An Evaluation and Pressure-Driven Design of Potable Water Plumbing Systems

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By

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Abstract

Potable water distribution systems are broken into major and minor distribution networks. Major water distribution networks refer to large-scale municipal pipe systems extending from the treatment plant to the upstream node of the water service line for buildings. Minor water distribution systems, also referred to as plumbing water distribution systems, run from the upstream node of the water service line to all interior plumbing fixtures and demand nodes associated with the building. Most texts and research papers focus on major systems, while only a small number of documents are available concerning the design and analysis of minor systems. In general, the available minor system documents are quite prescriptive in nature. This thesis presents a comprehensive evaluation of contemporary plumbing water distribution system design. All underlying theory is explained and advantages and drawbacks are discussed. Furthermore, contemporary methods for designing minor distribution systems have come under recent scrutiny. Issues have been raised regarding the accuracy of water demand estimation procedures for plumbing systems, namely, Hunter's method. Demand estimates are crucial for designing minor piping systems. The formulation and

application of a pressure-driven design approach to replace Hunter-based design methods is presented. EPANET, a commonly used hydraulic modeling software package, is utilized to evaluate network behavior. Example applications are presented to illustrate the robustness of a pressure-driven approach, while also allowing the evaluation of plumbing water distribution system performance under worst-case loading conditions.

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CHAPTER 1: Introduction

1.1 Major and Minor Systems

Design of plumbing systems has largely remained as an empirical tool. The available books are prescriptive and hardly venture into the principles behind their prescriptions. The hydraulic problem itself is quite intricate. We define the "major system" to be the water distribution system extending from the source/treatment plant to the street level of the plumbing system. We designate the plumbing system as the "minor system". The major system is designed for continually imposed demand at the demand nodes.

We know domestic (plumbing) water demands are intermittent, which is a critical observation. The intermittent demand volumes are much less than the total demand of plumbing systems. Hence, smaller diameter pipes can be used for design. However, knowledge of the intermittent demand is essential. Based on this fact, Hunter (1940) proposed a probabilistic demand approach for designing plumbing water distribution systems. He suggested taking the design demand value as the aggregate demand that can be exceeded with only 1% probability. As a consequence, plumbing pipes in general have diameters less than 1 inch for most buildings up to 5 – 7 stories. For buildings taller than 7 stories, the structures are divided into zones of about 7 stories each and each zone's plumbing system is designed independently.

The major systems have a relatively longer travel time (from treatment plant to minor system) than minor systems, and should have sufficient supply during emergencies. Because of this consideration, the major systems are designed for the full demands. The minor system is totally dependent on the energy level available at its connection to the major system. Minor system design depends on the following: (1) energy level of the major system at the lateral connection (2) 1% exceedance demand that accounts for the intermittent usage of fixtures (3) energy required for proper functioning of the fixture that is most hydraulically distant from the major system lateral.

1.2 Objectives

This thesis serves as a complete documentation on plumbing system design and evaluation. All design procedures and the engineering principles are presented and well explained. The thesis has the following objectives:

1. Provide a comprehensive synthesis of the design methodology fully documenting the underlying principles.
2. Formulate and solve the plumbing problem as a “pressure-driven” problem.

1.3 Organization of the Thesis

The thesis is organized as follows. Chapters 2, 3, and 4 explain the key design elements of plumbing water distribution systems. More specifically, Chapter 2 discusses the theory behind plumbing demand estimation. The Hunter curve procedure used in plumbing design codes is fully explained and weaknesses are pointed out. Chapter 3 presents the conceptual analysis of plumbing systems. The modeling procedure for cold water and hot water pipes is discussed in a comprehensive manner. The role of pumps and gravity tanks is included. Chapter 4 contains the step by step design procedure. The role of the required pressure at the hydraulically most distant point is analyzed from the perspective of critical energy slope. The importance of the critical slope in supplying water to all the demand nodes is fully explained. Design examples are given. Chapter 5 presents a pressure-driven analysis. The resulting formulation is solved with the aid of the SOLVER program in MS Excel. The same formulation is again solved with the hydraulic network program EPANET. The formulation serves as a basis, not only for analysis, but for design, as well. By iteratively updating pipe diameters, it is possible to obtain a more efficient design in comparison to the usual Hunter curve based approach. Chapter 6 provides a detailed overview of interior fire protection systems. This chapter, although primarily a literature review, presents all water demands attributed to fire protection. Chapter 6, in conjunction with Chapter 2 – 5, define all water demands required within a building. A summary of the findings and possible future research areas are given in Chapter 7. Chapter 8 contains all works cited within this thesis. Appendix 1 contains a primer on EPANET explaining the procedure for pressure-driven analysis.

CHAPTER 2: Demand Analyses for Plumbing Water Distribution Systems

2.1 Introduction

Plumbing water distribution systems are designed on the idea of the most probable peak demand loading, which reflects the worst-case scenario for a system. These types of systems require different considerations than large-scale water distribution networks. The difference is primarily attributed to uncertainty regarding the use of plumbing fixtures, hence uncertainty in demand loadings. There are two methods that have been proposed to aid in the design of plumbing water systems. Currently, the plumbing industry uses Roy B. Hunter's method for approximating peak demand loadings on a building's water distribution system. This method was developed in the 1940's and, over the last 25 years, has come under scrutiny for its overly-conservative estimates (Konen, 1980; Breese, 2001). Another method, which is not cited in any major U.S. plumbing codes, has been developed by the American Water Works Association (AWWA). The "fixture value method" was introduced in 1975 and presented in AWWA's M22 Manual. This is an empirical approach based on data obtained from water meter data loggers. Both procedures are separately discussed in detail in the following paragraphs.

2.2 BMS 65 - "Methods of Estimating Loads in Plumbing Systems" (Hunter's Method)

It is expedient to first discuss some background information regarding the Hunter method. Hunter's main goal was to standardize plumbing regulations in the United States. George (2001) states that Hunter's (National Bureau of Standards, 1940) "BMS 65 Methods of Estimating Loads in Plumbing Systems" report provides the plumbing industry with a tool for estimating the demand loads on a plumbing water distribution system by applying a common unit (fixture unit) to all different types of fixtures. George also points to Hunter's (1940) "BMS 66 Plumbing Manual" as the basis for code development and contemporary plumbing codes (e.g. The International Plumbing Code and The Uniform Plumbing Code).

Hunter observed the following. All fixtures are not used simultaneously. The durations of use are different and times between uses are different and both serve as characteristic parameters in the sense that they, together, determine the rate of flow with a plumbing pipe. For example, a flush valve is typically assumed to operate over a 9 second period providing a volume of 4 gallons. This yields a design flow of 27 gpm [(4/9)*(60) = 26.6 gpm]. Consider a building with 20 flush valves. The pipe capacity should be based on the number of flush valves that are used simultaneously. This is, given that there are 20 flush valves, we need to determine how many of these 20 fixtures will be operated at any given instant. Hunter interpreted it probabilistically. He defined that the probability of exceeding the defined threshold number of units, *m*, is only 1%. For example, for a probability of use, *S*, equal to 0.03 (that a fixture will be in use) and using the binomial probability law for a total of 20 fixtures, we obtain *m* equal to 3 for $P\left[X > m\right] \le 0.01$. That is, $P[X > m] = 1 - P[X \le m] = 1 - \sum_{i=0}^{m} \begin{pmatrix} 10 \\ < \\ i \end{pmatrix} (0.03)^{i} (0.97)^{20-i}$.

We find the probability of using more than 3 fixtures simultaneously out of 20 is less than 1%. Therefore, the design flow should be 3(27) = 81 gpm for 20 fixtures.

It is clear that the above calculation depends on the probability that a fixture is in use, denoted by *S*. Hunter defined *S* as, *S* = the duration of use, *t*, divided by the time between uses, *T*. S = 9/300 for flush valves. Note that for the probabilities to sum to 1, we should have the duration of use included in time between uses. Figure 2.1 provides support for this definition. At any given instant a fixture is operational or non-operational.

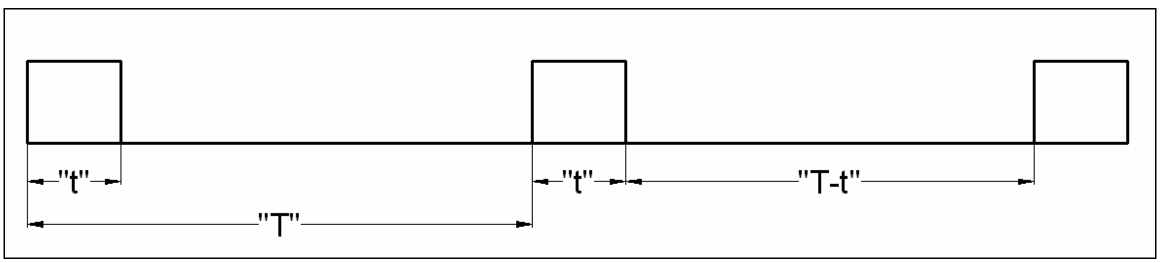

Figure 2.1: Fixture Operation

While it is conceivable that the duration, *t*, will nearly be a constant, the time between uses, *T*, will not. Ideally, one may define *S* as the ratio of total time of fixture use within a day (in minutes) to the length of a day (in minutes).

Adopting Hunter's definition of a fixture's probability of use, we can determine these probabilities based on the duration of use and the time between uses (or the number of uses within a day). This probability enables us to determine at most how many fixtures will be simultaneously operated out of a given number of fixtures at a chosen probability threshold (1% chosen by Hunter).

By utilizing the volume of water disposed of through a certain fixture type, Hunter was able to relate the water use to the total number of fixtures at the 1% probability threshold. As an example, we observed that if there are 20 flush valves a demand of 81 gpm will not be exceeded 99% of the time (or will be exceeded only 1 % of the time). For 100 fixtures with a probability of use of 0.03 for any one fixture, we find through the binomial distribution that the probability of using more than 8 fixtures simultaneously is less than 1%. For these 8 fixtures at 27 gpm per fixture, the supply pipe should have a design flow of 216 gpm. It is clear that Hunter's method provides a design demand value for a specified number of fixtures such as 100, but it does not tell us how many fixtures should be provided. The required number of fixtures is clearly dependent on the peak occupancy rate, which are governed by local regulations. Table 2.1 contains an example from the City of San Jose, CA (Wang, 2004).

