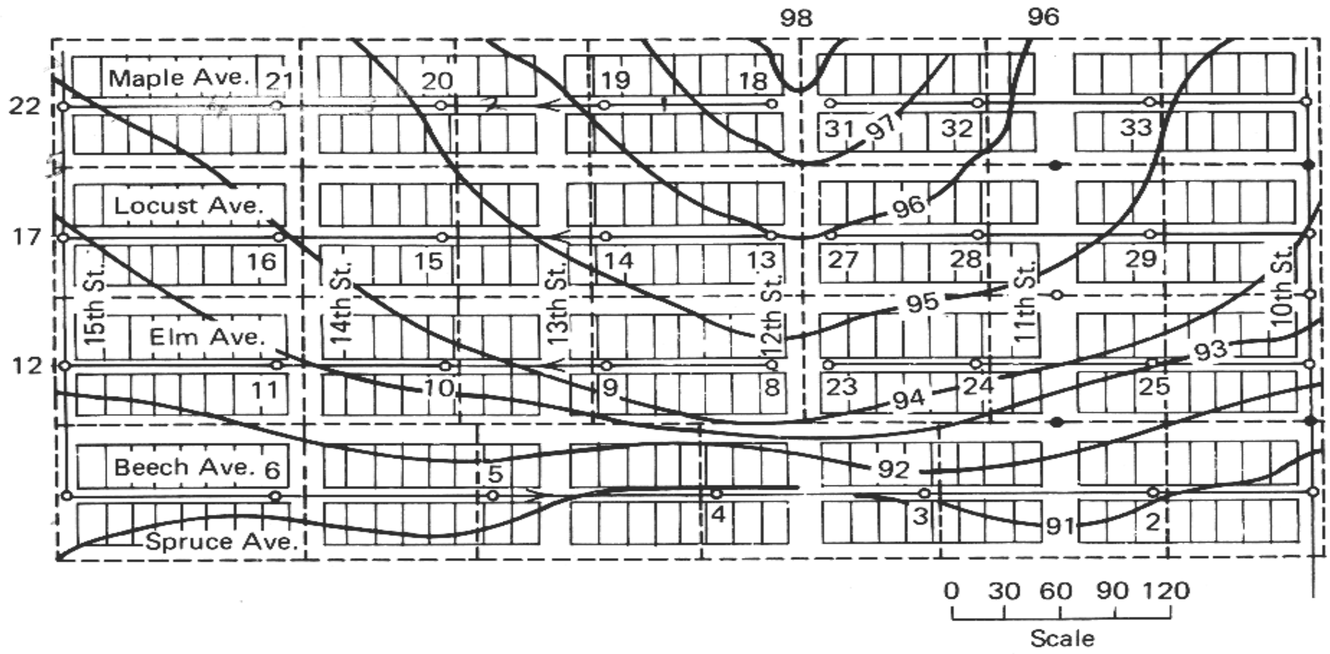


FIGURE 16-7
Portion of a sanitary sewer system.

(*The area tributary to each line is shown by the dash lines on the map)*

- Population density = 10,000 persons/km^2
- Maximum flow rate = 1500 L/c.day (including infiltration)
- Minimum cover over sewer pipes = 2 m
- Minimum sewer diameter = 200 mm
- Minimum velocity = 0.6 m/s, and n = 0.013

