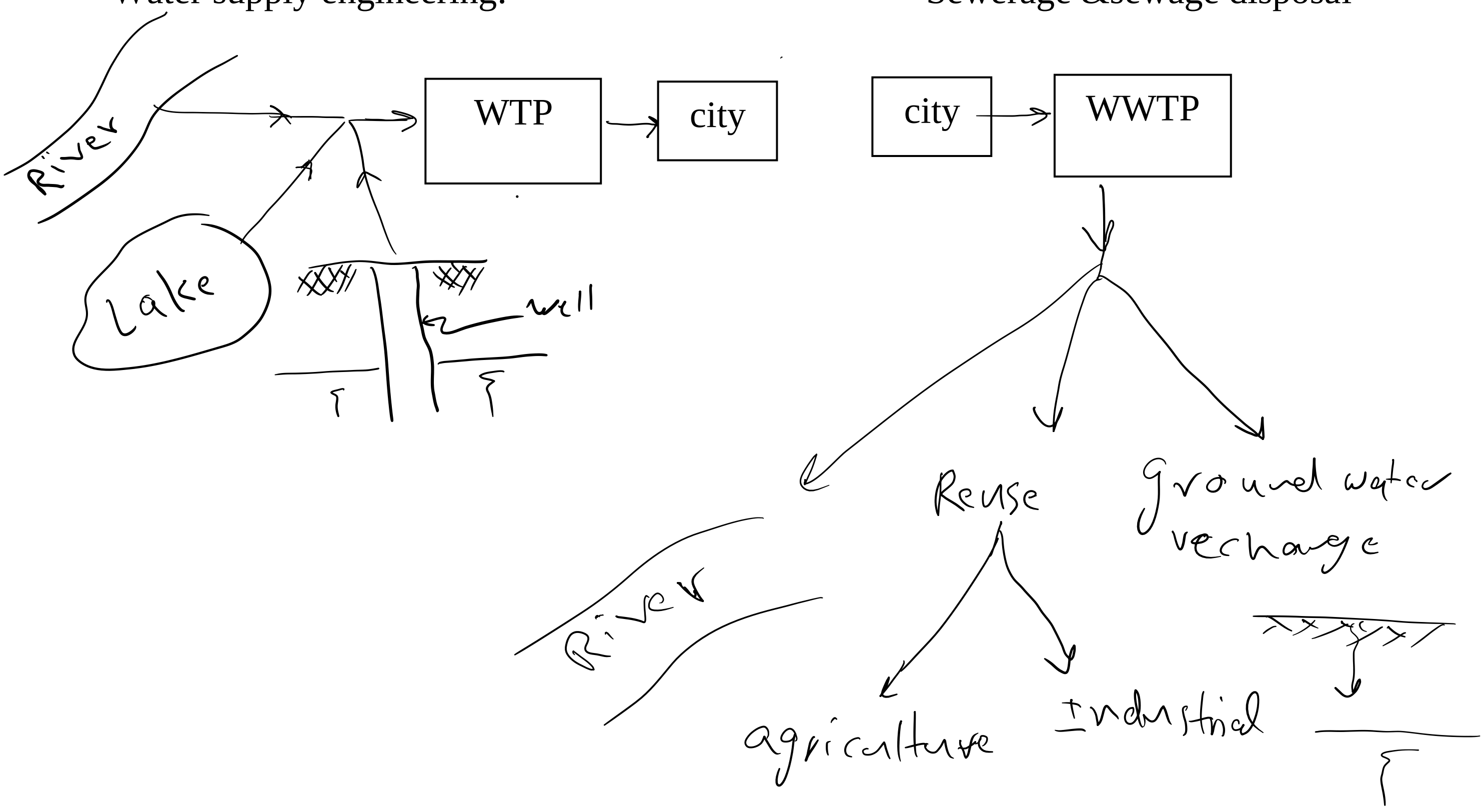
Water supply & waste water networks

Sanitary Engineering (municipal Engineering)

Water supply engineering.

Sewerage &sewage disposal



Components of water Distribution System

- pumping station
- fire hydrant
- Distribution piping
  - pipes
  - valves
- Storage
  - tanks
  - pipes
  - valves

Water supply Distribution system

Flow in Pipes

1. ***Darcy-weisbach equation***

$h_L = \frac{fL}{d} \cdot \frac{v^2}{2g}$

$h_L = head\ looss$

f= resistance coefficient

L= length of pipe

D= diameter of pipe

V= velocity in pipe

g= gravitational acceleration

2. ***Chezy's formula***

$$v^2 = \frac{8g}{f} Rs$$

where $C^2 = \frac{8\,g}{f}$ ….C= chezy Coefficient

$v^2 = C^2 R\,S$ or $v = C\sqrt{RS}$

V= velocity in the pipe in m/sec

R= the hydraulic Radius of the pipe in **m**

$$R = \frac{A}{\rho} \ldots A\left(area \frac{\pi}{4} D^2\right) \ldots\ldots \rho = wetted\ perimeter(\pi D)$$

S= the hydraulic gradient

3. ***Manning Formula***

$$v = \frac{1}{n} R^{\frac{2}{3}} S^{\frac{1}{2}}$$

$n = roughness\ coefficient$

4. ***Hazen-William formula***

$v = kCR^{0.63} S^{0.54}$

C= a constant depending upon the relative roughness of the pipe (C=100 for old cast iron)

K= is an experimental Coefficient and unit conservation equal to (0.849) SI units

The hazen-Williams Diagram conducted for C=100 , however

If we use Hazen-Williams Diagram for pipes with $C \neq 100 \ and$

$given$ Q &D

Find $S_c=S_{100}\ (100/C)^{1.85}$

$given\ Q\ \&\ S$

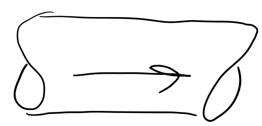
Find $D_c= D_{100}(100/C)^{0.38}$

$given\ D\ \&\ S$

$Q_c= Q_{100} * C/100$

Example : find maximum flow use Darcy, Chezy, Manning & Hazen-William Formulas

hL=3.5 m, L= 200m, D= 317mm, C=150, f=0.0125, n=0.009



solution:

Darcy-weisbach equation

$$h_L = \frac{fL}{d} \cdot \frac{v^2}{2g}$$

$3.5= 0.0125*200*v^2 /2*9.81*0.317$

V=2.95 m/s

$Q=V*A=2.95*(\pi/4)*0.317^2 =0.233m^3/sec$

Chezy's formula

$$v = C\sqrt{RS}$$

$$C = \sqrt{\frac{8*9.81}{0.0125}} = 79.24$$

R= $R = \frac{A}{\rho} = \frac{d}{4} = \frac{0.317}{4} = 0.07925m$

$S=h_L/L=3.5/200=0.0175$

$$v = 79.24\sqrt{0.07925 * 0.0175} = 2.95\ m/s$$

$Q=2.95*0.317^2 *\pi/4= 0.233\ m^3/s$

Manning Formula

$$v = \frac{1}{n} R^{\frac{2}{3}} S^{\frac{1}{2}}$$

$$v = \frac{1}{0.009}\left(\frac{0\cdot 317}{4}\right)^{\frac{2}{3}}\left(\frac{3.5}{200}\right)^{0\cdot 5} = \frac{2.7m}{s}$$

$Q=2.7*0.317^2 *\pi/4=0.212 \frac{m^3}{s}$

Hazen-William formula

$$v = kCR^{0.63}S^{0\cdot 54}$$

$$0.849 * 150 * \left(\frac{0.317}{4}\right)^{0.63} * \left(\frac{3.5}{200}\right)^{0.54} = 2.9\frac{m}{s}$$

$Q=2.9* 0.317^2 *\pi/4= 0.228\ m^3/s$

Example: Consider a 200mm pipe 1500m in length which carries a flow of 2m^3/min, find the velocity and head loss in pipe (C=100) use Hazen-Williams chart.

Solution :

Q= 2m^3/ min=0.033m^3/s

*From chart v=1.05m/s , S=10*10^{-3}*

أي ان فواقد الشحنة تساوي 10 متر لكل 1000 متر

$h_l = 10*10^{-3} *1500=15m =150\ kpa$

Example: for the previous example find head loss if C=120

$S_{120} =S_{100} *(100/120)^{1.85} =7.13*10^{-3}$

$h_l =7.13*10^{-3} *1500=10.7m= 107\ kpa$

(note : 10.19 m H_2O =100 kpa)

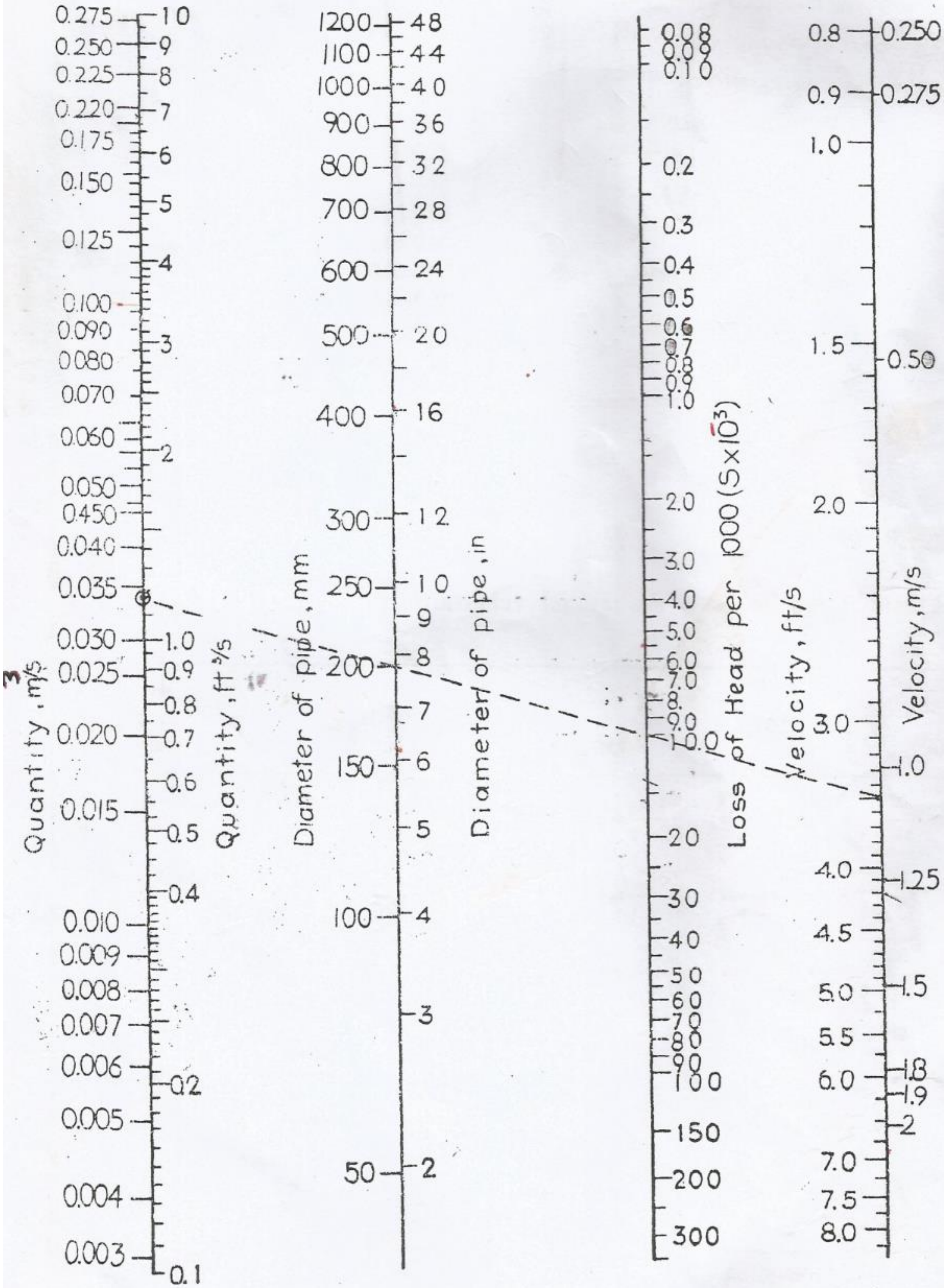


Figure 6-8 Flow in old cast-iron pipes. (Hazen-Williams $C = 100$.)