Table 2.1: Minimum Number of Plumbing Fixtures (Retail Stores)

Water Closets (fixtures per person)		Urinals (fixtures per person)	Lavatories (fixtures per person)	Drinking Fountain (fixtures per person)
Male	Female	Male:		
1: 1 - 100	1: 1 - 25	0: 0 - 25	One for each two water closets	0: 1 - 30
2: 101 - 200	2: 26 - 100	1: 26 - 100		1: 31 - 150
3: 201 - 400	4: 101 - 200	2: 101 - 200		
	6: 201 - 300	3: 201 - 400		
	8: 301 - 400	4: 401 - 600		
*Over 400, add one fixture for each additional 500 males and one for each 150 females		Over 600, add one fixture for each additional 300 males		One additional drinking fountain for each 150 persons thereafter

Hunter considers three different types of fixtures namely, flush valves, flush tanks, and bath tubs. He uses the probabilities of use as 9/300, 60/300, and 60/900 for these three types, respectively. At a 1% threshold level, we can determine how many plumbing fixtures will be simultaneously used for each type of fixture. Hunter uses 27 gpm for flush valves, 4 gpm for flush tanks, and 8 gpm for bath tubs. Using these conversions, we can convert the units of those fixtures in simultaneous usage to their equivalent water demand. Hunter collapses the three different fixture groups into a single group by utilizing equivalent fixture ratios (weights). At a demand of 150 gpm, either 56 units of flush valves, 133 units of flush tanks, or 167 units of bath tubs will be used. This produces a ratio of 56 : 133 : 167 or 1 : 2.375 : 2.982. By considering various demand levels, the average equivalent fixture ratios are taken to be 1 : 2 : 2.5 for flush valves, to flush tanks, to bath tubs. Fixture ratios are inversely related to the demand. By assigning 10 fixture units to each single flush valve, we have 5 fixture units for each flush tank and 4 fixture units for each bath tub. Therefore, the equivalent fixture units permit the usage of single demand curve. For example, consider 20 flush valves and 20 flush tanks at a probability of exceedance of 1%. 3 flush valves and 8 flush tanks will be operated simultaneously for a total demand of 3(27) + 8(4) = 113 gpm. Using equivalent fixture units we have a total of 20(10) + 10(5) = 300 fixture units. That is, 300 fixture units have a demand of 113 gpm at a 1% exceedance probability level. The above procedure is presented in steps in the following paragraphs.

Step 1:

Let S = the probability that a fixture is being used. It is estimated by S = t/T in which; t = duration of fixture use, T = time between operations of a fixture. Over the duration of use "t" seconds, at an average flow rate of "q" (gpm), the fixture must provide the required volume of water "Q" gallons. That is t*q = Q, and the plumbing pipe carrying a flow at "q" (gpm) to a single fixture can provide for that fixture's demand Q. If there are "m" fixtures in simultaneous operation, the plumbing system demand is m*q. Three types of fixture are considered: (1) flush valves for water closets, (2) flush tanks for water closets, and (3) bath tubs. Hunter's (1940) probabilities usage for the three fixtures are shown in Table 2.2.

Table 2.2: Values of "t/T", "q", and "Q"

Fixture Type	S = t/T	Required Flow Rate, q	Demand Satisfied in "t" secs, t*q = Q
	(sec/sec)	(gpm)	(gallons)
Flush Valve	9/300	27	4
Flush Tank	60/300	4	4
Bath Tub	60/900, 120/900	8	8 or 16

Step 2:

Find "m" such that $P[X > m] = 0.01$ in which X = the number of fixtures in simultaneous use out of "n" fixtures.

$$P[X > m] = \sum_{x=m+1}^{n} \begin{pmatrix} n \\ c \\ x \end{pmatrix} S^{x}(1 - S)^{n-x} \qquad (1)$$

Representative *m*-values are provided below (Hunter, 1940). These numbers are based on [exceedance probability level, β for P(x>m) = β for β < 0.01]

Table 2.3: Threshold number of fixtures, m, for selected number of fixtures, n

Flush Valve S = 9/300		Flush Tank S = 60/300		Bath Tub S = 60/900	
n	m	n	m	n	m
6	2	3	2	3	2
16	3	5	3	8	3
30	4	15	7	15	4
66	6	25	10	31	6
107	8	40	14	59	9
151	10	132	36	113	14
199	12	240	60	221	23
299	16	305	74	309	30

In Table 2.3, for flush valves we observe that out of 199 fixtures, no more than 12 fixtures will be used simultaneously. This situation corresponds to a cumulative probability of 99%, or exceeding the use of 12 fixtures is only 1%. It is important to note that this value may be construed as low for some situations. For example, a crowded conference break at a large hotel may induce a large number of patrons to use the bathrooms over a small time interval. It takes only 9 seconds for the fixture to deliver the required 4 gallons per flush, however, it is assumed that the fixture will be engaged only once in five minutes (300 seconds). It is not clear whether the field data will support five minute intervals in a crowded environment. However, the rather high flow rate of 27 gpm for flush valves should compensate even during shorter times between uses.

<u>Step 3:</u>

Calculate the demand in gallons per minutes (gpm) as “mq” and plot it against the corresponding “n” value as shown in Figure 2.2.

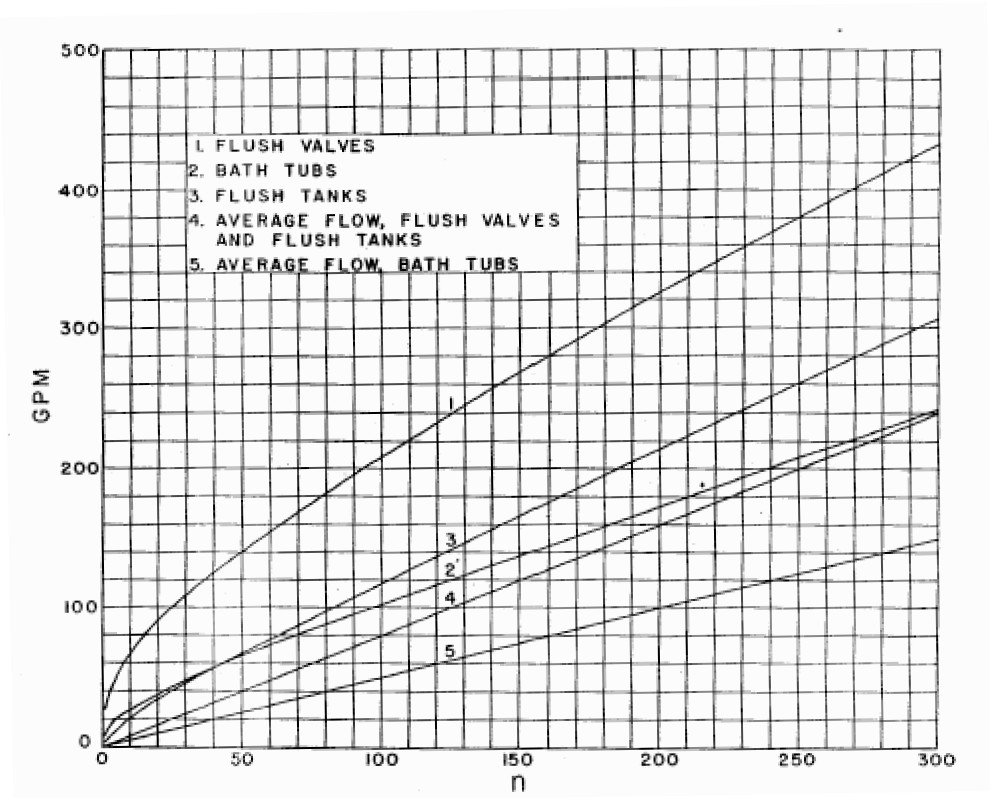

Figure 2.2: Probable Flow in Relation to *n* (Hunter, 1940)

For example, for n = 107, Figure 2.2 yields a total demand of 216 gallons for flush valves. This corresponds to mq = 8(27) = 216 gallons, in which m is obtained from Table 2.3. Figure 2.2, curves 4 and 5, represent the average flow taken as:

$$\frac{nq}{T} = n\frac{tq}{T} = n\frac{t}{T}q = x_{AVE}q \qquad (2)$$

where, $x_{AVE} = n\left(\frac{t}{T}\right) =$ the average value for the binomial distribution representing the average number of fixtures in use out of n fixtures. Our primary interest is curves 1, 2, and 3 which are collapsed into a near single curve in Step 4.

Step 4:

From Figure 2.2, for selected demands, we can find the number of fixtures that are in use for the three categories as shown in Table 2.4:

Table 2.4: Relative Number of Fixtures in Use for Selected Demand Rates (based on Figure 2):

Demand Rate	Flush Valve	Flush Tank	Bath Tub
(gpm)	n	n	n
150	56	133	167
200	93	185	238
250	133	238	312
300	176	292	398

On average, we find the number units of flush tanks and bath tubs that are equivalent to one unit of flush valve in Table 2.5. The equivalent number of demand producing units in Table 2.5 is calculated by dividing the respective Table 2.4 entries by the respective number of flush valves. From row 1 we have 133/56 = 2.375 units of flush tanks, which is equivalent to the demand of a single flush valve.

Table 2.5: Equivalent Number of Demand Producing Fixtures

	Flush Valve	Flush Tank	Bath Tub
	1	2.375	2.982
	1	1.989	2.559
	1	1.789	2.346
	1	1.659	2.261
Average	1	1.953	2.537
Rounded Average	1	2	2.5

We arbitrarily assign a weight per fixture of 10 for the flush valve. Because the number of units are inversely related to load producing weight, we obtain a weight of 5 (= 10/2) for flush tank and 4 (= 10/2.5) for bath tubs in the fixture ratio of 1 : 2 : 2.5 for flush valves, flush tanks, and bath tubs, respectively. These weights are called *fixture units*. Therefore, each unit of flush valve corresponds to 10 fixture units; each unit of flush tanks has 5 fixture units, and each unit of bath tub has 4 fixture units. Using these fixture units, Figure 2.2 curves 1,2, and 3 are plotted as demand (gpm) against fixture units in Figure 2.3.