Table 16.1 Data for flow in a sanitary sewer system

Line no. (1)	On street (2)	From man-hole (3)	To man-hole (4)	Length of line, m (5)	Increment of area, m² (6)	Increment of population (7)	Total tributary population (8)	Sewage flow, l/day (9)	Sewage flow, m³/min (10)
0	15th	...	22	....	....	....	5725	8,587,500	5.96
1	Alley between Maple and Locust	18	19	90	10,000	100	100	150,000	0.10
2	Alley between Maple and Locust	19	20	90	7,000	70	170	255,000	0.18
3	Alley between Maple and Locust	20	21	90	7,000	70	240	360,000	0.25
4	Alley between Maple and Locust	21	22	120	12,000	120	360	540,000	0.38
5	15th	22	17	87	....	....	6085	9,127,500	6.34
6	Alley between Locust and Elm	13	14	90	10,000	100	100	150,000	0.10
7	Alley between Locust and Elm	14	15	90	7,000	70	170	255,000	0.18
8	Alley between Locust and Elm	15	16	90	7,000	70	240	360,000	0.25
9	Alley between Locust and Elm	16	17	120	12,000	120	360	540,000	0.38
10	15th	17	12	87	....	....	6445	9,667,500	6.71
11	Alley between Elm and Beech	8	9	90	10,000	100	100	150,000	0.10
12	Alley between Elm and Beech	9	10	90	7,000	70	170	255,000	0.18
13	Alley between Elm and Beech	10	11	90	7,000	70	240	360,000	0.25
14	Alley between Elm and Beech	11	12	120	12,000	120	360	540,000	0.38
15	15th	12	7	87	....	....	6805	10,207,500	7.09
16	Alley between Beech and Spruce	7	6	120	12,000	120	6925	10,387,500	7.21
17	Alley between Beech and Spruce	6	5	120	9,000	90	7015	10,522,500	7.31
18	Alley between Beech and Spruce	5	4	120	12,000	120	7135	10,702,500	7.43
19	Alley between Beech and Spruce	4	3	120	11,000	110	7245	10,867,500	7.55
20	Alley between Beech and Spruce	3	2	120	11,000	110	7355	11,032,500	7.66
21	Alley between Beech and Spruce	2	1	90	7,000	70	7425	11,137,500	7.73

Line MH.18 – MH.19

L = 90 m

Ground elevations = 97.74 m and 96.4 m

Tributary area = 10,000 m^2 = 0.01 km^2

Tributary population = 0.01 km^2 x 10,000 persons/km^2 = 100 people

Max. flow rate = 1500 L/c.d x 100 = 150,000 L/d = 150 m^3/d = 0.1 m^3/min

S = (97.74 – 96.4) / 90 = 0.0149

At S = 0.0149 and Q = 0.1 m^3/min --> D < 200 mm

Then at S = 0.0149 and D = 200 mm --> Q = 2.4 m^3/min and V = 0.59 m/s < 0.6 m/s, Therefore the slope (S) has to be increased to give V = 0.6 m/s

At S = 0.018 and D = 200 mm --> Q = 2.8 m^3/min and V = 1.42 m/s

q/Q = 0.1/2.8 = 0.04 --> v/V = 0.45 --> v = 1.42 x 0.45 = 0.64 m/s > 0.6 m/s (OK), then D = 200 mm , S = 0.018, Q_f = 2.8 m^3/min, q = 0.1, v = 0.64 m/s

Invert at the upper end = 97.74 – 2 – 0.2 = 95.54 m

Invert at the lower end = 95.54 – (0.018 x 90) = 93.92 m

Line MH.19 – MH.20

L = 90 m

Ground elevations = 96.4 m and 95.27 m

Tributary area = 7,000 m^2 = 0.007 km^2

Tributary population = 0.007 km^2 x 10,000 persons/km^2 = 70 people

Total tributary population = 100 + 70 = 170 people

Max. flow rate = 1500 L/c.d x 170 = 255,000 L/d = 255 m^3/d = 0.18 m^3/min

S = (96.4 – 95.27) / 90 = 0.0125 = 0.013

At S = 0.013 and Q = 0.18 m^3/min --> D < 200 mm

Then at S = 0.013 and D = 200 mm --> Q = 2.35 m^3/min and V = 1.22 m/s > 0.6 m/s

q/Q = 0.18/2.35 = 0.08 --> v/V = 0.53 --> v =1.22 x 0.53 = 0.65 m/s > 0.6 (OK), then D = 200 mm , S = 0.013, Q_f = 2.35 m^3/min, q = 0.18, v = 0.65 m/s