$1 \frac{ft^3}{sec} = 0.0283 \frac{m^3}{sec}$

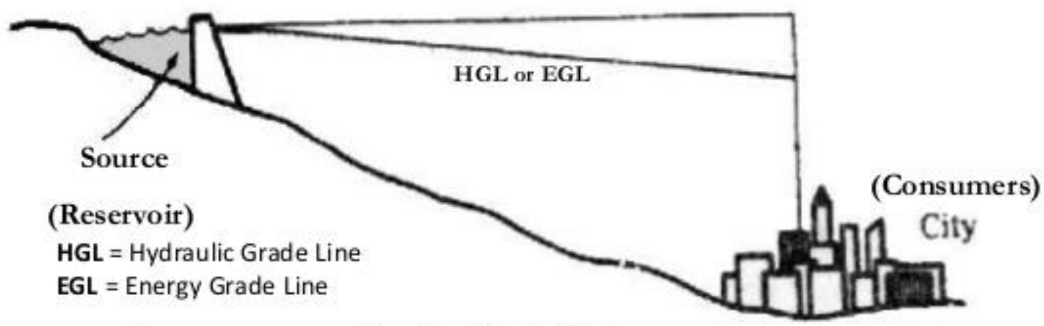
Method of Water distribution

1. *Gravitational distribution*

تستخدم هذه الطريقة عندما يكون موقع المصدر المائي في مكان مرتفع بحيث يوفر ضغط كافي في الانابيب الرئيسة للاستخدامات المختلفة والحريق.

Various methods of water supply distribution systems in a town adapted are;

1. Gravity system: The source of supply is at a sufficient elevation above the distribution area (i.e. consumers). So that the desired pressure can be maintained.



Gravity-Supply System

Advantages of Gravity supply:

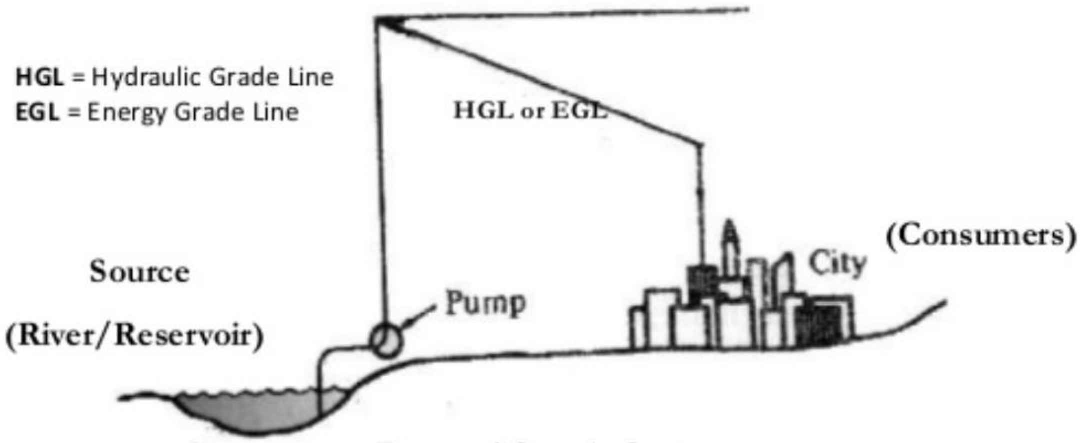
- No energy costs
- Simple operation
- Low maintenance costs
- No sudden pressure changes

2. *Direct pumping (pumping without storage)*

من خلال هذه الطريقة يتم ضخ الماء مباشرة الى انابيب الشبكة ومن مساويء هذه الطريقة:

- اي انقطاع للتيار الكهربائي عن المضخات يؤدي الى انقطاع الماء عن الشبكة
- الضغط في الشبكة يكون غير منتظم اذ يتغير مع تغير الاستهلاك لذا يتم وضع مضخات بسعات مختلفة تعمل تبعا لكمية الماء المسحوبة
- تغير ضغط الماء يؤدي الى التلف السريع للانابيب والملحقات الاخرى

- The source of water is lower than the area
- The source cannot maintain minimum pressure required.
- Pumps are used to develop the necessary head (pressure)

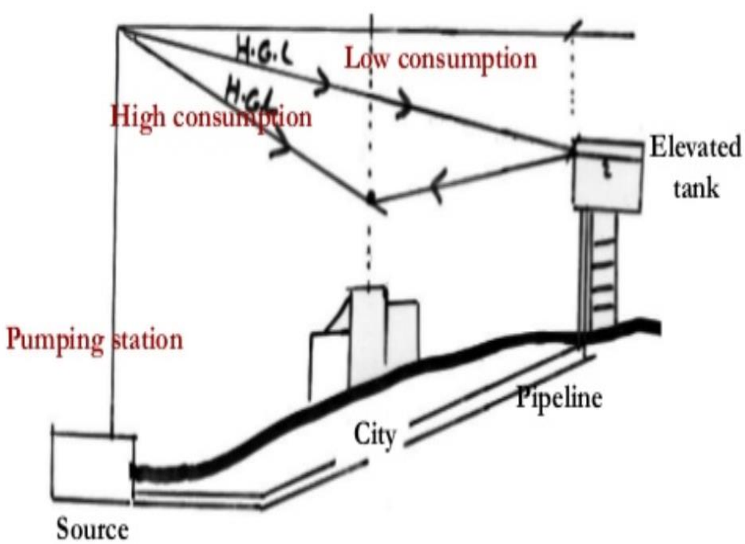


Pumped-Supply System

Disadvantages of pumped supply:

- Complicated operation
- Requires maintenance
- Dependent on reliable power supply

3. *Pumping with storage*



في هذه الطريقة يتم ضخ الماء بواسطة المضخات ويخزن الماء الزائد خلال فترات الاستهلاك الواطئ في خزانات عالية، اذ يتم استغلاله في فترات الاستهلاك العالي والعجز لتعزيز الماء الذي يضخ بواسطة المضخات

محاسن هذه الطريقة

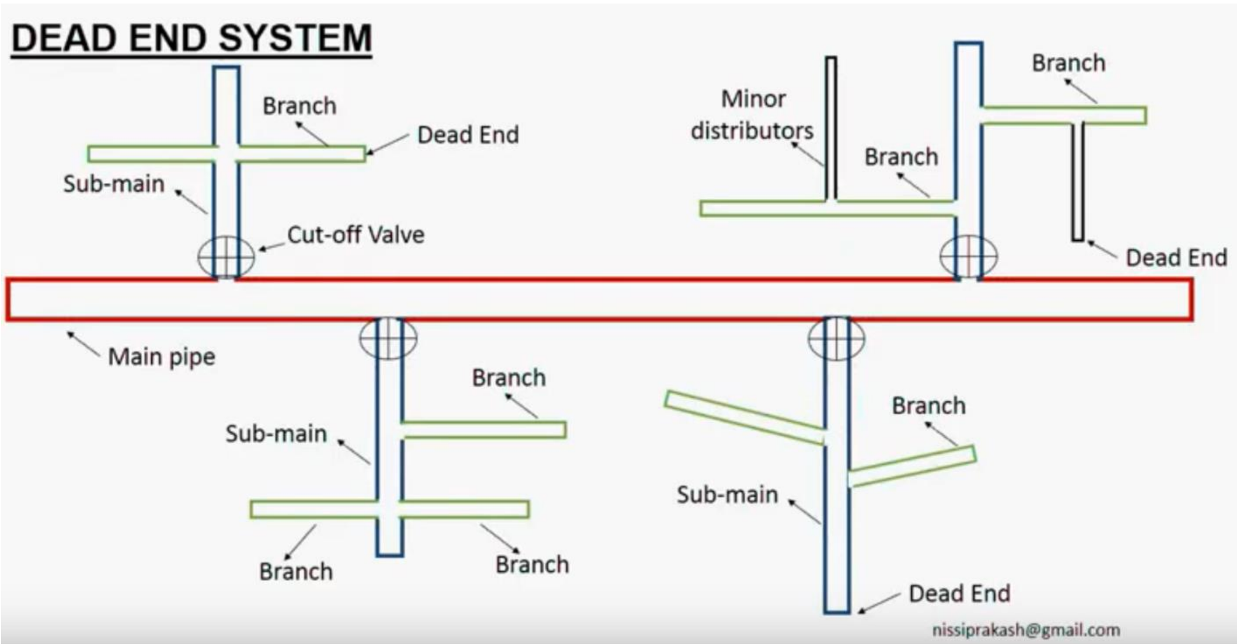
- وتساعد هذه الطريقة على جعل معدلات الضخ منتظمة ،
- كما يقل احتمال تضرر الانابيب وملحقاتها بسبب تغير الضغط في الشبكة حيث ان الضغط في الشبكة سيكون منتظم تقريبا
- يمثل الماء في الخزان مدخرا لأي طاريء مثل الحرائق وانقطاع التيار الكهربائي عن المضخات.

Patterns of water supply distribution system

يوجد عدد من الاشكال التي تتخذها شبكة اسالة الماء من اهم هذه الاشكال اوالأنواع الرئيسة من هذه الاشكال

1. Branching pattern with dead ends

هذا النظام او الشكل يشبه تفرعات الشجرة ويتكون من الخط الاساسي والذي يتفرع الى الخطوط الرئيسة ثم الخطوط الثانوية والتي تغذي المياه للابنية



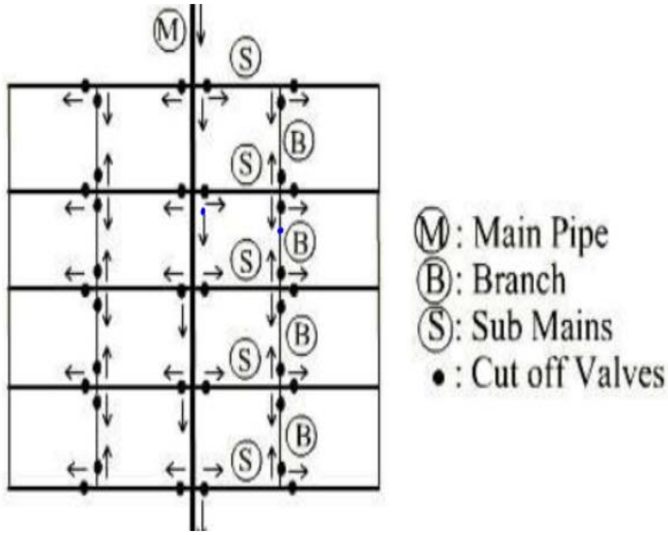
وفي هذه الشبكات يكون اتجاه جريان الماء في اتجاه واحد والماء يجهز للمنطقة بواسطة خط متفرد ومن محاسن هذه الطريقة

1. طريقة بسيطة جدا لتوزيع الماء
2. تصميم مثل هذه الشبكة سهل
3. اكثر اقتصادية من الشبكات الاخرى

مساويء هذه الطريقة :

1. تجمع الترسبات في النهايات الميتة مما يؤدي الى تولد الروائح والطعم
2. حدوث كسر في احد اجزاء الشبكة يؤدي انقطاع الماء عن الاجزاء اللاحقة
3. عدم تساوي توزيع الضغط في الشبكة او عدم كفاية الضغط عند ربط مناطق اضافية في الشبكة

2. Grid pattern



في هذا النظام جميع الانابيب تكون مرتبطة مع بعضها البعض بشكل شبكة من كلتا نهايتي كل انبوب والماء يصل لأي نقطة في الشبكة من اكثر من اتجاه واحد

محاسن هذه الطريقة:

1. الماء في منظومة الاسالة له حرية الحركة من اكثر من اتجاه واحد
2. توزيع الضغط بانتظام خلال الشبكة
3. في حالة حدوث كسر في احد الانابيب لاينقطع الماء عن بقية الانابيب

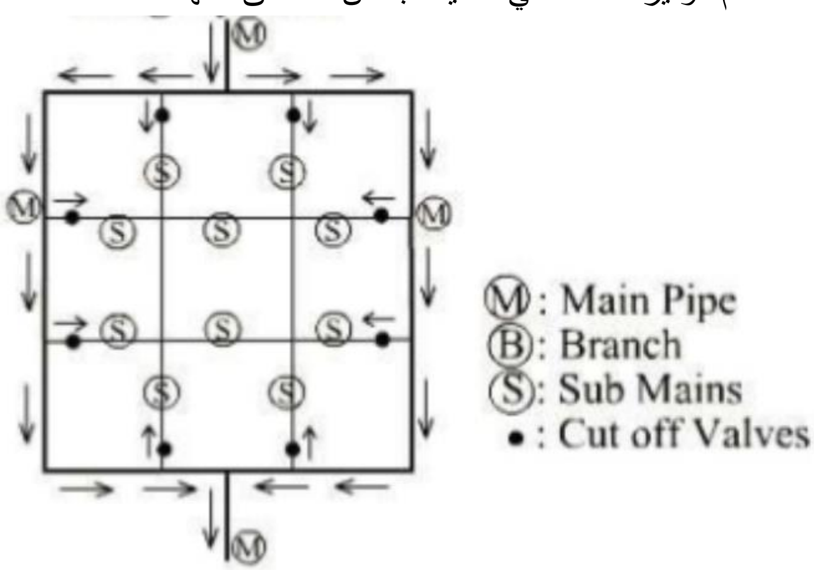
مساويء هذه الطريقة

1. تصميم هذه الشبكة اكثر تعقيدا
2. الكلفة العالية نتيجة اطوال الانابيب والوصلات اللازمة