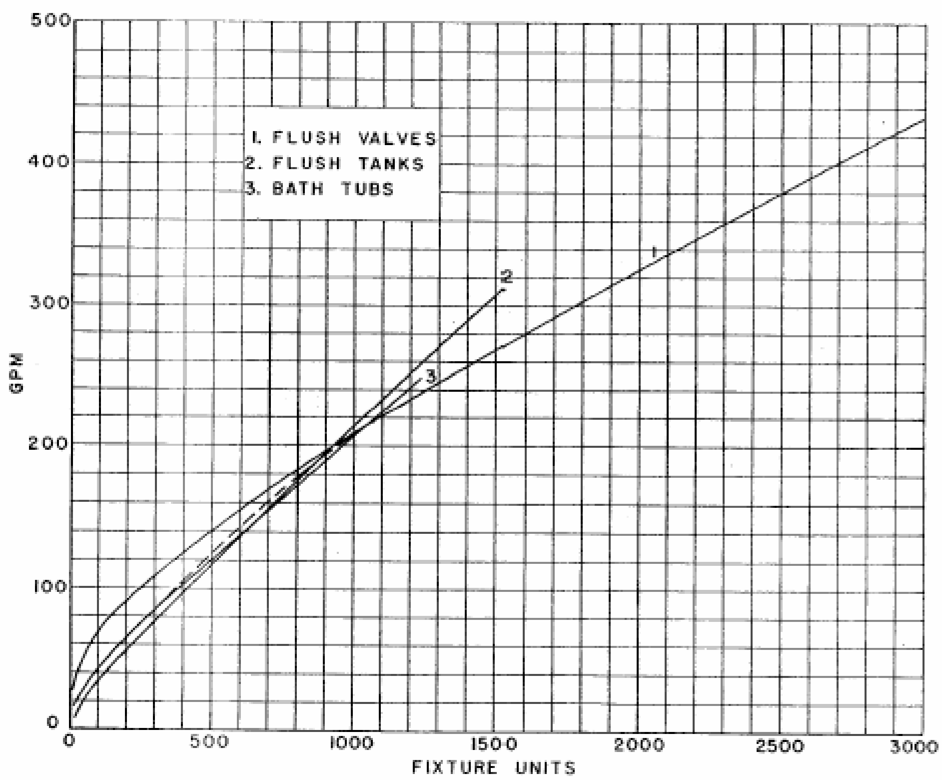

Figure 2.3: Relation of Demand to Fixture Units (Hunter, 1940)

For large values of fixture units, all the three curves collapse into one; however, flush valve and flush tank/bath tub curves show slight discrepancies from 0 to 1000 fixture units. The flush tank curve has slightly larger flowrate values within this range. Therefore the flush tank and bath tub curves are combined and the flush valve curve is kept separate from 0 to 1000 fixture units. Figure 2.4 shows the final product of Hunter's work, which is commonly referred to as "Hunter's Curve".

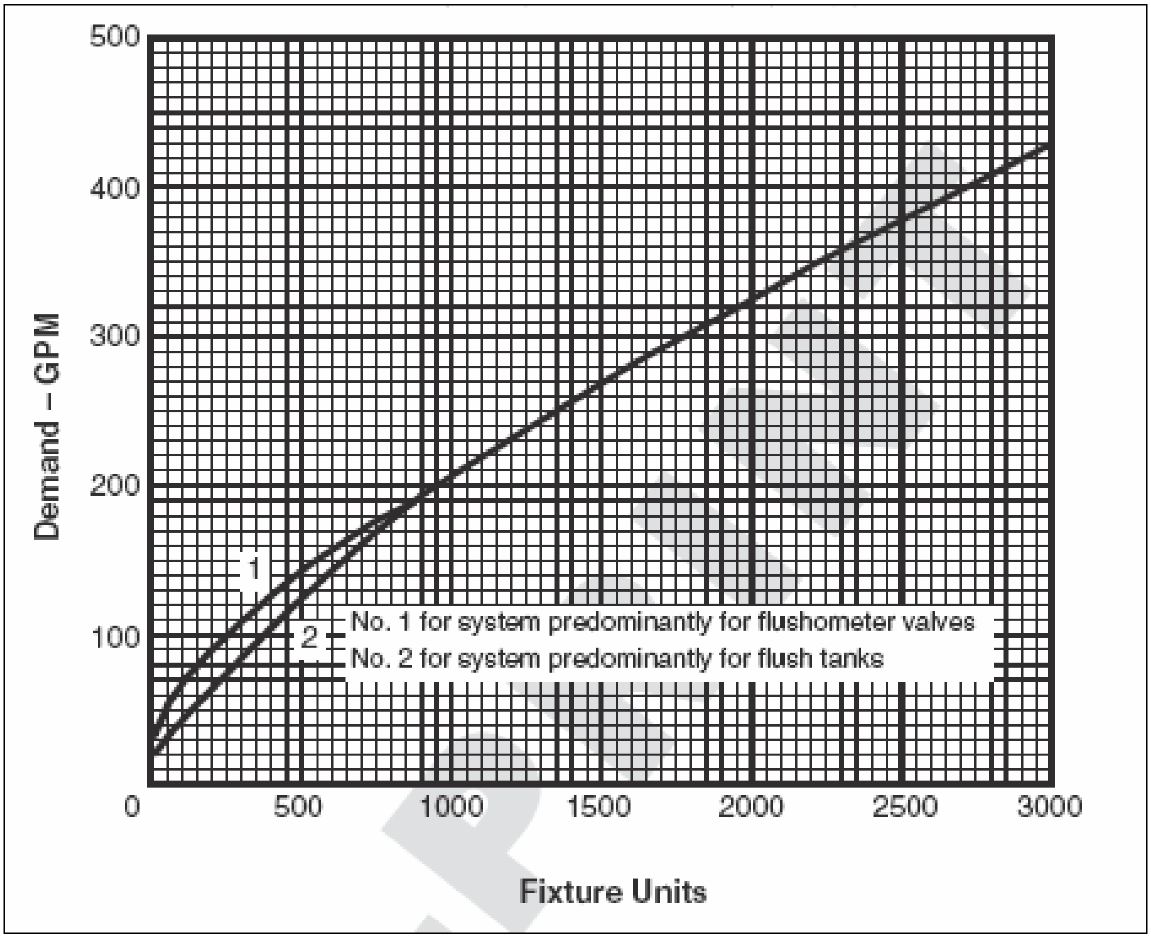

Figure 2.4: Hunter's Curve (Hunter, 1940)

Hunter's derivation was based on only three simple plumbing fixtures. Clearly, residential occupancies within the United States are typically outfitted with several different types of water-demanding fixtures. Fixture unit values have been derived for various common fixture types and are shown in Table 2.6. These values have been adapted from those shown in the IPC.

Table 2.6: Fixture Units for Various Fixtures

Fixture	Occupancy	Type of Supply Control	Demand Load Values (Water Supply Fixture Units)		
			Cold	Hot	Total
Bathroom Group	Private	Flush Tank	2.7	1.5	3.6
Bathroom Group	Private	Flush Valve	6	3	8
Bathtub	Private	Faucet	1	1	1.4
Bathtub	Public	Faucet	3	3	4
Bidet	Private	Faucet	1.5	1.5	2
Combination Fixture	Private	Faucet	2.25	2.25	3
Dishwashing Machine	Private	Automatic		1.4	1.4
Drinking Fountain	Offices, etc.	3/8" Valve	0.25		0.25
Kitchen Sink	Private	Faucet	1	1	1.4
Kitchen Sink	Hotel, Restaurant	Faucet	3	3	4
Laundry Trays (1 to 3)	Private	Faucet	1	1	1.4
Lavatory	Private	Faucet	0.5	0.5	0.7
Lavatory	Public	Faucet	1.5	1.5	2
Service Sink	Offices, etc.	Faucet	2.25	2.25	3
Shower Head	Public	Mixing Valve	3	3	4
Shower Head	Private	Mixing Valve	1	1	1.4
Urinal	Public	1" Flush Valve	10		10
Urinal	Public	3/4" Flush Valve	5		5
Urinal	Public	Flush Tank	3		3
Washing Mach. (8 lbs)	Private	Automatic	1	1	1.4
Washing Mach. (8 lbs)	Public	Automatic	2.25	2.25	3
Washing Mach. (15 lbs)	Public	Automatic	3	3	4
Water Closet	Private	Flush Valve	6		6
Water Closet	Private	Flush Tank	2.2		2.2
Water Closet	Public	Flush Valve	10		10
Water Closet	Public	Flush Tank	5		5
Water Closet	Public or Private	Flushometer Tank	2		2

The relation between Hunter's curve and the International Plumbing Code (IPC) is displayed in Appendix E of the IPC. Section E102 of the "Sizing of Water Piping System" Appendix refers to Table E102, which is a list form of Hunter's curve values (these values correspond identically with those shown in Figure 2.4). This table can be used by designers to "Estimate the supply demand of the building main and the principal branches and risers of the system" (IPC, 2000). Although this relationship is not clearly stated within the code, it is verified by checking the demand (gpm) for a fixture unit value of 1000. Table E102 shows a demand of 208 gallons per minute. Hunter's derivation shows 1000 fixture units producing a demand of 210 gallons per minute. Thus, the association between Hunter's curve and the IPC is validated.

Hunter's curve breaks down probabilistic water supply system demand into a single, easy-to-use demand curve. But, as with any simple method, there are disadvantages that need to be addressed.

One disadvantage of utilizing Hunter's curve is the elapsed time between its conception and the publication of the IPC. Over the past sixty years new technologies and ideologies have been developed regarding the design of plumbing distribution systems. An example is the movement of the industry towards low-demand fixtures. For example, a flush valve (Hunter's Type A fixture) in the 1940's had a flow rate of q = 27 gpm, flow time, t, of 9 seconds, and a recurrence time, T, of 300 seconds. The probability of use of this fixture type is S = 0.03. Contemporary flush valves are currently restrained to a flow volume of 1.6 gallons over a period, t, of 4 seconds (Breese, 2001). The reduction in flow time causes a decrease in the probability of use and flow rate to S = 0.013 and q =24 gpm, respectively. A comparison can be made using these new values. The results are given in the following tables.