Invert at the upper end = 93.92 m

Invert at the lower end = 93.92 – (0.013 x 90) = 92.75 m

Line MH.20 – MH.21

L = 90 m

Ground elevations = 95.27 m and 93.93 m

Tributary area = 7,000 m^2 = 0.007 km^2

Tributary population = 0.007 km^2 x 10,000 persons/km^2 = 70 people

Total tributary population = 170 + 70 = 240 people

Max. flow rate = 1500 L/c.d x 240 = 360,000 L/d = 360 m^3/d = 0.25 m^3/min

S = (95.27 – 93.93) / 90 = 0.015

At S = 0.015 and Q = 0.25 m^3/min --> D < 200 mm

Then at S = 0.015 and D = 200 mm --> Q = 2.5 m^3/min and V = 1.3 m/s > 0.6

q/Q = 0.25/2.5 = 0.1 --> v/V = 0.55 --> v = 1.3 x 0.55 = 0.72 m/s > 0.6 (OK)

But at S = 0.011 and D = 200 mm --> Q = 2.1 m^3/min and V = 1.1 m/s > 0.6

q/Q = 0.25/2.1 = 0.12 --> v/V = 0.58 --> v = 1.1 x 0.58 = 0.64 m/s > 0.6 (OK)

Then D = 200 mm, S = 0.011, Q_f = 2.1 m^3/min, q = 0.25 m^3/min and v = 0.64 m/s

Invert at the upper end = 92.75 m

Invert at the lower end = 92.75 – (0.011 x 90) = 91.73 m

Line MH.21 – MH.22

L = 120 m

Ground elevations = 93.93 m and 93.69

Tributary area = 12,000 m^2 = 0.012 km^2

Tributary population = 0.012 km^2 x 10,000 persons/km^2 = 120 people

Total tributary population = 240 + 120 = 360 people

Max. flow rate = 1500 L/c.d x 360 = 540,000 L/d = 540 m^3/d = 0.375 m^3/min

S = (93.93 – 93.69) / 120 = 0.002

At S = 0.002 and Q = 0.38 m^3/min --> D < 200 mm

Then at S = 0.002 and D = 200 mm --> Q = 0.9 m^3/min and V = 0.47 m/s < 0.6 (not OK)

At S = 0.007 and D = 200 mm --> Q = 1.7 m^3/min and V = 0.89 m/s > 0.6 (OK)

q/Q = 0.38/1.7 = 0.22 --> v/V = 0.68 --> v = 0.89 x 0.68 = 0.61 m/s > 0.6 (OK)

Then D = 200 mm, S = 0.007, Q_f = 1.7 m^3/min, q = 0.38 m^3/min and v = 0.61 m/s

Invert at the upper end = 91.73 m

Invert at the lower end = 91.73 – (0.007 x 120) = 90.89 m

Line MH.22 – MH.17

L = 87 m

Ground elevations = 93.69 m and 92.99

Total tributary population = 360 + 5725 (from 15th street) = 6085 people

Max. flow rate = 1500 L/c.d x 6085 = 9,127,500 L/d = 9,175.5 m^3/d = 6.34 m^3/min.

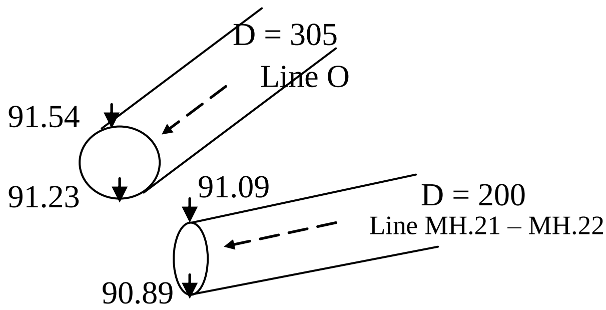
S = (93.69 – 92.99) / 87 = 0.008

At S = 0.008 and Q = 6.34 m^3/min --> D = 305 – 380 mm

At S = 0.008 and D = 380 mm --> Q = 9.8 m^3/min and V = 1.45 m/s > 0.6 (OK)