3. Grid pattern with loop

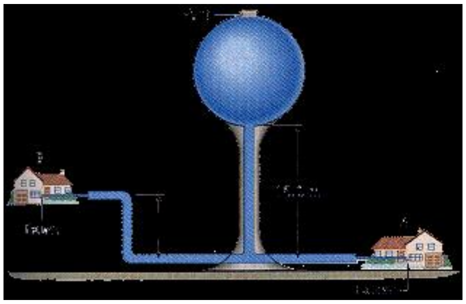
يمكن توفير الحلقات في هذا النظام لتوفير الضغط في المدينة لبعض المناطق المهمة فيها مثل صناعة معينة

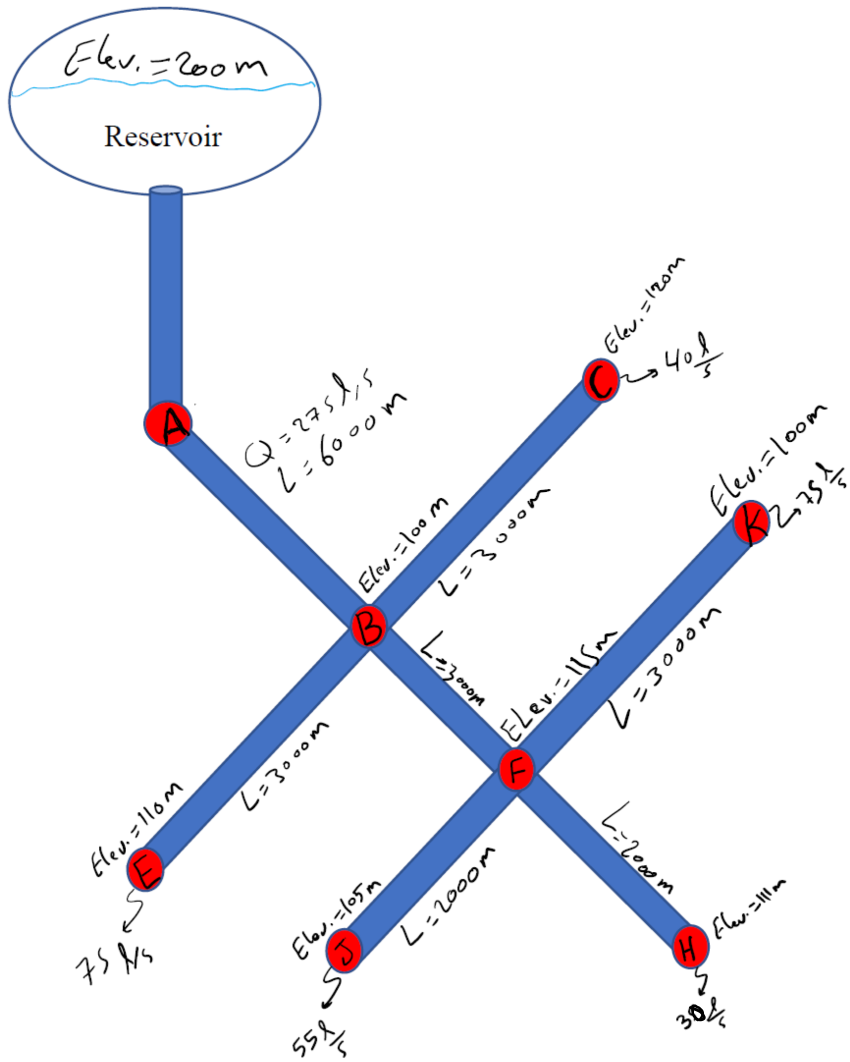
او منطقة تجارية



Design of Dead End System

Ex: for the network show Design the pipes AB, BE, BC & FH assume required pressure at each end points 250 kpa . assume (100kpa =10 m H_2O)





$$\boxed{V = k\,C\,R^{0.63}\,S^{0.54}}$$ → Hazen-william eq.

$$\boxed{d = \frac{Q^{0.38} \times 0.282}{S^{0.205}}}$$

$$\boxed{h_L = \frac{Q^{1.854} \times 2.03 \times 10^{-3} \times L}{d^{4.878}}}$$

pipe BC

Required head at point C = 120 + 25 = 145 m

allowable h_L at pipe BC = available head at point B − required head at point C

$= 167.3 - 145 = 22.3\ m$

$$S = \frac{h_L}{L} = \frac{22.3}{3000} = 7.43 \times 10^{-3}$$

$$Q = 40\ l/s = 0.04\ \frac{m^3}{s}$$

By eq:

$$d = \frac{0.04^{0.38} \times 0.282}{0.00743^{0.205}} = 0.226\ m$$

$$= 226\ mm$$

we take $\boxed{d = 250\ mm}$

$$h_L = \frac{0.04^{1.854} \times 2.03 \times 10^{-3} \times 3000}{0.25^{4.878}} = 13.5\ m$$

available head at point C = 167.3 − 13.5 = 153.8 m > 145 m

المتوفرة B

الضياع بالطاقة الحاصل بالأنبوب BC .

for pipe Bf

required head at point f = 115 + 25 = 140 m

allowable head loss at pipe Bf = 167.3 − 140 = 27.3 m

$S = \frac{27.3}{3000} = 9.1 \times 10^{-3}$ $Q = 0.16 \frac{m^3}{s}$

$d = \frac{0.16^{0.38} \times 0.282}{0.0091^{0.205}} = 0.368 m = 368 mm$

∴ we take $\boxed{d = 400 mm}$

$h_L = \frac{0.16^{1.854} \times 2.03 \times 10^{-3} \times 3000}{0.4^{4.878}} = 17.8 m$

available head at point f = 167.3 − 17.8
= 149.5 m > 140 m
∴ O.K

for pipe f H

required head at point H = 111 + 25 = 136 m

allowable head loss at pipe f H = 149.5 − 136 = 13.5 m

$S = \frac{13.5}{2000} = 6.76 \times 10^{-3}$ $Q = 0.03 \frac{m^3}{s}$

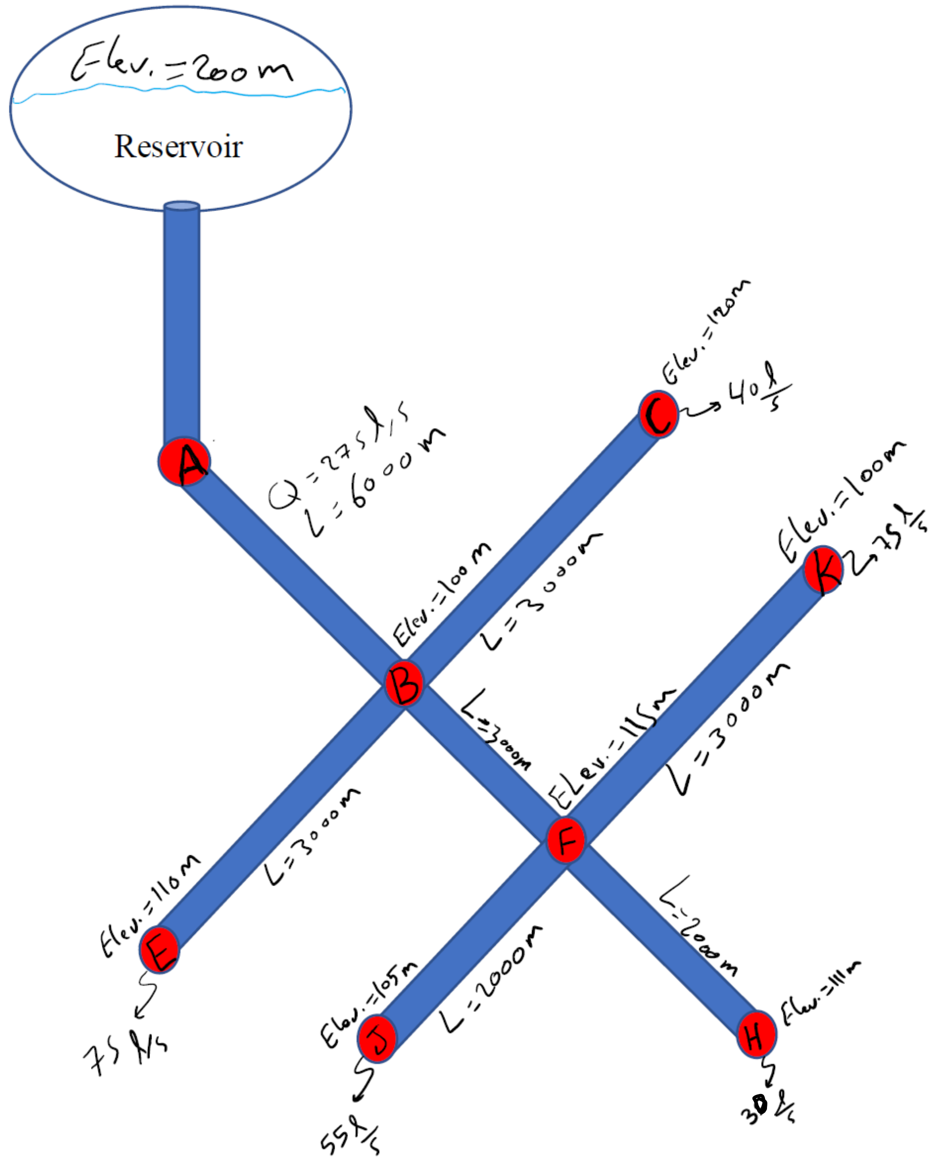
$d = \frac{0.03^{0.38} \times 0.282}{0.0067^{0.205}} = 0.207 m = 207 mm$

use d = 250 mm → $h_L = \frac{0.03^{1.854} \times 2.03 \times 10^{-3}}{0.25^{4.878}} \times 2000 = 5.27 m$

available head at point H = 149.5 − 5.27 = 144.23 m > 136 m
∴ O.K

Design of Dead End System

Ex: for the network show Design the pipes AB, BE, BC & FH assume required pressure at each end points 250 kpa . assume (100kpa =10 m H_2O)



Solution:

In any node… $\sum Q_{in} = \sum Q_{out}$.

As 100 kpa=10 m H_2O then 250 kpa= 25 m

Required head at any point =required pressure (m)[given]+Elevation of the point

Allowable head loss at pipe= available head at start point – required head at end point

Pipe AB

Q= 275 L/s= 0.275 m^3/s

L= 6000m

Point B Elevation = 100m

:. the required head at point B = 25 + 100= 125 m

Available head at point A= 200m

Allowable head loss at pipe AB=200-125= 75 m

$:. s = \frac{75}{6000} = 12{,}5^{*}10^{-3}$

From Hazen-Williams nomogram

d~(between 400 -500 mm)

we take the greater Diameter ….So we take d= 500mm

then we correct the hydraulic gradient S & h_L

:. $S_{actual} = 5.5 * 10^{-3}$ then….. $h_{L\ actual} = S*10^{-3} * L = 5.5 * 10^{-3} * 6000 = 33m$

:. Available head at point B=200-33=167m

Available head at any point **must be** > required head at the same point

Pipe BE

The required head at point E = 110+ 25= 135 m

Available head at point B= 167m

Allowable head loss at pipe BE =167-135= 32 m

:. $s = \frac{hl}{l} = \frac{32}{3000} = 10.7^{*}10^{-3}$ & Q=0.075m^3/s

From Hazen-Williams nomogram

d~(between 250 -300 mm)

we take the greater Diameter ….So we take d= 300mm

then we correct the hydraulic gradient S & h_L

:. S_{actual} =6.5 * 10^{-3} then….. $h_{L\ actual}$=S*10^{-3} * L=6.5 * 10^{-3} * 3000= 19.5m