Table 2.7: Total Demand Values for n = 50 Total Fixtures

	Unit Flow Rate	Prob. Of Use (S)	# Fixtures on, *m* (n = 50 fixtures)	Total Demand = *mq*
	(gpm)			(gpm)
OLD	27	0.03	5	135
NEW	24	0.0133	3	72

Table 2.8: Total Demand Values for n = 100 Total Fixtures

	Unit Flow Rate (gpm)	Prob. Of Use (S)	# Fixtures on, *m* (n = 100 fixtures)	Total Demand = *mq* (gpm)
OLD	27	0.03	8	216
NEW	24	0.0133	5	120

The above values were found using the binomial solution method. The new flush valve values cut the total demand value by 47% and 44% for *n* fixtures of 50 and 100, respectively. This is an example for a flush valve fixture type only, but it does show the dramatic effect the flow time, *t*, and the flow rate, *q*, values have on the final water demand.

Another shortcoming of the Hunter Method is its reliance on the probability of use parameter "*S*". *S*, as discussed in the above derivation, is described by the "t/T" value, where "t" represents the time the fixture is in use, and "T" is the time between each consecutive use. Hunter (1940) originally developed values for "t" based on reports by the Subcommittee on Plumbing of the United States Department of Commerce Building Code Committee, which was published in 1924. Other experiments also played a role, including Thomas R. Camp's (1924) paper "The Hydraulics of Water Closet Bowls and Flushing Devices" and unpublished reports from the National Bureau of Standards for the Plumbing Manufacturer's Research Associateship. This information helped in approximating "t/T" values for water closets, which are dependent on the rates of supply and volumes provided to plumbing fixtures. Probability of use values, *S*, for faucet controlled valves, such as those in bathtubs, are more complex to obtain. This is largely due to individual customer preference on water usage. Hunter developed *S*-values for bathtubs during situations of "congested service". Values were derived from experiments and assumed average rates of supply. This type of approach yields inherent uncertainty for usage values. Breese (2001) supports this stating that the value “T” is difficult to accurately quantify. Buildings do not all have the same common use, and thus create differences in water usage patterns. For example, a hotel will have many people waking and showering in the early morning. The time between shower usages may break down and create demand overloads. This type of occurrence violates the binomial distribution’s assumption of random use (Breese, 2001). Another example was cited by Lynn Simnick (2004), who is the Secretary of the Division of the IPC under the International Code Council. She stated that a sports stadium, where large demand loads occur during intermissions, could result in a breakdown of Hunter’s method. The IPC does not directly address this issue in its Appendix E - “Sizing of Water Piping System”. This inadequacy can lead to design inefficiencies and failures. This disadvantage is shown in the above Tables 2.7 and 2.8, where the new flush valve (Type A) fixtures have a smaller probability of use due to the decrease in flow time, *t*. A small change of “t/T” can have large impacts on the system requirements. Caution is necessary when using set values for *S*.

2.3 American Water Works Association – “Fixture Value Method”

The AWWA method of estimating water demands is presented in their M22 Manual titled, “Sizing Water Service Lines and Water Meters”. Referred to as the “Fixture Value Method”, this approach is an empirically derived technique. The motivation for the development of the Fixture Value Method can be attributed to AWWA statement that, “experience has shown that in many cases the Hunter curve approach overestimates demands in the buildings to which it is supplied” (AWWA, 2004).

The M22 manual provides peak demand curves for specific building categories. These curves are derived from “mechanical data loggers used to collect peak flow data” (AWWA, 2004).

A fixture value is defined as “simply the best estimate of the peak instantaneous demand of a given fixture or appliance” (AWWA, 2004). The M22 Manual also states that these values “represent the peak flow in gallons per minute of each fixture or appliance when it is operated without the interference of other fixtures at 60 psi”. This approach yields fixture values that are specific to each fixture type (this refers to all different makes and models of plumbing fixtures, not Hunter’s three model fixture types), and are represented in gallons per minute. For example, the M22 Manual suggests a fixture value of 35 gpm and 4 gpm for water closets with flush valves and flush tanks, respectively. Designers can also modify fixture values based on personal preference. The application of fixture values to peak demand loadings is quite different than Hunter’s technique.

Using the aforementioned mechanically logged flow data, the AWWA was able to compile peak flow measurements for different customer classes. These customer classes include residential, apartments, hotels, commercial, and public (AWWA, 2004). The various classes are shown on different curves (see Figures 2.5 and 2.6), and allow the fixture value method to account for the diverse water usages characteristic of different customer types. Peak demand graphs are created by plotting the measured average peak flow rates per customer class versus the cumulated fixture value. The resulting pair of graphs represent “Probable customer peak water demands vs. Fixture values”. These curves depict “low-range” (under 1,300 combined fixture values) and “high-range” (up to 13,000 combined fixture values) conditions. Figures 2.5 and 2.6 have been adapted from AWWA’s low-range and high-range fixture value curves. The M22 manual should be consulted to obtain actual numerical data associated with each curve.

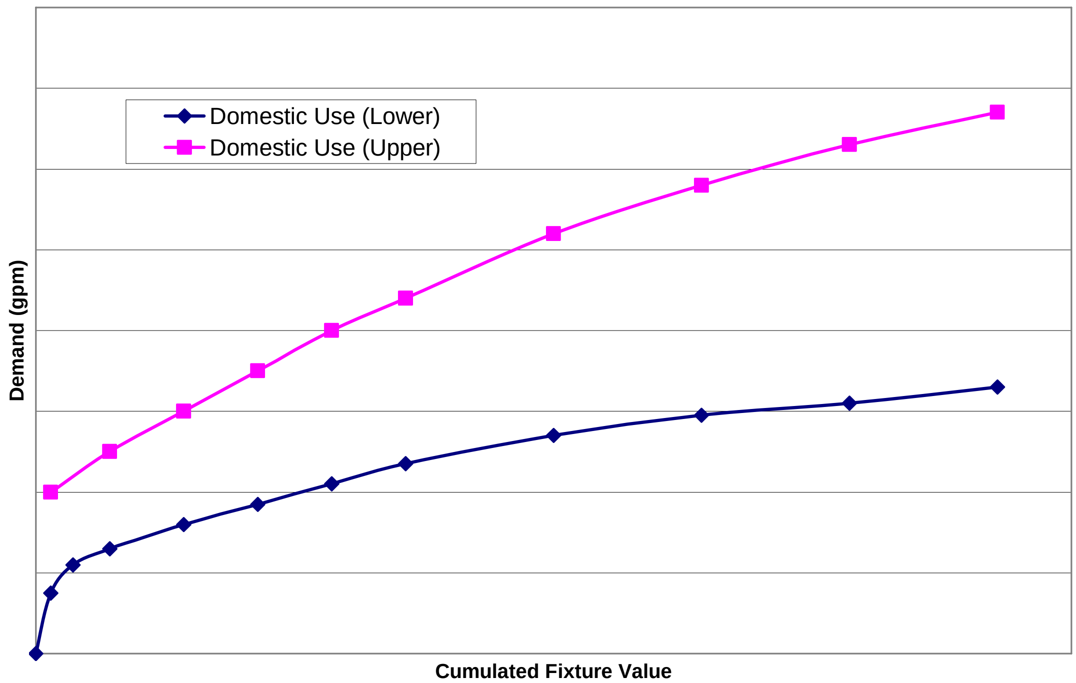

Figure 2.5: Demand vs. Cumulated Fixture Value – Low Range

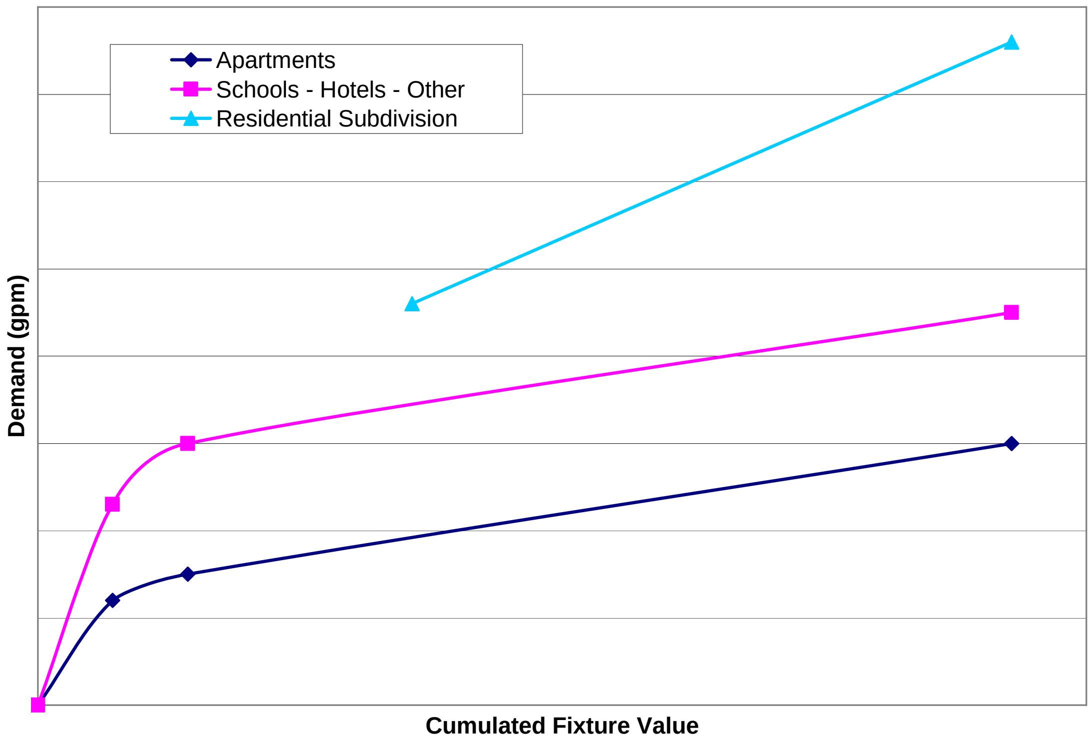

Figure 2.6: Demand vs. Cumulated Fixture Value - High Range

The fixture value curves represent demand loadings attributed to "domestic use" only. Figure 2.5 shows an upper and lower curve for domestic uses. The lower curve shows small-scale residential-like occupancies (small-scale refers to buildings that are not typical single family houses). It is comprised of buildings such as apartments, condominiums, motels, and trailer parks. The upper curve represents commercial-like occupancies. It includes hotels, shopping centers, restaurants, public buildings, and hospitals. Figure 2.6 also displays demands for domestic uses. The lower "apartment" curve represents the aforementioned small-scale residential occupancies and the middle "Schools – hotels – other" curve shows the commercial demands. The upper "residential subdivision" curve is reserved for larger-scale residential occupancies. Domestic use refers to “that water demand that is caused by the single and multiple use of plumbing fixtures and wash-down facilities” (AWWA, 2004). Continuous demands, like those loadings associated with irrigation systems (i.e. lawn sprinklers, hose bibs, etc.) or mechanical equipment, must be added separately to the peak domestic water demand.