Depth of sewer at the lower end of line MH.21 – MH.22 = 93.69 – 91.09 = 2.6 m > 2.0 m (minimum cover)

Thus, we can use lower slope (say 0.004) to minimize excavation



At S = 0.004 and D = 380 mm --> Q = 6.95 m^3/min and V = 1.04 m/s

q/Q = 6.34/6.95 = 0.91 --> v/V = 1.02 --> v = 1.02 x 1.04 = 1.06 m/s > 0.6 (OK)

As sewer changes size then the crowns must be matched

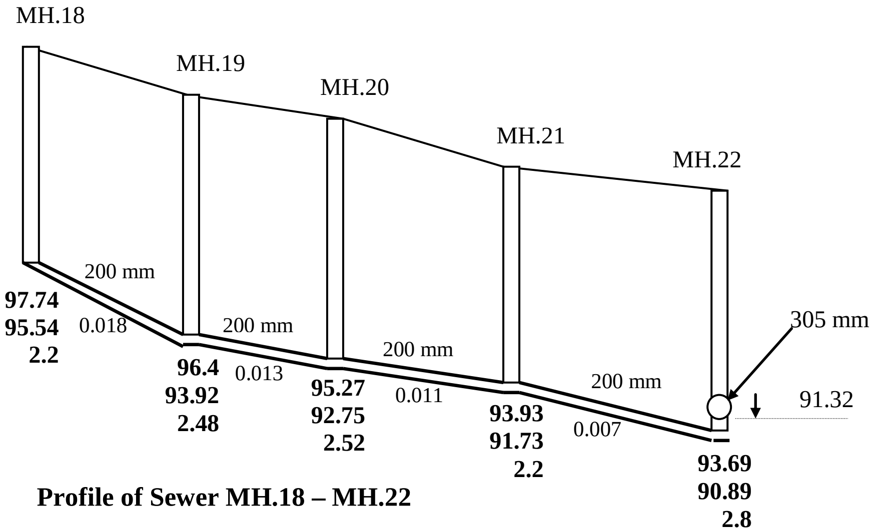
The crown of line MH.22 – MH.17 at the upper end must be at 91.09 m and its invert = 91.09 – 0.38 = 90.71 m

Thus, the invert at the lower end = 90.71 – (0.004 x 87) = 90.36 m

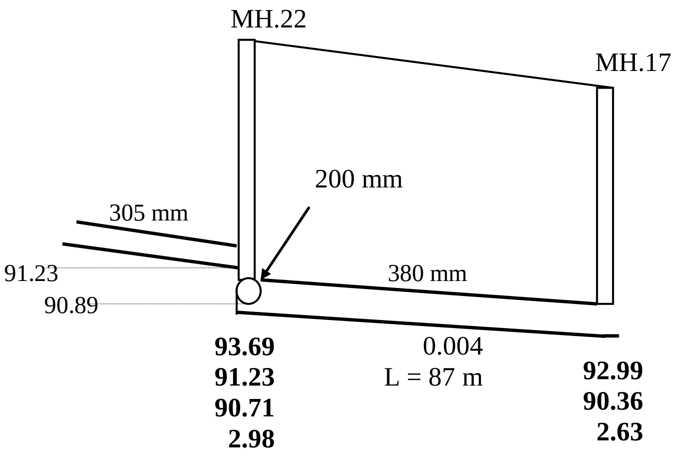
Depth of sewer at the lower end = 92.99 – (90.36 + 0.38) = 2.25 m > 2 m (OK).