:. Available head at point E=167-19.5=147.5 m

Available head at E=147.5m > required head at E=135m :. O.K

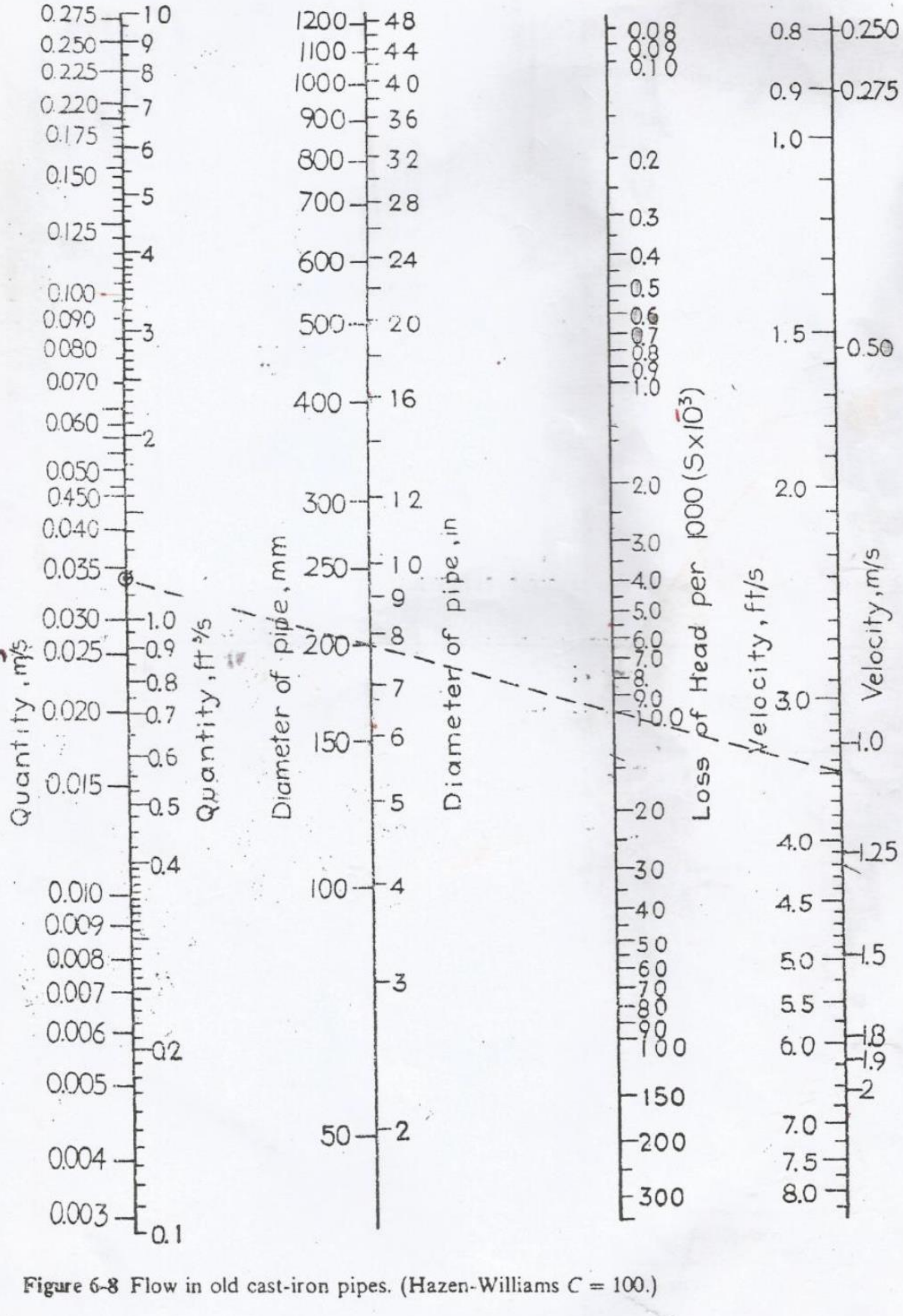


Figure 6-8 Flow in old cast-iron pipes. (Hazen-Williams $C = 100$.)

$1 \frac{ft^3}{sec} = 0.0283 \frac{m^3}{sec}$

7-6 The Pipe System

The network of pipes which makes up the distribution system may be subdivided into primary or arterial lines, secondary lines and small distribution mains.

The *primary or arterial mains* form the basic structure of the system and carry flow from the pumping station to and from elevated storage tanks and to the various districts of the city. These lines are laid out in interlocking loops with the mains not more than 1 km (3000 ft) apart. Looping assures continuous service even if a portion of the system is shut down for repairs and provides flow from two directions for fire demand. The arterials should be valved at intervals of not more than 1.5 km (1 mi) and all smaller lines connecting to them should be valved so that failure in the smaller lines does not require shutting off the larger. Large primary mains should be provided with blowoff valves at low points and with air and vacuum relief valves at high points.

The *secondary lines* form smaller loops within the primary mains and run from one primary line to another. They are located at spacings of two to four blocks and thus serve to provide large amounts of water for fire fighting without excessive pressure loss.

The *small distribution mains* form a grid over the entire service area—supplying water to every user and to the fire hydrants. They are connected to primary, secondary, or other small mains at both ends and are valved so that the system can be shut down for repairs without depriving a large area of water. The size of the small mains is generally dictated by fire flow except in residential areas with very large lots.

Velocities at maximum flow, including fire flow, normally do not exceed 1 m/s (3 ft/s), with an upper limit of 2 m/s (6 ft/s), which may occur in the immediate vicinity of large fires. The size of the small distribution mains is seldom less than 150 mm (6 in) with cross mains located at intervals of not more than 180 m (600 ft). In high-value districts the minimum size is 200 mm (8 in), with cross-mains at the same maximum spacing. Major streets are provided with lines not less than 305 mm (12 in) in diameter.

Lines which provide only domestic flow may be as small as 100 mm (4 in) but should not exceed 400 m (1300 ft) in length if dead-ended or 600 m (2000 ft) if connected to the system at both ends. Lines as small as 50 and 75 mm (2 and 3 in) are sometimes used in small communities. The length of such lines should not exceed 100 m (300 ft) if dead ended and 200 m (600 ft) if connected at both ends. Dead ends should be avoided whenever possible, since the supply is less certain and the lack of flow in such lines may contribute to water quality problems.

7-7 Design of Water Distribution Systems

The detailed design of a water distribution system is affected by local topography, existing and expected population densities, and commercial and industrial demand. First, the flow must be disaggregated to individual subareas of the system as described in Art. 7-4. Next, a system of interlocking loops must be laid out as described in Art. 7-6. The disaggregated flows are then assigned to the various nodes of the system. The design then involves determination of the sizes of the arterials, secondary lines, and small distribution mains required to ensure that the pressures and velocities desired in the system are maintained under a variety of design flow conditions. These design conditions are based on the maximum daily flow rate plus one or more fires, depending on the size of the community. The fire flow rate depends upon the character of the individual subarea as discussed in Arts. 2-6 and 7-4. In general, those fire locations which are most distant either vertically or horizontally from the pumping plant will be critical for design; however, it is usually necessary to assume various fire locations in order to ensure that all areas are adequately protected.

Consideration of the design problem described above leads to the obvious conclusion that, in general, there are many possible solutions which will satisfy the design constraints. The task then becomes determining the "best" solution. Such an optimization problem for a looped pipe network is very complicated since the distribution of flows in the pipes is a function of the design, hence simplified techniques are often used.[4] Even so, the optima for the various design conditions will not be the same, so that the final design which satisfies all required conditions may not be an optimum for any particular condition.

The usual engineering approach to design of looped pipe systems involves layout of the network as described in Art. 7-6, assignment of estimated pipe sizes (perhaps the minima of Art. 7-6), and calculation of resulting flows and head losses. The pipe sizes are then adjusted as necessary to ensure that the pressures at the various nodes and the velocities in the various pipes meet the criteria established for the community. For a given set of pipe sizes, the calculation of flows and pressures is normally a reasonably straightforward task which can be performed in a variety of ways.

The *Hardy Cross* method[5] and its modifications have been used in design and analysis of water distribution systems for many years. The method is based upon the hydraulic formulas of Chap. 3, which are used to calculate the energy losses in the elements of the system. It is not unusual to neglect the losses in fittings, since these will be small with respect to those in long pipes. The energy loss in any element of the system may be expressed as

$$h_i = k_i Q_i^x \tag{7-1}$$

where h_i = energy loss in element i

Q_i = flow in that element

k_i = constant depending on pipe diameter, length, type, and condition

x = 1.85 to 2 normally, depending on equation used

For any pipe in a loop of the system, the actual flow will differ from an assumed flow by an amount Δ:

$$Q_i = Q_{i0} + \Delta \tag{7-2}$$

where Q_i = actual flow in pipe
Q_{i0} = assumed flow
Δ = required correction

Substituting Eq. (7-2) in (7-1) gives

$$k_iQ_i^x = k_i[Q_{i0}^x + xQ_{i0}^{(x-1)}\Delta + \cdots] \tag{7-3}$$

The remaining terms in the expansion may be neglected if Δ is small compared to Q_i. For any loop, the sum of the head losses about the loop must be equal to zero. This statement is mathematically equivalent to saying there is only one pressure at any point. Thus, for any loop,

$$\sum_1^n k_iQ_i^x = 0 \tag{7-4}$$

where n is the number of pipes in the loop. Then, from Eq. (7-3),

$$\sum_1^n k_iQ_i^x = \sum_1^n k_iQ_{i0}^x + \sum_1^n xk_iQ_{i0}^{(x-1)}\Delta = 0 \tag{7-5}$$

Equation (7-5) may then be solved for the correction:

$$\Delta = -\frac{\sum_1^n k_iQ_{i0}^x}{\sum_1^n xk_iQ_{i0}^{(x-1)}} = -\frac{\sum_1^n h_i}{x\sum_1^n h_i/Q_{i0}} \tag{7-6}$$

The procedure may be outlined as follows:

1. Disaggregate the flow to the various blocks or other subareas of the community.
2. Concentrate the disaggregated flows at the nodes of the system.
3. Add the required fire flow at appropriate nodes.
4. Select initial pipe sizes using the criteria of Art. 7-6.
5. Assume any internally consistent distribution of flow. The sum of the flows entering and leaving each node must be equal to zero.
6. Compute the head loss in each element of the system. Conventionally, clockwise flows are positive and produce positive head loss.
7. With due attention to sign, compute the total head loss around each loop:

$$\sum_i^n h_i = \sum_1^n k_iQ_{i0}^x$$

8. Compute, without regard to sign, the sum

$$\sum_1^n k_iQ_{i0}^{x-1}$$

9. Calculate the correction for each loop from Eq. (7-6) and apply the correction to each line in the loop. Lines common to two loops receive two corrections with due attention to sign.
10. Repeat the procedure until the corrections calculated in step 9 are less than some stipulated maximum. The flows and pressures in the initial network are then known.
11. Compare the pressures and velocities in the balanced network to the criteria of Arts. 7-5 and 7-6. Adjust the pipe sizes to reduce or increase velocities and pressures and repeat the procedure until a satisfactory solution is obtained.
12. Apply any other fire flow conditions which may be critical and reevaluate the velocities and pressure distribution. Adjust the pipe sizes as necessary.

The procedure outlined above is almost always performed on a computer. A simple example using tabular calculations is presented here in order to aid in understanding the technique.

Example 7-1 Figure 7-9 represents a simplified pipe network. Flows for the area have been disaggregated to the nodes, and a major fire flow has been added at node *G*. The water enters the system at node *A*. Pipe diameters are based on the flows and the criteria discussed above. The calculations are tabulated in Tables 7-2 through 7-4, and the corrected flows after each iteration are shown on Fig. 7-10. The calculations are continued in this example until the corrections are less than 0.2 m^3/min (50 gal/min).

The network is divided into the loops *ABHI*, *BEFGH*, and *BCDE*. Any other system might be used (*ABCDEFGHI*, *ABHI*, and *BCDE*, for example), provided all lines are included in at least one loop. In Table 7-2 the pipe identification, assumed flows, length, and diameter are listed in the first four columns. The slope of the hydraulic grade line (in this case calculated from the Hazen-Williams equation with $C = 100$) is tabulated in the fifth column. The head loss in each line is the product of s and the length and is positive or negative depending on the direction of the flow in each line.

Columns 6 and 7 are summed and the correction calculated as shown. Note that the flows in lines common to two loops are positive in one loop and negative in the other. The calculated corrections are applied, with attention to sign, to the flows in each loop. Lines common to two loops receive both corrections. The corrected flows entered in Table 7-3 are then reanalyzed in the same fashion to yield a second set of corrections. The iteration in Table 7-4 gives corrections which are equal to or less than the stipulated maximum. The last corrections are applied to yield the final flows of Fig. 7-10.