The fixture value method was originally based on a water meter outlet pressure of 35 psi (AWWA, 1975). The second edition (2004) changed this value to 60 psi. This is an important parameter due to the relationship between pressure and flowrate at fixture outlets. Plumbing fixtures that do not have pressure regulating valves are susceptible to varying demand flows. This has a direct impact on demand estimation, and the M22 Manual accounts for this with a table of “Pressure Adjustment Factors”. These values are shown in Table 2.9.

Table 2.9: AWWA M22 Pressure Adjustment Factors (AWWA, 2004)

Working Pressure at Meter Outlet (psi)	Pressure Adjustment Factor
35	0.74
40	0.80
50	0.90
60	1.00
70	1.09
80	1.17
90	1.25
100	1.34

Adjustment factors must be applied to systems that rely on outlet pressures deviating from the 60 psi value. The adjustments are applied directly to the probable peak demand flow for domestic uses.

The M22 Manual (1975) lists the following procedure estimating customer demand:

1. Required system characteristics:

    a. Pressure at the water meter outlet
    b. Type of customer (i.e. customer class)
    c. Number and type of fixtures
2. Determine combined fixture value:
    a. Total the number of similar fixtures and multiply by their respective fixture value
    b. Sum all fixture values for each type of fixture in the system
3. Determine "Probable customer peak water demand" using the applicable low-range or high-range graph at the water meter outlet.
4. If the design pressure at the meter is above, or below, the 60 psi design value, a pressure correction factor must be used. Simply multiply the peak water demand by the pressure factor (See Table 2.9).
5. Add any continuous demands to the domestic loading to find the total customer peak demand

The result from this procedure provides the designer with an empirically derived value for the "probable customer peak water demand". The demand value is unique to the calculated combined fixture value and specified customer class.

2.4 Hunter's Method versus AWWA's Fixture Value Method:

There are many differences between Hunter's method and the AWWA method that must be discussed.

First, Hunter's approach is the integration of empirically derived fixture use data with a theoretical probability model, namely the binomial distribution. His procedure is based on "congested use", which is reflected in his choice of a 1% failure rate (Hunter, 1940). Konen (1980) states, "what Hunter did was to project the number of fixtures of one type that would operate simultaneously with the conditions that this number would be exceeded not more than one percent of the time". He then utilized the observed demand ratio of 1:2:2.5 between the three common fixture type (flush valves, flush tanks, and bathtubs, respectively) to collapse the data into a single curve (i.e. Hunter's curve). The M22 Manual is a pure empirical approach for estimating water distribution system demand loadings. Both the independent variable (fixture value) and the dependent variable (average peak flow rate) are based on measured observations. Water meter data points representing average peak flows are plotted against combined fixture values. A trend line is fit between the data points to create a continuous relationship between the ordinate and abscissa values.

Second, the AWWA fixture value method presents different graphs for varying customer types. Figure 2.5 and 2.6 display the low-range and high-range demand plots. These compartmentalized curves provide the engineer with a robust method for designing a diverse range of building classes. The Hunter method does not directly provide values for different customer class types. It can be argued that Hunter's method does present, indirectly, a classification scheme. Commercial and industrial applications will typically be dominated by flush valve fixtures. Residential systems are commonly flush tank, depending on their size. In Hunter's BMS 65 publication, he has included two separate curves for flush valve and flush tanks systems. The IPC also provides these values in a tabular format in Appendix “E” of the 2000 code. A designer can successfully apply both flush-tank and flush-valve curves to a single system, but must be prudent to account for system diversity. In this case, system diversity refers to networks utilizing both flush tanks and valves.

Third, the fixture value method also includes a provision for adjusting demand based on varying pressure (at the water meter). This design stipulation is supported by the emitter equation, a derivation of the orifice equation, which relates flow rate and flowrate through the use of an emitter coefficient. It is stated in Equation 3.

$$Q = K\sqrt{P} \qquad (3)$$

Q is the flowrate (gpm), K is the emitter coefficient, and P is the pressure drop over the orifice (psi). The Hunter method does not include any such pressure adjustment option. Fixture unit values were derived for constant fixture supply rates. The basis of these values lies on the selection of the fixture’s flow time, t, and the flowrate, Q, supplied to the fixture. For flush valves, whose flow time values, t, can be reasonably approximated due to valve mechanics and pressure regulators, these constant supply rates work reasonably well. Other fixtures supply rates heavily rely on individual usage preferences and flow pressures.

Fourth, a comparison between AWWA’s M22 method and Hunter’s method shows large differences in peak water demand values. These discrepancies are of interest because both methods attempt to model the same phenomenon, namely customer water demand. Furthermore, when the two methods are compared against actual observed flows, AWWA’s empirical approach has a much better correlation. Figure 2.7, adapted from the M22 manual, shows the three curves superimposed upon one another. The M22 should be consulted for numerical comparison.

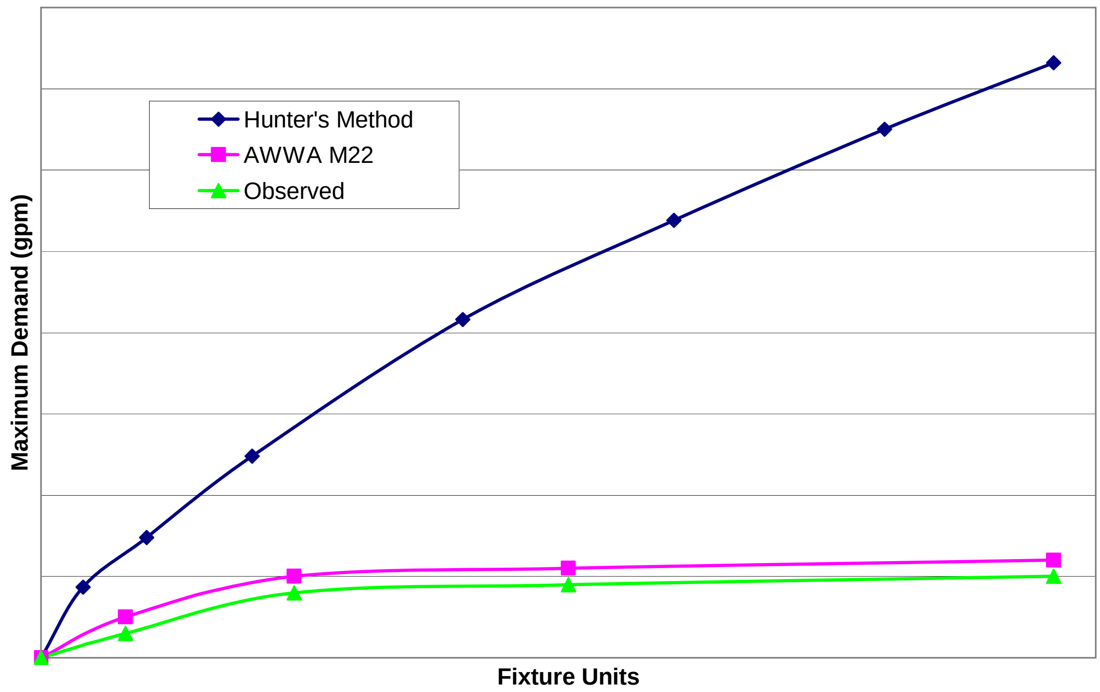

Figure 2.7: Hunter's Curve versus AWWA M22 Curve

Here, Hunter's curve is based on the demand values corresponding to a flush tank dominated plumbing system. The AWWA curve is derived from the fixture value demands shown in Figures 2.5 and 2.6 for multifamily housing (i.e. apartments), which have been converted to equivalent fixture units. The observed curve was produced from recorded water meter data from 36 multi-family buildings in the Denver, CO area (AWWA, 2004). It is clear that Hunter's method, in this situation, is drastically overestimating peak demands. Referring to Tables 2.7 and 2.8, which show the effects of reduced flow plumbing fixtures on Hunter's demand values, we can cite a possible cause for this discrepancy. Furthermore, the M22 method is based on water meter data points representing average peak flows. Hunter's method, due to it reliance on the binomial probability distribution, is based on the most probable instantaneous peak flow. This fundamental difference between the two methods may cause the Hunter's curve to be unfairly compared against the M22 curve.

Finally, it is important to note that the AWWA method was developed to size water service lines only. This is apparent by citing the experimental procedures used to acquire flow data. Measurements were taken at the water meter, and not at individual supply lines or fixtures within the distribution system. Sizing smaller branches becomes a problem due to the poor resolution of the "low-range" curve for smaller numbers of fixture values (see Figure 2.5). This is not to say that the fixture value method cannot be used for smaller branch applications, but its accuracy may be suspect. In general, the best use for the fixture value method is for water service lines. We also endorse the use of the M22 method for larger branches within the system. Hunter's method tables, found in the IPC, display demand estimates for fixture unit values as small as one. This small-scale resolution allows for the sizing of even the smallest branches within the distribution system. But, as mentioned with the AWWA method, small-scale applications for Hunter's method may be suspect, as well. This statement can be validated by using an example with hypothetical *S* values. Here, a branch is serving 5 water closets fitted with flush valves in a public restroom. The hypothetical probability of use is 0.1 (Hunter used 0.03). The limited failure criterion is set at 1%, meaning the system will overload only one percent of the time. A table of cumulative binomial probabilities states that only two fixtures need to be "on" for the system to exceed 99% cumulative probability.