Table 16-2 Design of a sanitary sewer system

Line no. (1)	To street (2)	From man-hole (3)	To man-hole (4)	Length of line, m (5)	Sewage flow, m^3/min (6)	Ground elevations: Upper man-hole (7)	Ground elevations: Lower man-hole (8)	Diam. of pipe, mm (9)	Grade of sewer (10)	Fall of sewer, m (11)	Velocity flowing full, m/sec (12)	Capacity flowing full, m^3/min (13)	Q/Q_{full} (14)	V/V_{full} (15)	V, m/sec (16)	Invert elevations: Upper man-hole (17)	Invert elevations: Lower man-hole (18)
0	15th	...	22	....	5.96	......	93.69	305	....	.....	.....	.....	.....	.....	.....	.....	91.23
1	Alley between Maple and Locust	18	19	90	0.10	97.74	96.40	200	0.0180	1.62	1.42	2.72	0.04	0.44	0.62	95.54	93.92
2	Alley between Maple and Locust	19	20	90	0.18	96.40	95.27	200	0.0130	1.17	1.20	2.30	0.08	0.53	0.64	93.92	92.75
3	Alley between Maple and Locust	20	21	90	0.25	95.27	93.93	200	0.0113	1.02	1.10	2.10	0.12	0.58	0.64	92.75	91.73
4	Alley between Maple and Locust	21	22	120	0.38	93.93	93.69	200	0.0070	0.84	0.89	1.70	0.22	0.68	0.61	91.73	90.89
5	15th	22	17	87	6.34	93.69	92.99	380	0.0040	0.35	1.04	6.95	0.91	1.02	1.06	90.71	90.36
6	Alley between Locust and Elm	13	14	90	0.10	96.04	95.37	200	0.0180	1.62	1.42	2.72	0.04	0.44	0.62	93.84	92.22
7	Alley between Locust and Elm	14	15	90	0.18	95.37	94.57	200	0.0130	1.17	1.20	2.30	0.08	0.53	0.64	92.22	91.05
8	Alley between Locust and Elm	15	16	90	0.25	94.57	93.81	200	0.0113	1.02	1.10	2.10	0.12	0.58	0.64	91.05	90.03
9	Alley between Locust and Elm	16	17	120	0.38	93.81	92.99	200	0.0070	0.84	0.89	1.70	0.22	0.68	0.61	90.03	89.14
10	15th	17	12	87	6.71	92.99	92.32	460	0.0015	0.13	0.71	6.97	0.96	1.03	0.73	88.88	88.75
11	Alley between Elm and Beech	8	9	90	0.10	94.85	94.30	200	0.0180	1.62	1.42	2.72	0.04	0.44	0.62	92.65	91.03
12	Alley between Elm and Beech	9	10	90	0.18	94.30	93.48	200	0.0130	1.17	1.20	2.30	0.08	0.53	0.64	91.03	89.86
13	Alley between Elm and Beech	10	11	90	0.25	93.48	92.90	200	0.0113	1.02	1.10	2.10	0.12	0.58	0.64	89.86	88.84
14	Alley between Elm and Beech	11	12	120	0.38	92.90	92.32	200	0.0070	0.84	0.89	1.70	0.22	0.68	0.61	88.84	88.00
15	15th	12	7	87	7.09	92.32	91.92	530	0.00092	0.08	0.62	8.18	0.87	1.02	0.63	87.67	87.59
16	Alley between Beech and Spruce	7	6	120	7.21	91.92	91.74	530	0.00092	0.11	0.62	8.18	0.88	1.02	0.63	87.56	87.45
17	Alley between Beech and Spruce	6	5	120	7.31	91.74	91.71	530	0.00092	0.11	0.62	8.18	0.89	1.02	0.63	87.45	87.34