The balanced network must then be reviewed to assure that the velocity and pressure criteria are satisfied. The velocities vary from 2.13 m/s (7 ft/s) in line *AB* to 0.22 m/s (0.72 ft/s) in line *ED*. The velocities in lines *AI*, *IH*, *AB*, *BE*, and *EF* exceed the criteria suggested above, and these lines might be increased in diameter. The pressure drop from node *A* to node *G* is 49 m of water or 480 kPa. If the pressure at node *A* were 500 kPa (Art. 7-5), the pressure at node *G* would be only

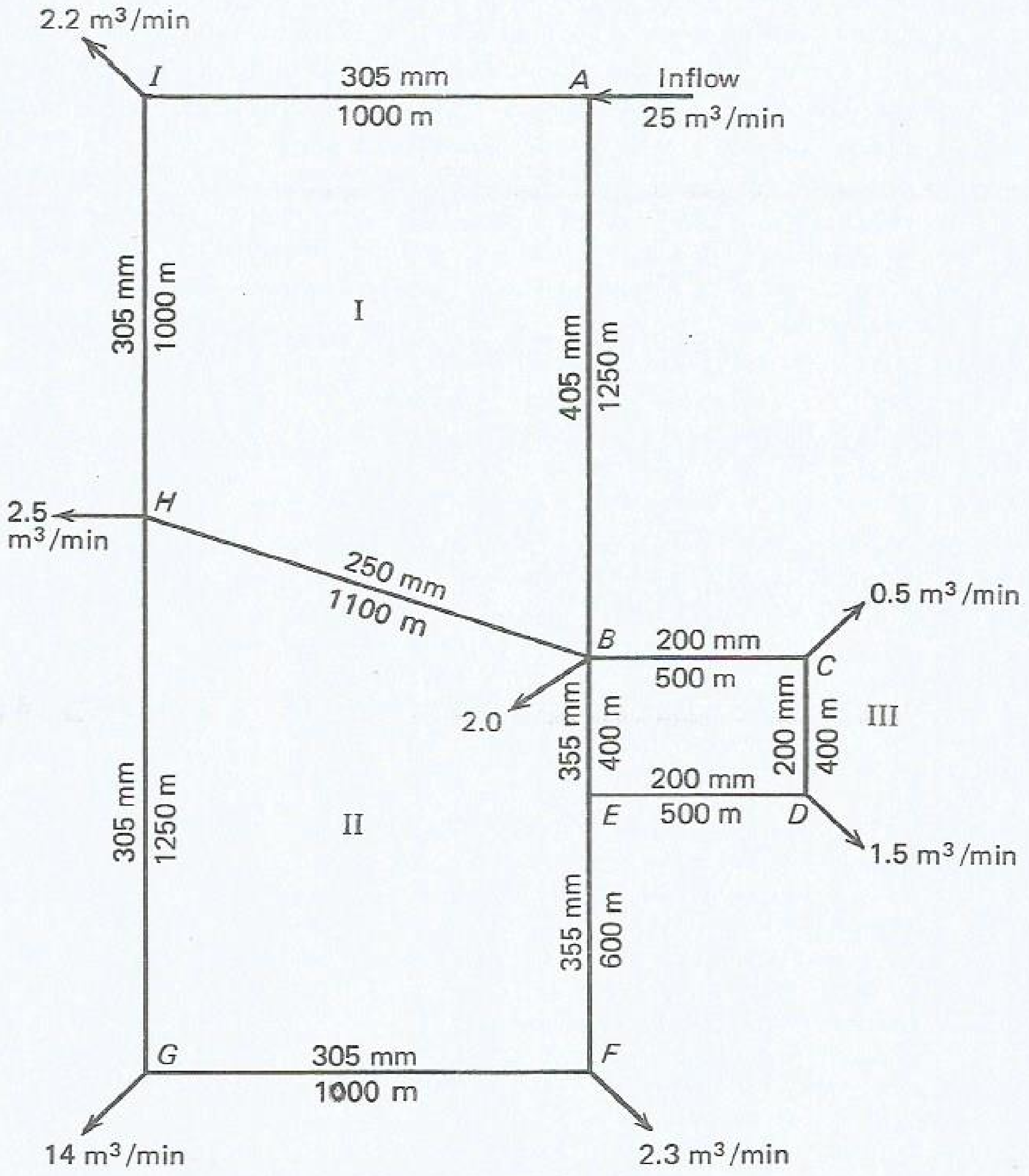


FIGURE 7-9
Simplified distribution system.

20 kPa—well below the normal minimum of 150 kPa. The pressure calculation here assumes points *A* and *G* are at equal elevations. If they are not, the static difference must be included in the calculation. The pipe sizes are evidently somewhat inadequate with regard to both velocity and pressure. The deficient lines should be increased in size and the procedure repeated until adequate pressure and velocity are obtained.

TABLE 7-2
Hardy Cross analysis—first correction

Loop I

Line	Flow, m^3/min	Dia., m	Length, m	s	h, m	h/Q, m/(m^3 · min)	
AB	13	0.40	1250	0.0110	13.75	1.058	
BH	2	0.25	1100	0.0033	3.63	1.815	الانبوب BH مشترك بين الحلقة 1 و 2
HI	−9.8	0.30	1000	−0.0260	−26.00	2.653	
IA	−12	0.30	1000	−0.0380	−37.80	3.150	
					−46.42	8.676	

$$\Delta_{I} = -\frac{-46.42}{1.85(8.676)} = 2.9 \ m^3/min$$

Loop II

Line	Flow, m^3/min	Dia., m	Length, m	s	h, m	h/Q, m/(m^3 · min)	
BE	7.5	0.35	400	0.0075	3.00	0.400	الانبوب BE مشترك بين الحلقة 2 و 3
EF	7.0	0.35	600	0.0066	3.96	0.566	
FG	4.7	0.30	1000	0.0067	6.68	1.423	
GH	−9.3	0.30	1250	−0.0236	−29.54	3.177	
HB	−2.0	0.25	1100	−0.0033	−3.63	1.815	
					−19.53	7.381	

$$\Delta_{II} = -\frac{-19.53}{1.85(7.381)} = 1.4 \ m^3/min$$

Loop III

Line	Flow, m^3/min	Dia., m	Length, m	s	h, m	h/Q, m/(m^3 · min)
BC	1.5	0.20	500	0.0058	2.91	1.937
CD	1.0	0.20	400	0.0028	1.10	1.110
DE	−0.5	0.20	500	−0.0008	−0.38	0.762
EB	−7.5	0.35	400	−0.0075	−3.00	0.400
					0.63	4.209

$$\Delta_{III} = -\frac{0.63}{1.85(4.209)} = -0.1 \ m^3/min$$

As noted above, the procedure outlined in the example is carried out by digital computer techniques. The basic Hardy Cross technique is readily programed[6] and is often used as an exercise in introductory computer courses. The capability of commercially available models includes simulation of in-line pumping and storage.[7] Recently developed models use linear theory or Newton-Raphson techniques to solve somewhat differently formulated energy equations.[8,9] These solution methods converge more certainly than the Hardy

TABLE 7-3
Hardy Cross analysis—second correction

Loop I

Line	Flow, m^3/min	Dia., m	Length, m	s	h, m	h/Q, $m/(m^3 \cdot min)$
AB	15.9	0.40	1250	0.0157	19.65	1.236
BH	3.5	0.25	1100	0.0094	10.34	2.954
HI	−6.9	0.30	1000	−0.0136	−13.60	1.971
IA	−9.1	0.30	1000	−0.0227	−22.70	2.495
					−6.31	8.656

$\Delta_{I} = 0.4$

Loop II

Line	Flow, m^3/min	Dia., m	Length, m	s	h, m	h/Q, $m/(m^3 \cdot min)$
BE	9.0	0.35	400	0.0105	4.20	0.467
EF	8.4	0.35	600	0.0093	5.58	0.664
FG	6.1	0.30	1000	0.0108	10.80	1.770
GH	−7.9	0.30	1250	−0.0175	−21.88	2.769
HB	−3.5	0.25	1100	−0.0094	−10.34	2.954
					−11.64	8.624

$\Delta_{II} = 0.7$

Loop III

Line	Flow, m^3/min	Dia., m	Length, m	s	h, m	h/Q, $m/(m^3 \cdot min)$
BC	1.4	0.20	500	0.0051	2.55	1.821
CD	0.9	0.20	400	0.0023	0.92	1.022
DE	−0.6	0.20	500	−0.0011	−0.55	0.917
EB	−9.0	0.35	400	−0.0105	−4.20	0.467
					−1.28	4.227

$\Delta_{III} = 0.2$

Cross procedure and, in some cases, do not require that the continuity equations be satisfied by an initial set of flow assumptions.

As with any solution to an engineering problem, the predictions of pressure and flow rates are only as accurate as the assumptions or measurements used to formulate the equations.

Appropriate values for friction losses, actual pump performance, and similar factors must be carefully defined.[10] When the model has been properly calibrated, predicted pressures in actual systems have been found to be within 35 to 70 kPa (5 to 10 lb/in^2) of measured values.[11]

7-8 Construction of Water Distribution Systems

Water lines are normally installed within the rights-of-way of the streets. Cover provides protection against traffic loads and freezing and varies from as little as

TABLE 7-4
Hardy Cross analysis—third correction

Loop I

Line	Flow, m^3/min	Dia., m	Length, m	s	h, m	h/Q, $m/(m^3 \cdot min)$
AB	16.3	0.40	1250	0.0165	20.63	1.265
BH	3.2	0.25	1100	0.0080	8.80	2.750
HI	−6.5	0.30	1000	−0.0122	−12.20	1.877
IA	−8.7	0.30	1000	−0.0209	−20.90	2.402
					−3.67	8.294

$\Delta_I = 0.2$

Loop II

Line	Flow, m^3/min	Dia., m	Length, m	s	h, m	h/Q, $m/(m^3 \cdot min)$
BE	9.5	0.35	400	0.0116	4.64	0.488
EF	9.1	0.35	600	0.0107	6.42	0.705
FG	6.8	0.30	1000	0.0132	13.20	1.941
GH	−7.2	0.30	1250	−0.0147	−18.38	2.552
HB	−3.2	0.25	1100	−0.0080	−8.80	2.750
					−2.92	8.436

$\Delta_{II} = 0.2$

Loop III

Line	Flow, m^3/min	Dia., m	Length, m	s	h, m	h/Q, $m/(m^3 \cdot min)$
BC	1.6	0.20	500	0.0066	3.30	2.063
CD	1.1	0.20	400	0.0033	1.32	1.200
DE	−0.4	0.20	500	−0.0005	−0.25	0.625
EB	−9.5	0.35	400	−0.0116	−4.64	0.488
					−0.27	4.376

$\Delta_{III} = 0.03$

0.75 m (2.5 ft) in the south to as much as 2.4 m (8 ft) in the north. Trench width must be great enough to provide room to join the pipe sections and install required fittings. Clearance of about 150 mm (6 in) on either side is normally adequate. This requires a trench width of about 1760 mm (68 in) for a 1220-mm (48-in) pipe. The trench width must be increased at joints and fittings. An extra depth of 150 mm (6 in) and an extra width of 250 mm (10 in) on either side should be provided for a distance of 900 mm (3 ft) at the joints.

Since water line trenches are relatively shallow, they are unlikely to require bracing except in unstable soils. In rock excavation, the trench should be cut to a level at least 150 mm (6 in) below the final grade of the pipe and a cushion of sand or clean fill should be placed between the rock and the pipe.

Backfill material should be free of cinders, refuse, and large stones. Careful backfilling decreases the load on the pipe and will decrease the proba-

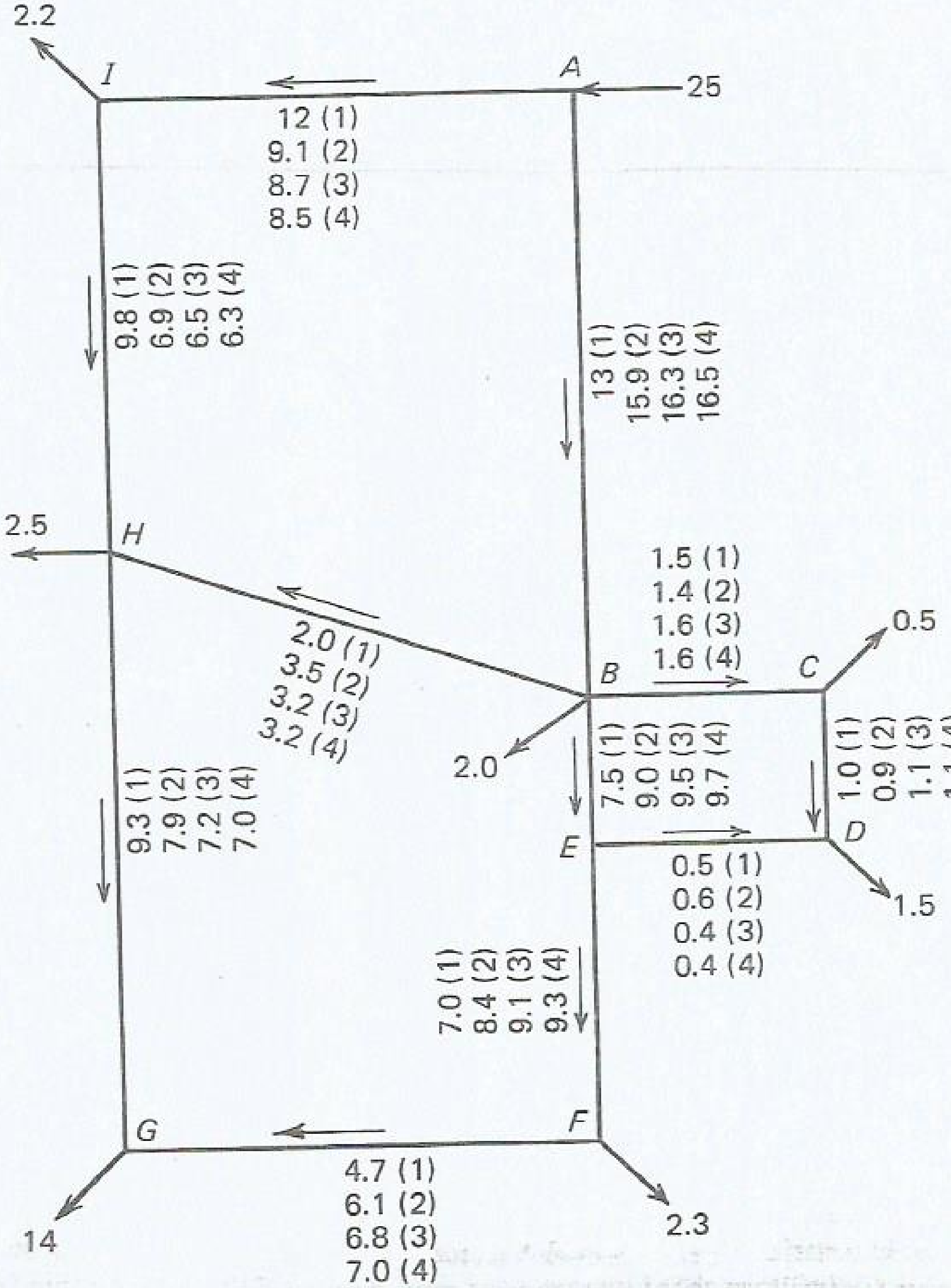


FIGURE 7-10
Corrected flows in distribution system.