Conversely, if three or more fixtures are running simultaneously, the system is automatically overloaded, and Hunter's method "fails". As the number and type of fixtures in a system becomes larger and more diversified, this discrepancy becomes less pronounced. Regardless, the example supports the claim that Hunter's method may be inaccurate for smaller sizing applications.

2.5 Selection of Appropriate Design Procedure:

Both of the above design procedures have their optimal applications. The Hunter's fixture unit method was developed specifically for sizing plumbing water distribution systems. The AWWA method was created for sizing water service lines. This is apparent in the derivations of both approaches. The Hunter curve takes into consideration the random usage of plumbing fixtures, while the fixture value method incorporates empirical data obtained at the water meter. Hunter's method allows a designer to apply fixture unit values to pipe sections serving only a few fixtures, although proven above as a questionable practice. The AWWA fixture value curve was derived from water meter data, and smaller downstream distribution lines are not fully represented in the curve's values (Figure 2.5 and Figure 2.6). This is supported by the poor resolution of the fixture value graphs. Demand (y-axis) is displayed increments of 10 gpm, and Combined Fixture Values (x-axis) are shown in increments of 100. Any points between these values will be based on visual interpolation. This selection technique provides an inadequate level of accuracy required for the distribution models employed later in this thesis.

The two methods produce peak demand flow values that are required for pipe sizing calculations. The AWWA approach, as stated above, is a method created for sizing water service lines only. The main objective of this paper is plumbing water distribution systems, which are comprised of laterals, branches, and risers. Because of this constraint, the AWWA M22 method is not considered any further.

CHAPTER 3: Plumbing System Configuration

3.1 Introduction

This chapter provides an overview of the various network configuration types that are utilized in contemporary plumbing water distribution systems. The term "configuration" refers to the logistical arrangement of a pipe network to deliver water to plumbing fixtures (this chapter does not include design procedures for pipe sizing). Although plumbing codes are used for design guidelines, they do not convey the inherent complexity of plumbing water supply systems. Codes also fail to communicate the many decisions engineers must make in the design process. For example, the International Plumbing Code (IPC) has only one statement relevant for the configuration of pressure boosted water distribution systems. It declares that pressure boosting equipment is necessary where the street main pressure is inadequate to serve buildings water demands (IPC, 2000). The three options, elevated tank, hydropneumatic tank, or booster pump, leave the designer with a number of decisions to make regarding the arrangement of the piping network. This chapter presents various network configurations and their applicability to plumbing water distribution design.

The main objective of a plumbing water distribution system is to provide all plumbing fixtures and equipment with potable water. Here, "plumbing fixtures and equipment" are represented by any device that demands water for operation. This objective is quite broad, but can be narrowed by further inspection of the problem. Harris (1990) states, "Plumbing fixtures and equipment should be provided with water in sufficient volume, and at adequate pressure, to enable them to function satisfactorily without excessive noise, under normal conditions of use". The network must be arranged such that it can fulfill the objective, based on the design criteria, within the bounds of the identified constraints.

3.2 Design Elements

Design constraints serve as limits that a proposed design cannot violate. A majority of the design constraints for a plumbing water distribution system are defined within the applicable plumbing codes (in this case the International Plumbing Code (IPC)). Pressure is the most predominant limitation of plumbing water distribution systems. There are three pressure values that bear weight on the final design. The diurnal minimum pressure at the street water main provides the available energy around which the system will be configured. The IPC (2000) states, “Where the water pressure in the public water main or individual water supply system is insufficient to supply the minimum pressures and quantities specified in this code, the supply shall be supplemented by an elevated water tank, a hydropneumatic pressure booster system or a water pressure booster pump”. Intuitively, water main pressure will vary for different locations and throughout the day. The maximum allowable static pressure (within distribution pipes) is declared by the IPC as 80 pounds per square inch (psi). This value serves as an upper pressure limit. The IPC also defines the minimum energy, or the low pressure, that must be maintained at every fixture for it to function properly. Various fixtures require different pressures, which are stated in Table 604.3 of the 2000 edition of the IPC. Table 3.1 is adapted from these values.

Table 3.1: Minimum Required Fixture Pressures

Fixture Type	Flow Rate (gpm)	Flow Pressure (psi)
Bathtub	4	8
Bidet	2	4
Combination Fixture	4	8
Dishwasher, Residential	2.75	8
Drinking Fountain	0.75	8
Laundry Tray	4	8
Lavatory	2	8
Shower	3	8
Shower, Temp Controlled	3	20
Sillcock, Hose Bib	5	8
Sink, Residential	2.5	8
Sink, Service	3	8
Urinal, Valve	15	15
Water Closet; Blowout, Flushometer Valve	35	25
Water Closet, Flushometer Tank	1.6	15
Water Closet, Siphonic, Flushometer Valve	25	15
Water Closet, Tank, Close Coupled	3	8
Water Closet, Tank, One-Piece	6	20

The fixture group with the highest elevation in the building is bound by this requirement. The pressure range between the minimum and maximum values corresponds to the operating pressures under which the distribution system will function. Plumbing systems are also constrained by hot water system requirement. The IPC (2000) states, “where the developed length of hot water piping from the source of hot water supply to the farthest fixture exceeds 100 feet, the hot water supply system shall be provided with a method of maintaining the temperature of hot water to within 100 feet of the fixtures”. The motivation for this constraint is linked to the decrease in temperature as hot water stays stagnate in a distribution pipe. Hot water is important to most customers and it should be supplied at an acceptable temperature and in a timely fashion to all fixtures that require it. Based on the foregoing discussion, the following aspects of a plumbing system are identified:

1. Minimum pressure at the water main
2. Maximum allowable static pressure
3. Minimum pressure required at highest fixture group
4. Hot water lines must not exceed 100’ in length from source

3.3 Hydraulic and Network Configuration Overview

Two design constraints, the minimum pressure at the street water main and the minimum pressure required by the highest fixture group, dictate the arrangement of a plumbing water distribution system. The difference between these values yields the “allowable amount of energy loss” a system may incur, while still retaining hydraulic functionality. Energy losses are attributed to frictional head loss. In addition, there is pressure loss when a fluid is moved in the vertical direction. This pressure loss, P, is related to the potential energy, or height, h, by:

$$P = \gamma h \qquad (1)$$

where, P is the pressure [F/L^2], γ is the specific weight of the fluid [F/L^3], and h is the height (static head) of the fluid above a reference datum [L] (Roberson and Crowe, 1993). For example, a vertically oriented pipe of 100 feet in height from the street main will require a minimum pressure of 43.3 lb/in^2 using a specific weight of 62.4 lb/ft^3, and assuming no frictional losses. Frictional losses change with varying flow conditions. They are broken into two interactions known as major and minor friction. Major friction loss occurs when flowing water interacts with the inner wall of a pipe. Major friction losses are modeled using various equations, including Darcy-Weisbach, Hazen-Williams, and Manning’s. Minor friction losses are due to flow perturbations caused by fittings, valves, or equipment. These dissipations can be estimated using empirical loss coefficients (specific to individual appurtenances) in conjunction with the Darcy-Weisbach equation, or as equivalent pipe lengths (Walski, 2003). The energy relationships are given by the energy equation (based on the first law of thermodynamics).

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

where, P is the pressure [F/L^2], γ is the specific weight of the fluid [F/L^3], V is the mean cross-sectional pipe velocity [L/T], g is the acceleration due to gravity [L/T^2], z is the elevation [L], h_p is the head gain across a pump [L], h_L is the summation of major and minor losses [L]. The term "pressure head" is given to the quantity P/γ and "velocity head" is represented by $V^2/2g$. Equation 2 can be rearranged to show the difference in pressure heads on the left-hand side and all other quantities on the right. This is shown below:

$$\left(\frac{P_1}{\gamma} - \frac{P_2}{\gamma}\right) = \left(\frac{V_2^2}{2g} - \frac{V_1^2}{2g}\right) + \left(z_2 - z_1\right) + h_L - h_p \qquad (3)$$

P_1 is the street main pressure and P_2 is the required minimum pressure at the highest plumbing fixture, which are two of the major design elements for designing plumbing water distribution systems. Equation 3 hydraulically defines the allowable amount of pressure loss a system may incur. The sum of the right-hand side of Equation 3 must be less than or equal to the difference of the pressure head values to retain hydraulic functionality. The velocity head values are typically very small compared to the pressure and elevation heads. The elevation heads are defined by the height of the building, and are considered constant values. This leaves the frictional headloss, h_L, and the pump head, h_p, values as the two major additional design factors.

3.4 System Configuration

It is necessary to discuss the various types of network configurations available to arrange plumbing water distribution systems. The first is referred to as a "serial" network, which is the simplest pipe layout. This arrangement is comprised of one water source and can have one or more intermediate nodes. There are no loops or branches in a serial network (Bhave, 1991). An example of this is shown below (oriented in the horizontal or vertical plane):

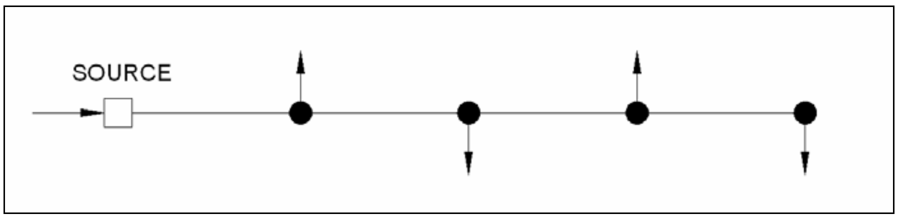

Figure 3.1: Serial Network

The second configuration is known as a "branched" or "dead-end" network, and consists of two or more serial networks. Multiple intermediate nodes allow for water to move from the upstream supply pipe to the downstream distribution pipe(s), which serve sink(s). Fluid flow is restricted to a single direction from the source to the sinks. Therefore, if an intermediate pipe is closed due to failure or repair, all downstream nodes are shutoff from the source (Bhave, 1991). Figure 3.2 shows a generic schematic of a branched network (oriented in the horizontal or vertical plane):