Profile of Sewer MH.18 – MH.22



Profile of Sewer MH.22 – MH.17

Example

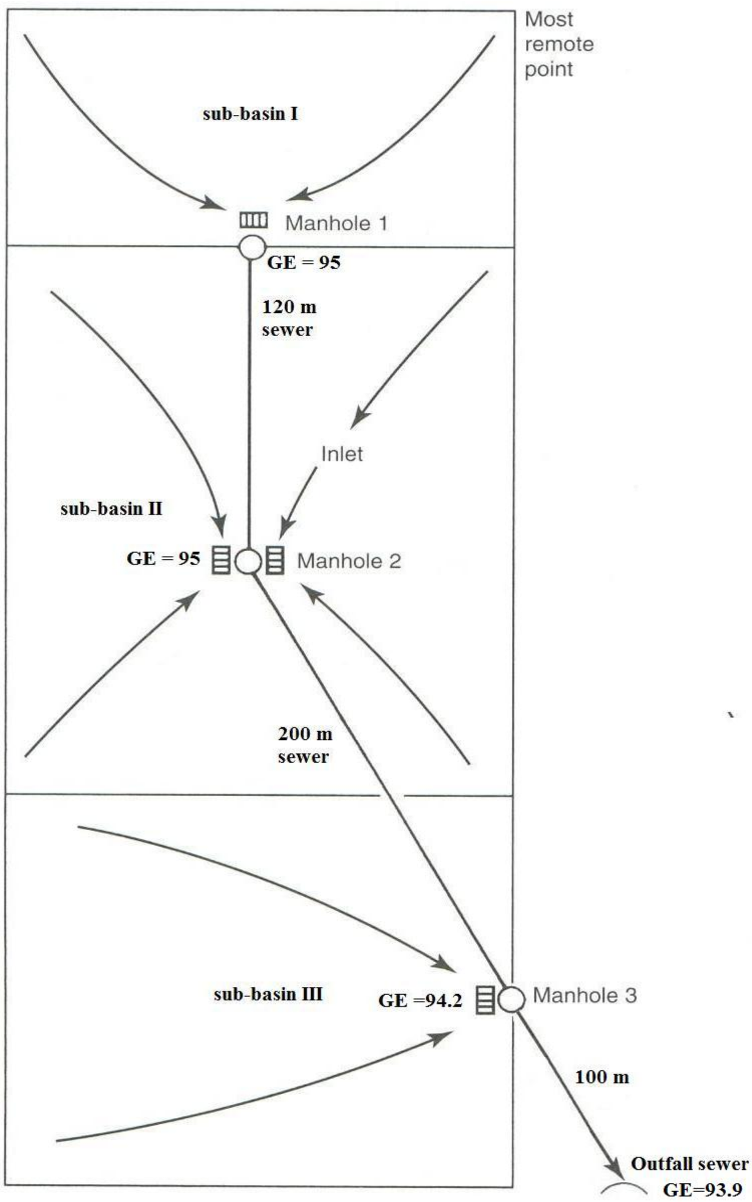
Design the stormwater drainage system for the area shown in the following figure. Draw a neat profile along the route of MH 3-MH 2-MH 1. On the profile show all pipe sizes, slopes, and ground and invert elevations.

The rainfall intensity (mm/h) for the area can be found from the following equation:

$i = 1200/(t + 45)$, where t is the rainfall duration (min)

The minimum allowable scouring velocity required to avoid the deposition of solids is 0.9 m/s and the maximum is 3 m/s. The minimum depth of the sewer below the street must be 1.5 m and coefficient of roughness, n = 0.013 is applicable.

Sub-basin	Drainage Area A (m^2)	Runoff Coefficient *C*	Inlet time (minutes)
I	7000	0.60	5.00
II	16000	0.50	12.00
III	12000	0.80	8.00



Solution

- **Line 1 (MH1-MH2) (**collects storm water from sub-basin I**)**

  A = 7000 m^2, t_c = 5 min

  i = 1200 / (t + 45) = 24 mm/h

  q = C i A = 0.6 x (24 /1000x60) x 7000 = 1.68 m^3/min

  Ground elevations: 95 and 95 m

  From nomograph at Q = 1.68 m^3/min and V_{min} = 0.75 m/s → D = 200 – 255 mm

  At D = 255 mm and S = 0.004 → Q = 2.4 m^3/min and V = 0.8 m/s

  q/Q = 1.68/2.4 = 0.70 → v/V = 0.95 → v = 0.95 (0.80)=0.76 m/s > 0.75 m/s (OK)

  Invert level at the upper end = 95 – 1.5 – 0.255 = 93.245m

  Invert level at the lower end = 93.245 – 0.004 x 120 = 92.765 m

- **Line 2 (MH2-MH3) (**collects storm water from sub-basin II and sub-basin I**)**