bility of failure. Standard pipe bedding conditions are illustrated in Fig. 6-1. Type 2 bedding is commonly used and requires that the fill material be placed by hand up to the centerline of the pipe in carefully tamped layers of not more than 75 mm (3 in). From the centerline to 300 mm (12 in) above the top of the pipe, the fill should be placed by hand. From 300 mm (12 in) above the top of

Mosul university
College of engineering
Civil Engineering Department
4th year

Sanitary Engineering-Networks

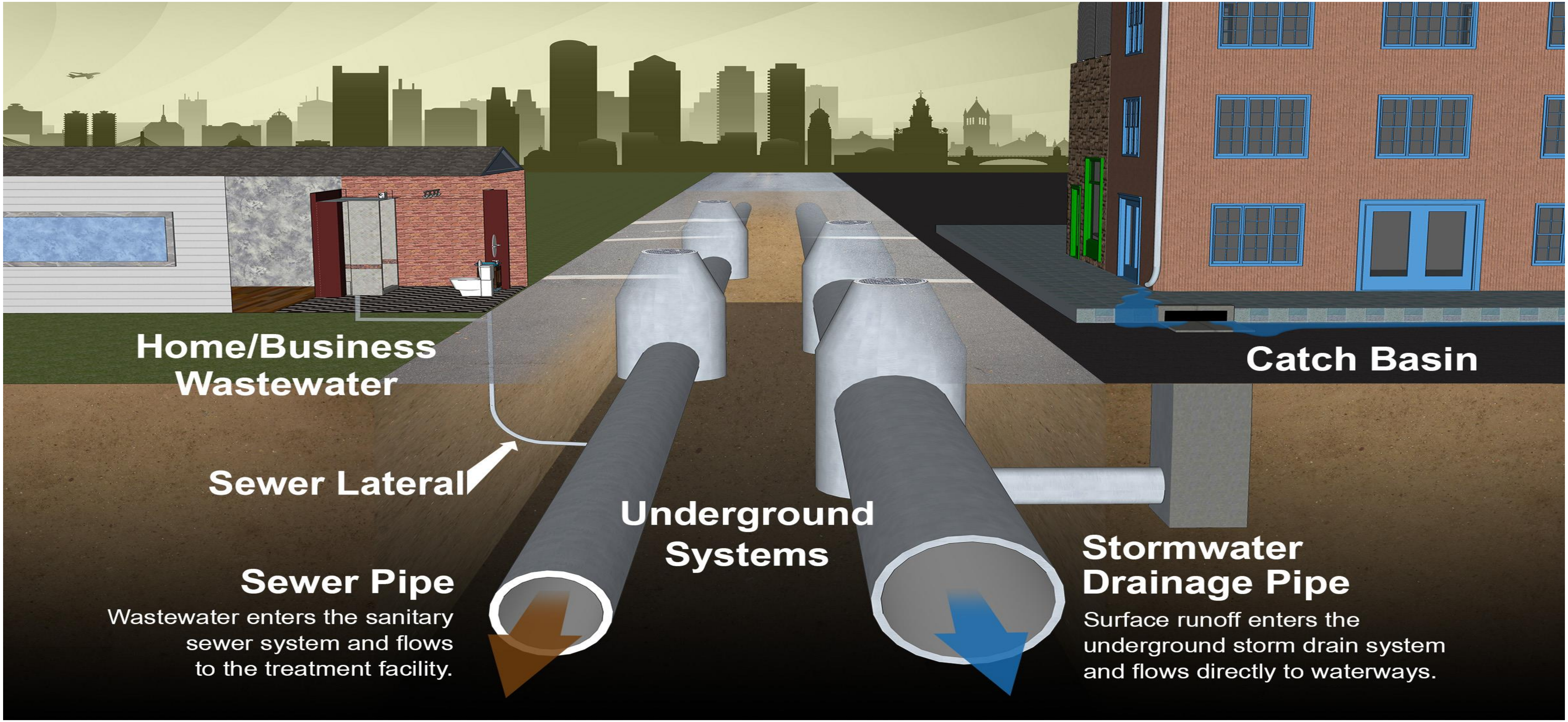
Design of Sewer Systems

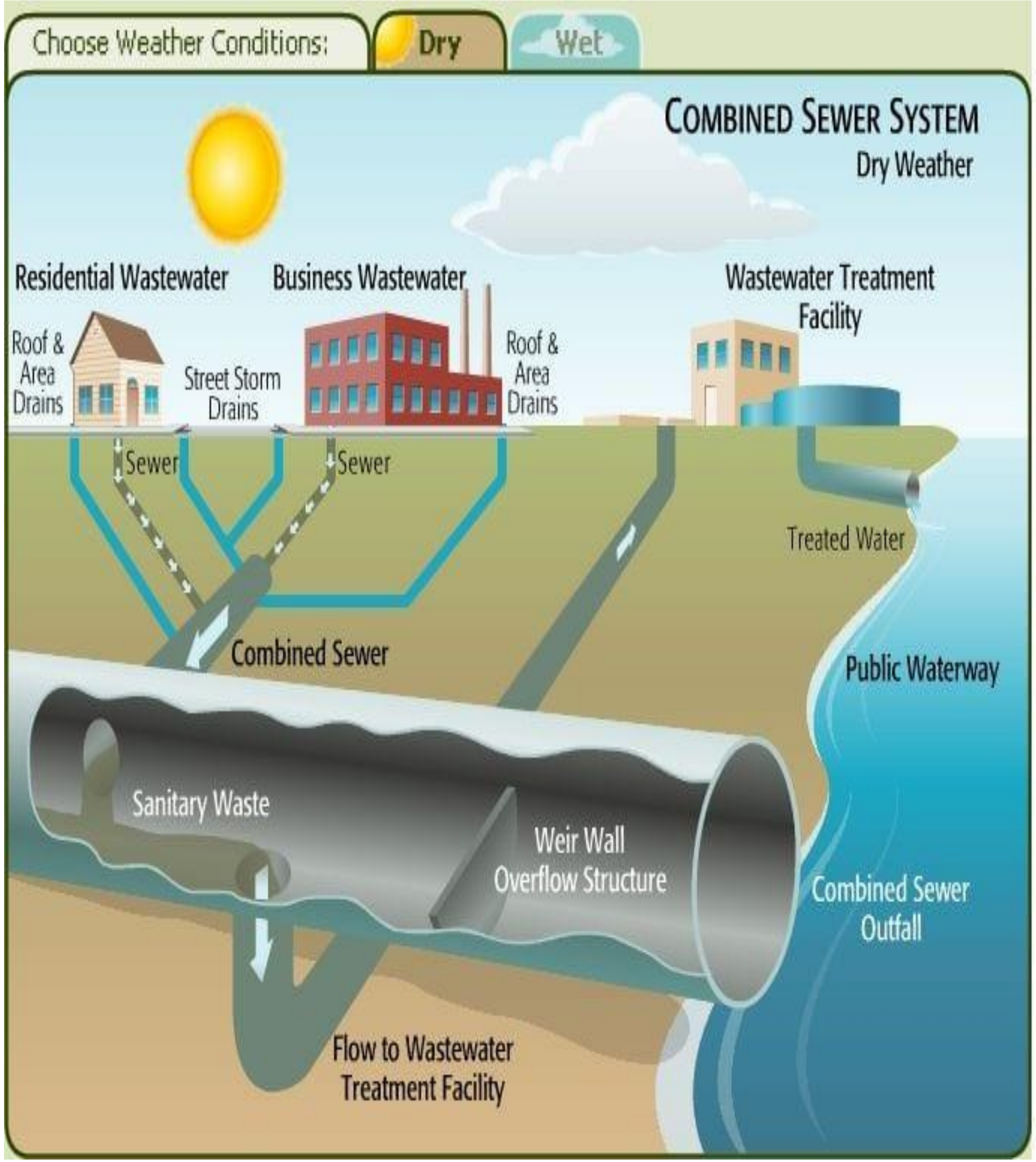
Mohammed Salim Mahmood

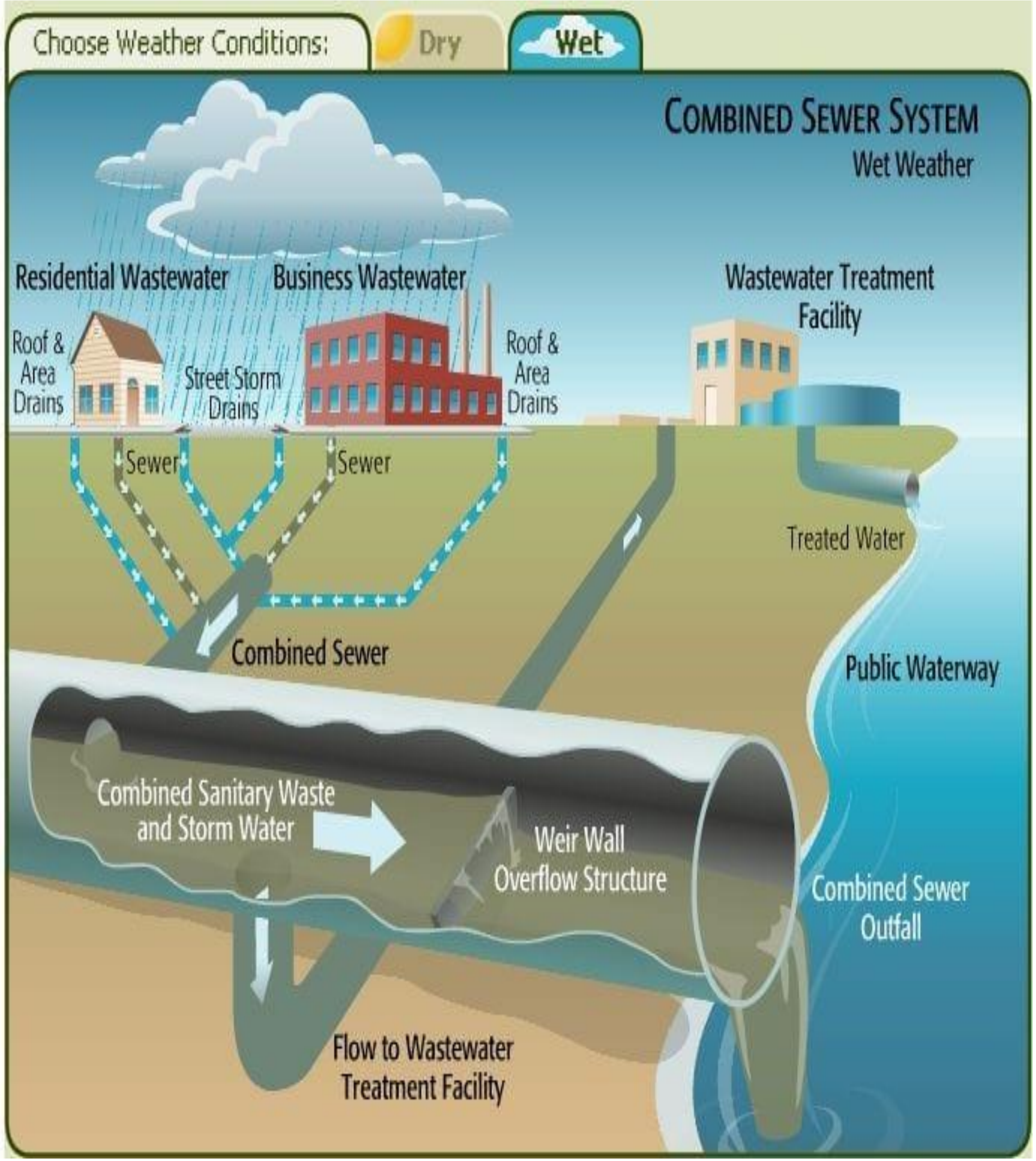
Design of Sewer Systems

Sanitary Sewer Design

Storm Sewer Design

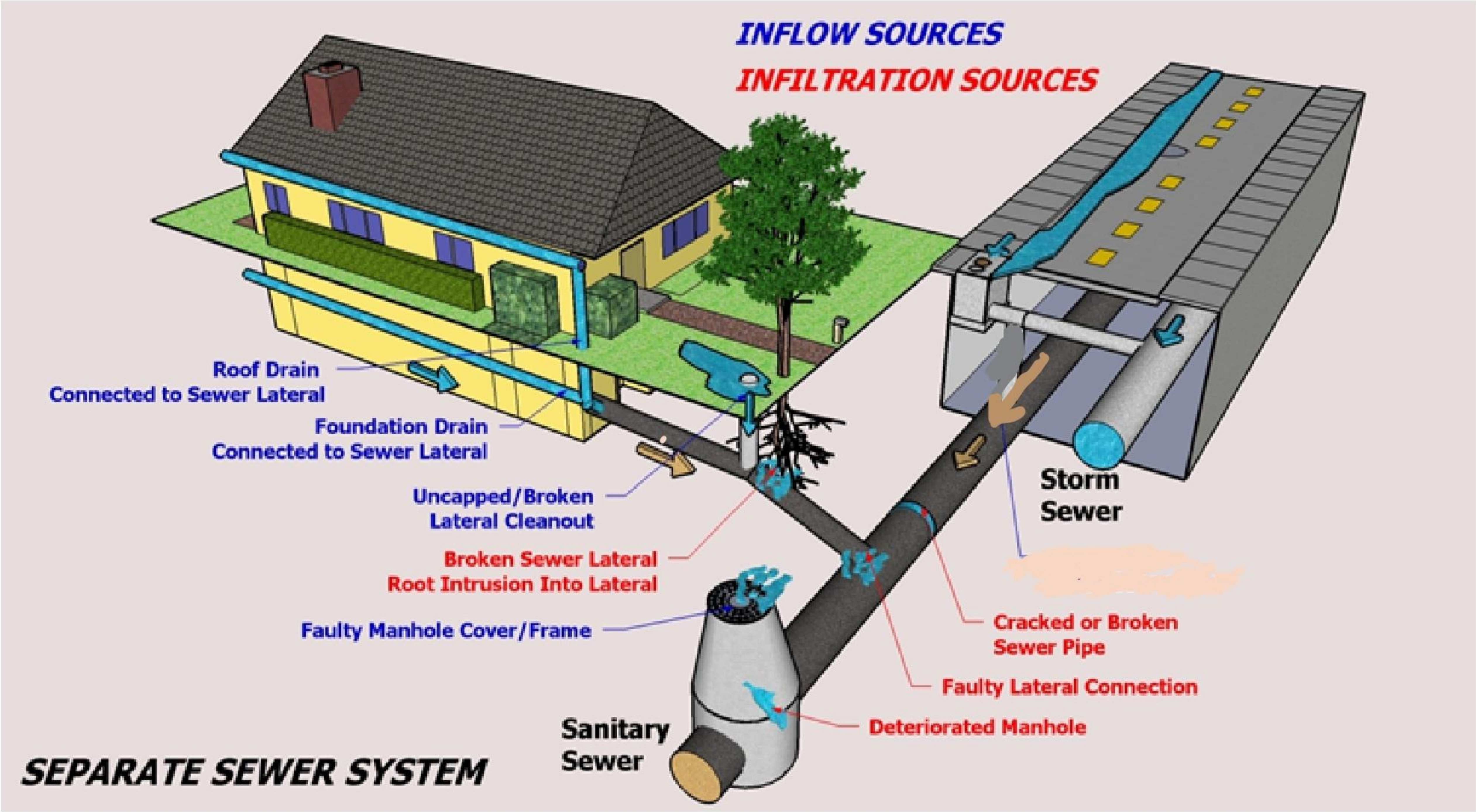



Choose Weather Conditions:
Dry
Wet
COMBINED SEWER SYSTEM
Dry Weather
Residential Wastewater
Business Wastewater
Wastewater Treatment Facility
Roof & Area Drains
Street Storm Drains
Roof & Area Drains
Sewer
Sewer
Treated Water
Combined Sewer
Public Waterway
Sanitary Waste
Weir Wall Overflow Structure
Combined Sewer Outfall
Flow to Wastewater Treatment Facility


Choose Weather Conditions:
Dry
Wet
COMBINED SEWER SYSTEM
Wet Weather
Residential Wastewater
Business Wastewater
Wastewater Treatment Facility
Roof & Area Drains
Street Storm Drains
Roof & Area Drains
Sewer
Sewer
Treated Water
Combined Sewer
Public Waterway
Combined Sanitary Waste and Storm Water
Weir Wall Overflow Structure
Combined Sewer Outfall
Flow to Wastewater Treatment Facility

Sewer Networks System

- General Consideration
- **Sewerage**: refer to the collection, treatment and disposal of liquid waste
- **Sewerage Works (or Sewage works):** include all the physical structures required for that collection treatment and disposal
- **Sewage:** is the liquid waste conveyed by sewer and may include domestic and industrial discharges as well as the storm sewage , infiltration and inflow.
- **Domestic (or Sanitary Sewage):** is that which originates in the sanitary convenience of dwellings, commercial or industrial facilities and institutions.
- **Industrial Wastewater:** includes the liquid discharges from industrial processes such as manufacturing and food processing.
- **Storm Sewage:** is flow derived from rainfall events and deliberately introduced into sewers intended for its conveyance
- **Infiltration:** is water which enters the sewers from the ground through leaks.
- **Inflow:** is water which enters the sewers from the surface during rainfall events through flows in the system or through connections to roof or basement drains.
- **A sewer:** is a pipe or conduit generally closed but normally not flowing full for carrying sewage.


INFLOW SOURCES
INFILTRATION SOURCES
Roof Drain
Connected to Sewer Lateral
Foundation Drain
Connected to Sewer Lateral