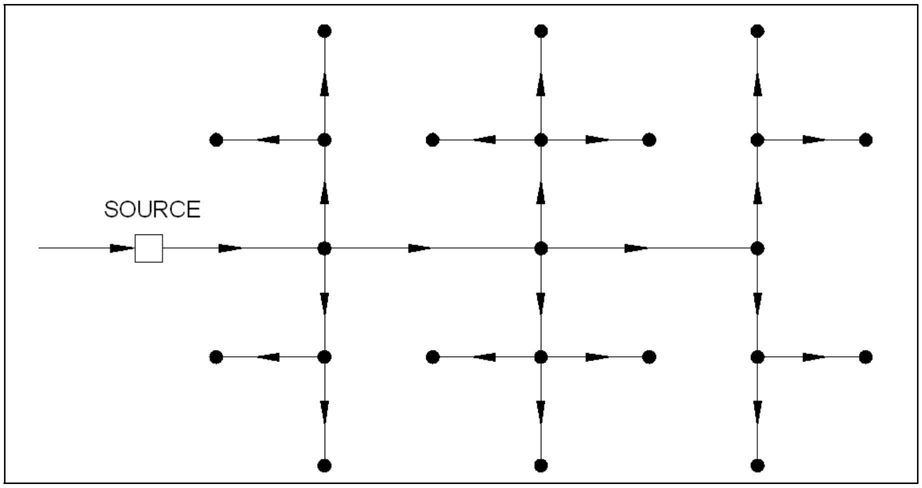

Figure 3.2: Branched Network

The third network configuration is a “looped network”. It is vital to mention that looped networks in plumbing systems should not be confused with those found in municipal (major) water systems. Municipal systems utilize loops to add robustness and reliability by creating multiple paths to a single demand node (Bhave, 1991). Plumbing systems will typically exploit loops for hot water circulation. As previously stated, the IPC requires a hot water system to maintain temperatures within 100 feet of plumbing fixtures (IPC, 2000). This situation is especially common in large buildings, such as hotels, that have long (horizontal or vertical) pipe runs. Loops are formed with branches that run from the water heater to the fixture supply lines, with a return branch back to the heater. The return line forms the loop, and water is kept continuously circulating by the means of a pump (Harris, 1990). The advantage is the decreased amount of time required to supply a fixture with water at an acceptably high temperature. The circulation loops do not improve reliability because plumbing fixtures are still served by a single distribution path (from the heater to the fixture). There are several circulation loop arrangements that may be used to satisfy the hot water constraint. Below is a representation of one of these hot water circulation systems (oriented in the vertical plane):

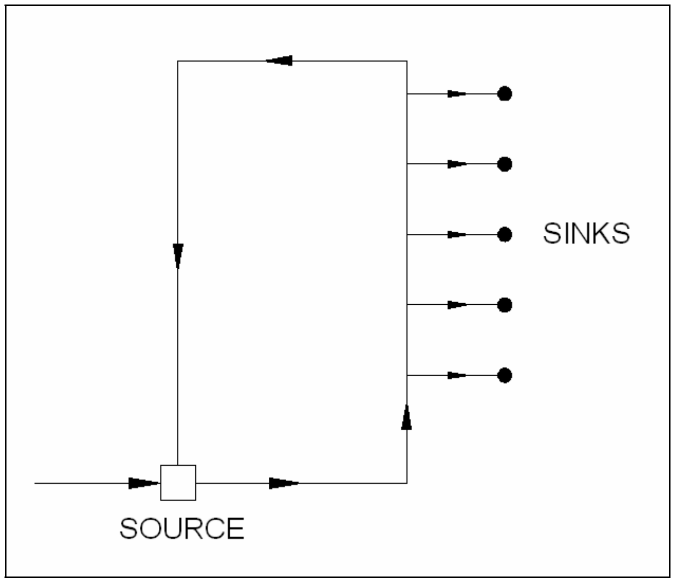

Figure 3.3: Hot Water Circulation System

These three network types, serial, branched, and looped, can be used separately, or integrated with one another. Contemporary plumbing system designs typically utilize two generalized configurations. Smaller buildings usually have branched networks for both the hot and cold water lines. Larger buildings, including horizontally extensive structures and high-rise structures, will have a combination of branched and looped arrangements.

In conclusion, the hydraulics overview provides a basis for the underlying energy relations common to water distribution networks. The energy equation states that head loss and pump-head values are the two major design factors an engineer must consider. The available network types, branched and looped, serve as building blocks for more complex applications. The confluence of system hydraulics and viable network types leads to several common plumbing water distribution configurations. These network configurations, including their hot and cold water systems, typical applications, and advantages and disadvantages, are described in the remainder of this chapter.

3.5 Low-Rise Application: Simple Upfeed Systems (Residential Households)

Simple upfeed configurations describe plumbing water distribution systems that utilize the water main for their sole energy source. These systems are the most simple of all arrangements because they do not utilize pumps or other pressure boosting equipment. Water is fed from the street main "up" through the network to the plumbing fixtures, hence "upfeed" system. Harris (1990) cites that these configurations have a "single pressure zone", meaning that all fixtures are serviced by the street water. Referring to energy equation (Equation 2), the pump-head term, h_p, is zero, implying that there is no extra energy being added to the system. Simple upfeed systems are typically comprised of branched networks for both hot and cold water lines. These "dead-end" setups are made up of three pipe classifications defined by Roy B. Hunter (BMS-65 report). The first being "laterals", which run horizontally through the system and are connected by junction nodes. The "water service line" is a specific type of lateral that runs from the "(water main) tap to the (water) meter" (AWWA (M22), 2004). The IPC requires that the water service line must terminate within five feet of entering the structure's foundation (IPC, 2000). "Risers" are situated in the vertical plane and, like laterals, are connected by junction nodes. "Branches" are those pipes that run from laterals/risers to points of demand, or "demand nodes" (Hunter, 1940). Here, the demand nodes are plumbing fixtures, which represent the "dead-ends" of the system. Typical applications for these simple upfeed systems are in low-rise buildings requiring a relatively low peak water demand and having small friction losses.

From the energy equation, if the street level pressure is not adequate and the energy losses are too high, the system will not be able to deliver water at adequate flows or pressures. The water main pressure value can vary diurnally and seasonally. Pressure variations are attributed to customer usage patterns. For example, diurnal (daily) variation occurs when large amounts of customers use water in the morning and the early evening for breakfast and dinner. Seasonal variations may occur during summer months when many people use sprinklers to irrigate their lawns, hence increasing usage. Typical street water main pressure values range from 40 to 80 psi. Because plumbing systems are served by the street water main pressure, and are designed for peak demand, the minimum street water main pressure at the water main is used to ensure a margin of safety in the design. The minimum pressure at the highest plumbing fixture is dependent on the type of fixture used to deliver the required flow. For example, it is generally taken that a flush valve will require a pressure of 25 psi to deliver 27 gpm. The height of most residential homes will not exceed two or three stories. With respect to the energy equation, the difference in elevation (z) values yields relatively small pressure-head losses. This leaves the head-loss, h_L, parameter as the major design factor in the system. Engineers and plumbers can optimize the head-loss value by employing various pipe sizing schemes through iteration (because headloss is dependent on velocity, which is governed by pipe diameter). The “Maximum Allowable Static Pressure” limitation does not typically come into play for low-rise buildings. This constraint would be applied only if the street water main pressure exceeds 80 psi, in which case a pressure reducing valve should be considered. The design limitation regarding hot water lines is also not generally applicable for low-rise buildings. Simple upfeed systems rely on “dead-end” network configurations for hot water distribution. This is acceptable because hot water pipes usually will not exceed 100’ in length. These branched arrangements do, however, allow hot water to remain stagnant in pipes while fixtures go unused. This causes customer’s to wait for the delivery of acceptably temperate water. These waiting times are commonly in the range of a couple of minutes, and hence the problem is reduced to a mere inconvenience. The large capital investment required for hot water recirculation systems is not justified in these situations. Figure 3.4 displays a typical water distribution system for a single-story, single-family residential structure.

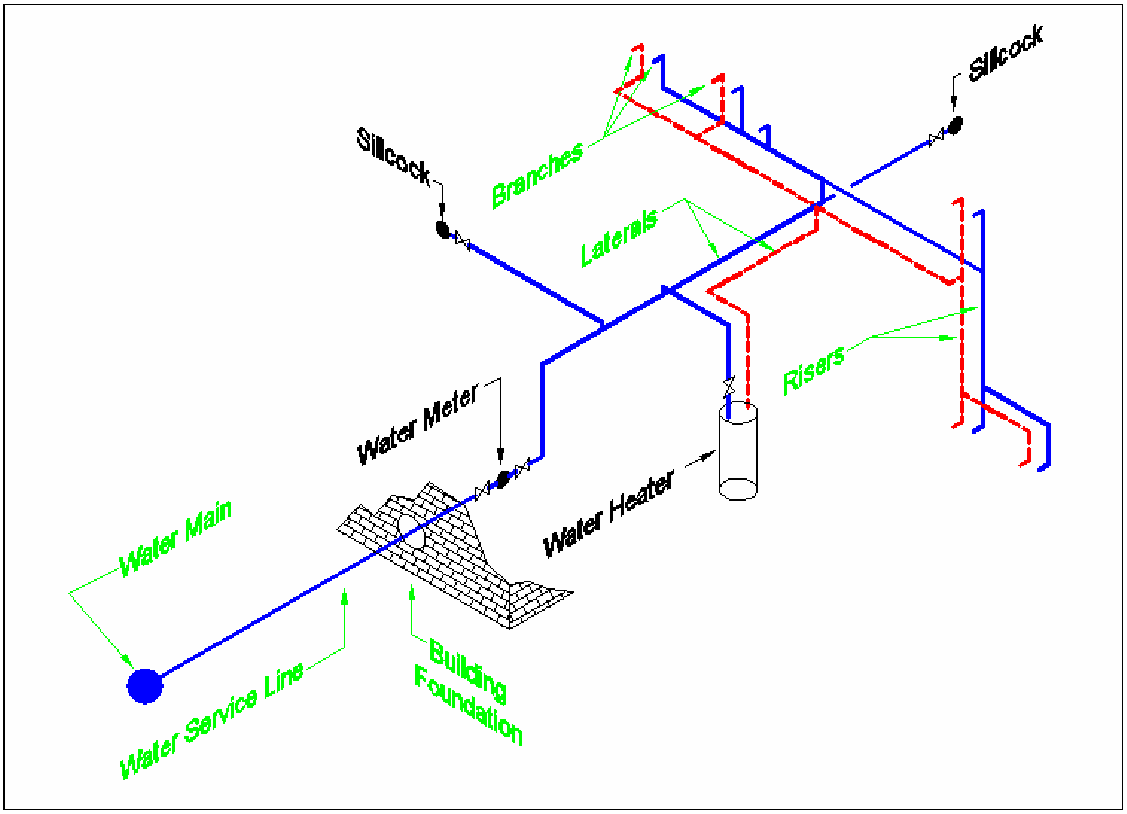

Figure 3.4: Typical Residential Water Distribution System

3.6 Medium and High-Rise Building System Cold Water Configurations

3.6.1 Design Overview

The International Building Code defines high-rise structures as, “buildings having occupied floors located more than 75 feet above the lowest level of fire department vehicle access” (IBC, 2000). Steele states that, based on engineering experience, “it might therefore be logical to classify only those buildings which exceed 250 ft in height as high-rise” (Steele, 1980 – “options”). Hydraulically speaking, these are very loose definitions, but they do preface the pressure complications encountered in medium and high-rise design. Rishel describes medium and high-rise structures as hydraulically “high-static” systems (Rishel, 2002). Figure 3.5, which shows a schematic of a high-rise plumbing system, supports this statement.