  Area contributing flow to line 2 = 7,000 + 16,000 = 23,000 m^2

  t_c = max. of :

  [12 min]

  [5 + (120 m / 0.76 m/s x 60 s/min)] = [5 + 2.63] = [7.63 min]

  i = 1200 / (12 +45) = 21.05 mm/h

  Ceff = (0.50 x 16000+0.6x7000)/23000 = 0.53

  q = C i A = 0.53 x 21.05/(1000x60) x 23,000 = 4.28 m^3/min

  Ground elevations: 95 m and 94.2 m

  S= (95-494.2)/200 = 0.004

  From nomograph at Q = 4.30 m^3/min and S =0.004 → D = 350 – 380 mm

  At D = 380 mm and S = 0.004 → Q = 7.0 m^3/min and V = 0.92 m/s

  q/Q = 4.28/7.0 = 0.62 → v/V = 0.9 → v = 0.9(0.92)=0.83 m/s > 0.75 m/s (OK)

  Invert at upper end = (92.765 + 0.255) – 0.380 = 92.64 m

  Invert at lower end = 92.64 – (0.004 x 200) = 91.84 m

  - **Line 3 (MH3 – outfall)**

Area contributing flow to line 3 = 7,000 + 16000 + 12,000 = 35,000 ^{m2}

t_c = max. of :

[5 + (120/0.76x60)+(200/0.83x60)] = [5 + 6.65] = [11.65 min]

12 + (200/0.83x60)= 16.02 min

8 min

i = 1200 / (16.01 +45) = 19.67 mm/h

Ceff = (0.60 x 7000+0.50 x 16000+.8x13000)/35000 = 0.646

q = C i A = 0.646 x (19.67/1000x60) x 35,000 = 7.41 m^3/min

Ground elevations: 94.2 m and 93.9 m

S = (94.2 – 93.9)/100 = 0.003

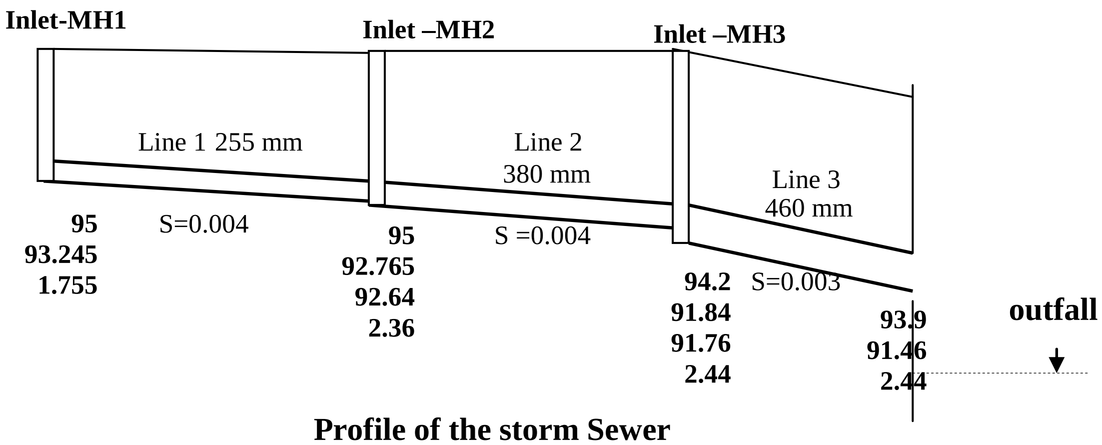
From nomograph, at S = 0.003 and Q = 7.41 m^3/min → D = 380 – 460 mm.

At D = 460 mm and S =0.003 → Q = 9.5 m^3/min and V = 1.0 m/s

q/Q = 7.41/9.5 = 0.78 → v/V = 0.98 → v = 0.98 m/s > 0.75 (OK)

Invert level at the upper end = (91.84 + 0.38) – 0.46 = 91.76 m

Invert level at the lower end = 91.76 – 0.003 x 100 = 91.46 m



Profile of the storm Sewer