Uncapped/Broken
Lateral Cleanout
Broken Sewer Lateral
Root Intrusion Into Lateral
Faulty Manhole Cover/Frame
Storm
Sewer
Cracked or Broken
Sewer Pipe
Faulty Lateral Connection
Deteriorated Manhole
Sanitary
Sewer
SEPARATE SEWER SYSTEM

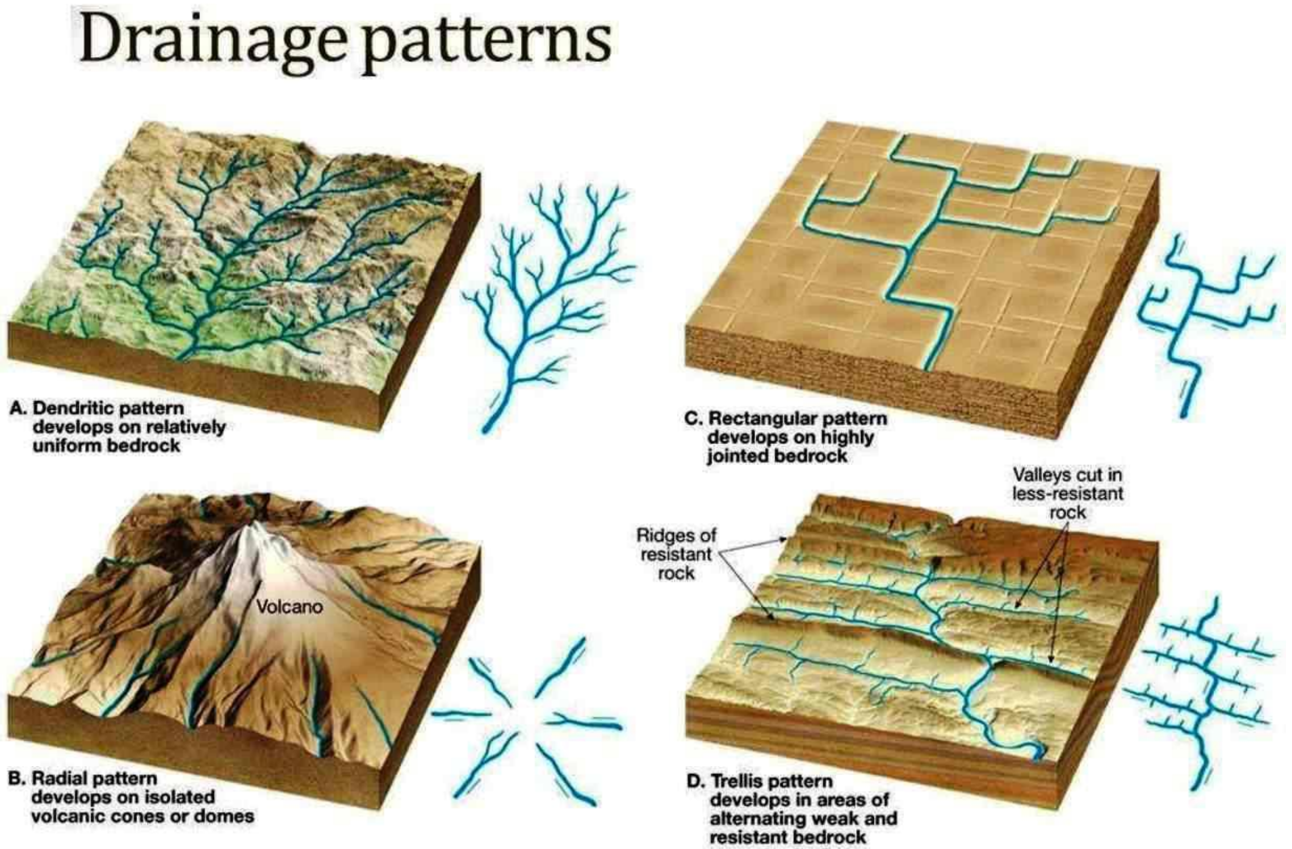
Storm Sewer Design

Storm Water Flow

- **Rainfall intensity (i):** the rate at which rain falls over an area in **mm/hr** , or in/hr
- **Rainfall duration (t):** the time in **minutes** at which rain falls at constant rate.
- **Rainfall frequency (T):** the period in **years** at which the storm happened again
- **Runoff:** the water that runs off immediately to appears in streams during rainfall events upon catchment area.

Factors which affect runoff

- **The topography** of the drainage area, it is degree of roughness and slope, affects the time of concentration of the runoff and there by cause high or low runoff rates.
- **Geology** of the area including perviousness or imperviousness of the sub terrain and slope of the strata
- **The slope** and the characteristics of the drainage area.
- **Solar radiation** and it is variation on the watershed will affect evaporation.



Calculation of runoff (storm Sewage)

The rational method

$Q = C\ i\ A$

Q= actual amount of runoff (volume/time)

C= runoff coefficient, i.e, the fraction of the incident precipitation which appear as surface flow. (unitless)

i= rainfall intensity (rainfall depth / time)

A= area

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

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

Description of area	C
Business	
وسط المدينة Downtown area	0.70–0.95
حي سكني 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
اطراف المدن والضواحي 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
ملعب Playgrounds	0.20–0.35
Railroad yards	0.20–0.40
غير مستفاد منه Unimproved areas	0.10–0.30

Example 13-2 Determine the runoff coefficient for an area of 0.2 km2, of which 3000 m2 is covered by buildings, 5000 m2 by paved driveways and walks, and 2000 m2 by Portland cement streets. The remaining area is flat, heavy soil covered by grass.

Solution:

From Table 13-2, one may obtain values of C for each area:

Roofs 0.70 to 0.95

Driveways and walks 0.75 to 0.85

Street 0.80 to 0.95

Lawn 0.13 to 0.17

The fraction of the area with each surface is

Roofs 3000/200,000 = 0.015

Driveways and walks 5000/200,000 = 0.025

Street 2000/200,000 = 0.010

Lawn 190,000/200,000 = 0.95

C_{min}= 0.015* 0.70+ 0.025 *0.75+ 0.010*0.8+0.95*0.13=0.16

$C_{max.}$ = 0.015* 0.95 + 0.025 * 0.85 + 0.010* 0.95 +0.95* 0.17 =0.21

The average value of C, depending on the specific values chosen for the individual areas, will thus lie between 0.16 and 0.21.

Rainfall intensity (i) = f (t)

Time of concentration (tc) : the time required for the maximum runoff to develop.

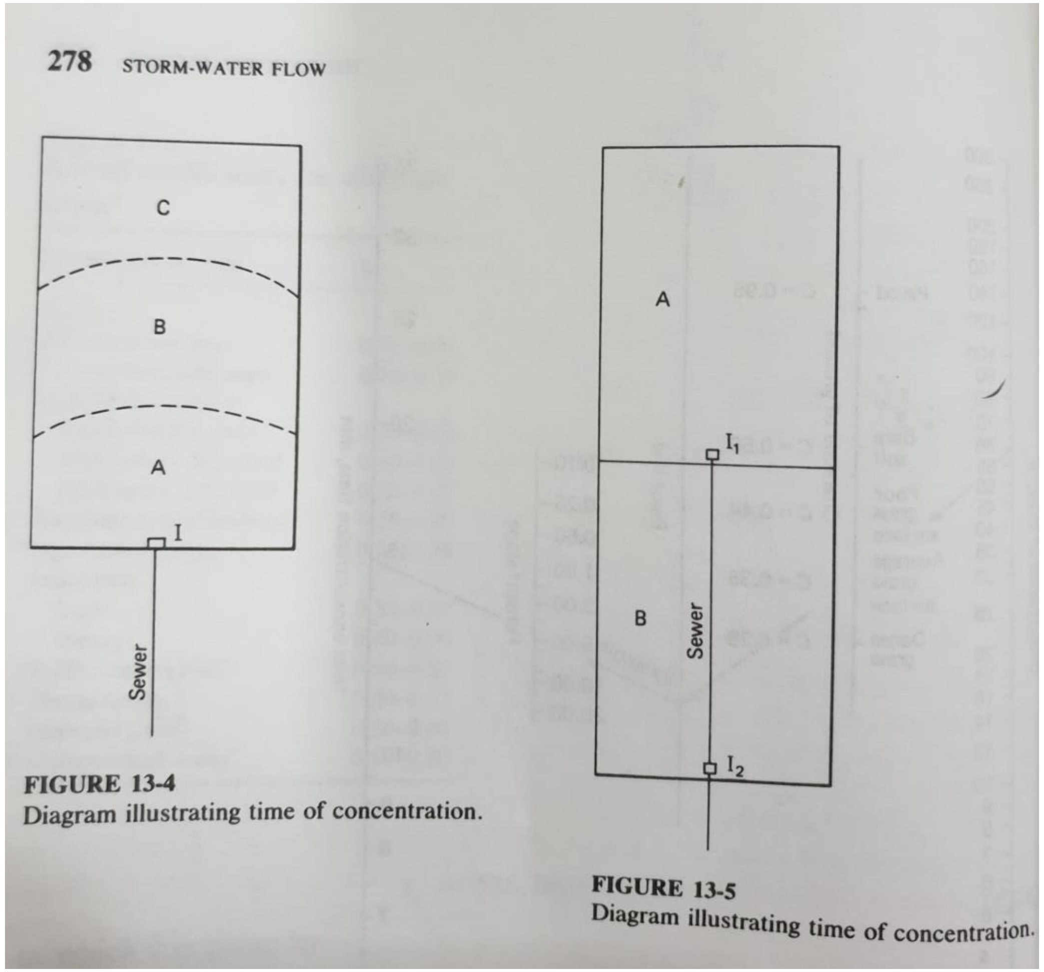


FIGURE 13-4
Diagram illustrating time of concentration.

FIGURE 13-5
Diagram illustrating time of concentration.

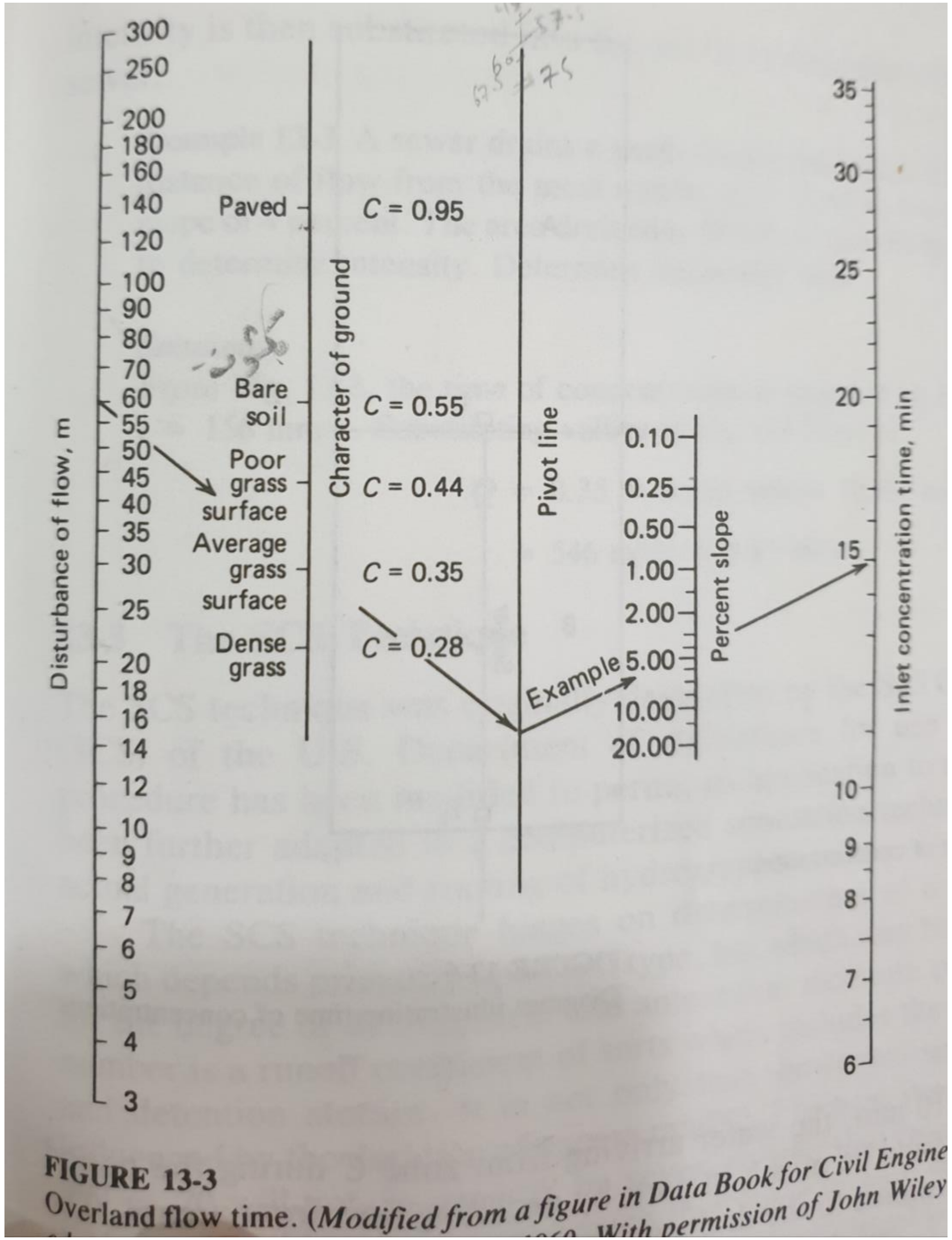


FIGURE 13-3
Overland flow time. (*Modified from a figure in Data Book for Civil Engine... With permission of John Wiley*

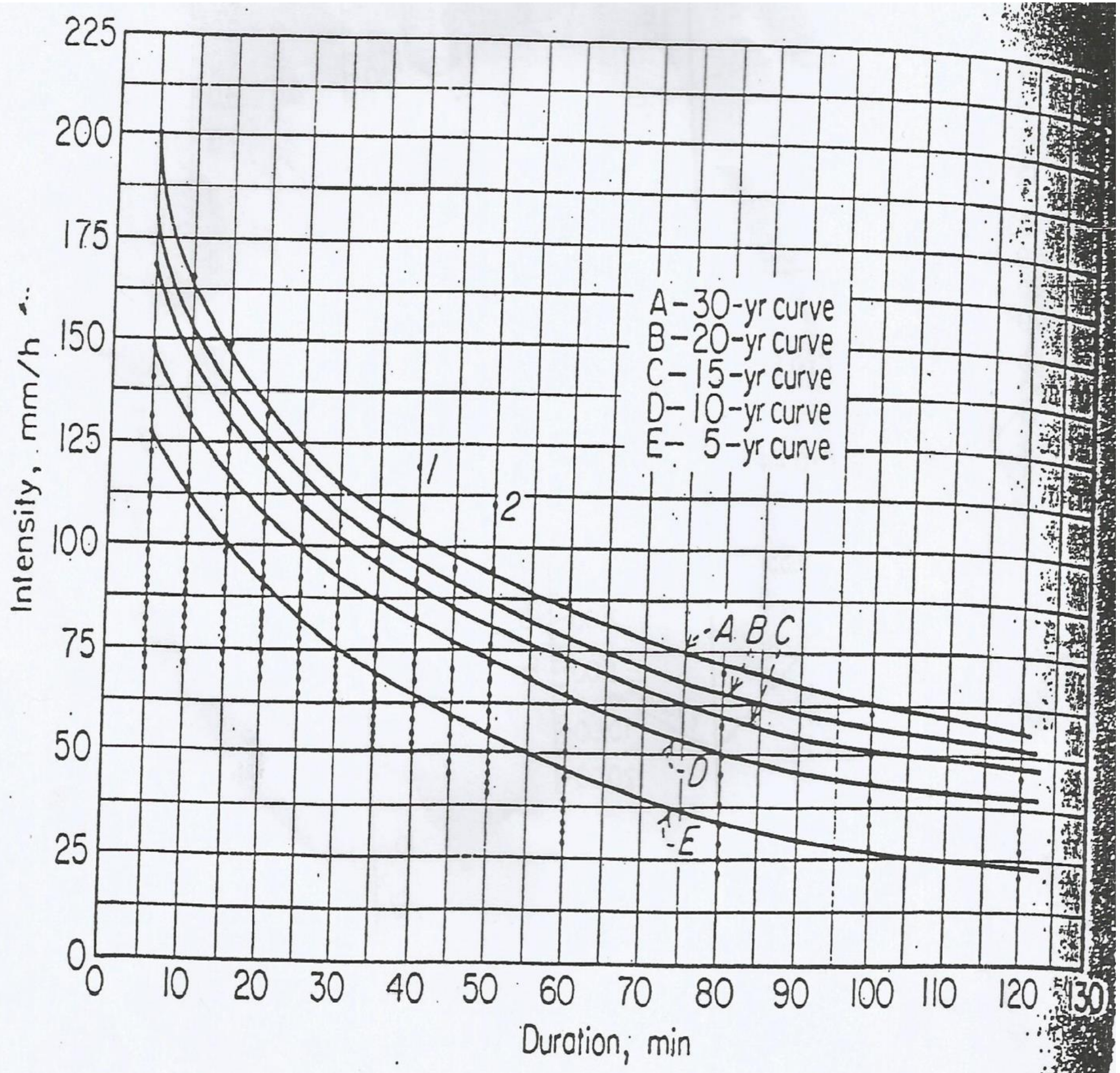


Figure 13-4 Rainfall curves derived from storm records.

Table 13-3 Precipitation formulas for various parts of the United States (i, mm/h; t, min)

Frequency, years	Area 1	Area 2	Area 3	Area 4	Area 5	Area 6	Area 7
2	$i = \frac{5230}{t+30}$	$i = \frac{3550}{t+21}$	$i = \frac{2590}{t+17}$	$i = \frac{1780}{t+13}$	$i = \frac{1780}{t+16}$	$i = \frac{1730}{t+14}$	$i = \frac{810}{t+11}$
5	$i = \frac{6270}{t+29}$	$i = \frac{4830}{t+25}$	$i = \frac{3330}{t+19}$	$i = \frac{2460}{t+16}$	$i = \frac{2060}{t+13}$	$i = \frac{1900}{t+12}$	$i = \frac{1220}{t+12}$
10	$i = \frac{7620}{t+36}$	$i = \frac{5840}{t+29}$	$i = \frac{4320}{t+23}$	$i = \frac{2820}{t+16}$	$i = \frac{2820}{t+17}$	$i = \frac{3100}{t+23}$	$i = \frac{1520}{t+13}$
25	$i = \frac{8300}{t+33}$	$i = \frac{6600}{t+32}$	$i = \frac{5840}{t+30}$	$i = \frac{4320}{t+27}$	$i = \frac{3300}{t+17}$	$i = \frac{3940}{t+26}$	$i = \frac{1700}{t+10}$
50	$i = \frac{8000}{t+28}$	$i = \frac{8890}{t+38}$	$i = \frac{6350}{t+27}$	$i = \frac{4750}{t+24}$	$i = \frac{4750}{t+25}$	$i = \frac{4060}{t+21}$	$i = \frac{1650}{t+8}$
100	$i = \frac{9320}{t+33}$	$i = \frac{9520}{t+36}$	$i = \frac{7370}{t+31}$	$i = \frac{5590}{t+28}$	$i = \frac{6100}{t+29}$	$i = \frac{5330}{t+26}$	$i = \frac{1960}{t+10}$