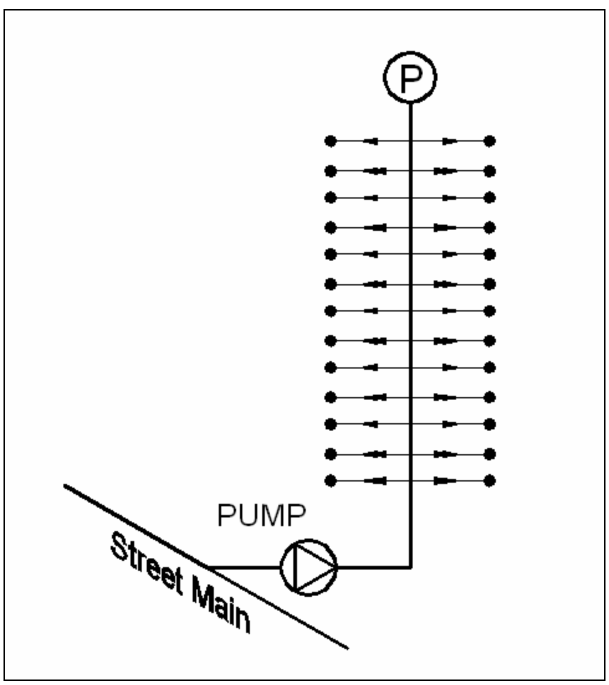

Figure 3.5: High-Static Network Configuration

“High-static” refers to the large static pressure head loss sustained when water traverses large vertical distances to reach top plumbing fixtures. These losses are analogous to the difference in elevation (z) values, which will dominate the energy equation (Equation 3). Furthermore, the difference between the street main pressure and the minimum required fixture pressure quantify the allowable energy loss through the system. If the losses exceed the allowable energy loss, energy must be added to the system. For example, if a street water main supplies a building at 50 psi, the resulting pressure head is roughly 115.5 feet. A building of 20 stories (at 10 feet per floor) will require 200 feet of pressure head to overcome the static head losses (not including the minimum required pressure at the fixtures or frictional losses). The difference in these values confirms that the pressure at the water main is completely inadequate to fulfill water demands. Additional energy is added to these networks via pumps.

One commonality between all medium and high-rise system arrangements is the utilization of pressures zones. This refers to the vertical partitioning of buildings into hydraulically independent pressure sectors. This procedure is necessitated by the IPC’s 80 psi maximum pressure requirement. The use of a single pressure zone in a high-rise application could result in static head values three or four times above the limitation. The above example calls for 200 feet of pressure head to overcome static losses, which translates to value of roughly 87 psi. This violates the maximum pressure constraint and forces an alternative configuration. Pressure zones are used as a solution to this obstacle. Zones can be configured to supply distribution piping from the bottom up (upfeed system) or from the top down (downfeed systems). Design criteria are used to decide which zone arrangement is the best option. Conventional plumbing designs include two basic configurations known as "pumped-upfeed" and "gravity-downfeed" systems. The remainder of this chapter concentrates on these two configurations. Additionally, due to the complexity of such configurations, cold and hot-water arrangements must be considered separately.

3.6.2 Pumped Upfeed Systems

Pumped upfeed systems are related to simple upfeed systems, but require a booster pump to increase the amount of energy available for distributing water. Stein states that these types of configurations are utilized in buildings that are, "too tall to rely on street main pressure, but not so tall as to necessitate heavy storage tanks on the roof" (Stein, 1992). Pumped upfeed arrangements are predominantly used in "medium-rise" buildings (Harris, 1990).

Cold-water systems in pumped-upfeed applications are configured as branched networks. Pressure zones are partitioned such that the maximum number of floors can be serviced while remaining within the design constraints. Figure 3.6, adapted from Harris (1990), shows a diagram of a typical pumped-upfeed configuration.

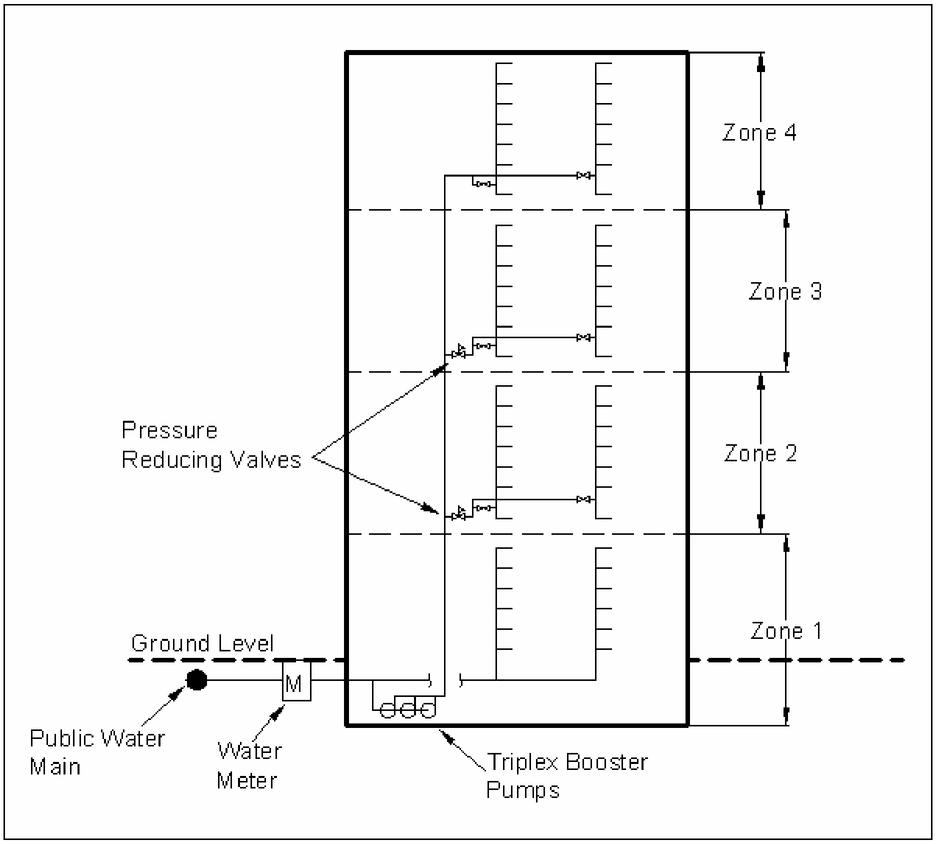

Figure 3.6: Typical Pumped-Upfeed Schematic

The diagram shows the first zone being fed by the pressure from the water main. It would be wasteful to utilize pump energy to serve this zone when adequate pressure is available from the municipality. Zones 2 through 4 are served by the booster pumps. A main riser extends from the pumps and to the inlets of zones 2 through 4. Here, the taps off the “pumped-cold-water riser” are located at the lowest elevation of each zone, and water is supplied from the bottom and fed up through the supply risers, laterals and branches. This arrangement produces the largest pressures at the bottom of the pressure zone. Energy is dissipated, in the form of frictional and static pressure losses, as the water flows against gravity to the top of the zone. Pressure reducing valves (PRVs) are utilized to regulate the inlet pressure values entering each zone (these can be seen in

Figure 3.6). The PRV is set at a value that corresponds to the maximum allowable pressure, viz. 80 psi. The vertical extent of a pressure zone is dictated by the minimum required pressure at the highest fixture group. The utilization of PRVs allows for the satisfaction of the pressure constraints. The power of employing pressure zoning lies in its ability to create independent sectors embedded within a larger system. Designers can then size pipes based on the demands per each pressure zone as compared to sizing for the entire system demands. This, in turn, reduces the required pipes sizes and hence costs less to construct the network. As a corollary, it is important to note that pressure in the main riser will exceed the IPC's 80 psi constraint. The largest pressures will be produced at the outlet of the pump discharge because this is the lowest elevation in the system (hence having a large amount of static head to pump against). The IPC allows this breach with an exception for "main supply risers" stated in section 604.8 (IPC, 2000).

Sanks (1998) states that pumped upfeed systems are good candidates for variable-speed pumps, due to their ability to function at a variety of operating points. A pump's operating point refers to the intersection of the "pump-head capacity curve" with the "system-head capacity curve" (Mays, 2001). In other words, the operating point is the amount of pressure the pump produces based on the flow demands of the system. Because the pumps are connected to plumbing fixtures through continuous runs of pipe, they must directly service the plumbing fixtures in each pressure zone. This leaves the pumps vulnerable to dynamic system demands. Moreover, varying amounts of head (h_p) must be produced to satisfy the system loadings. A variable speed pump can change its output depending on the system head conditions. Steele (1980) states (with respect to booster pump systems), "System pressure is supposed to be held at a constant level throughout the flow range. Flow demand can change quickly and frequently in the average building". Variable speed pumps, when properly sized, can deliver a wide range of water demand while maintaining "at each outlet a pressure within 2 psi of the design pressure for that outlet" (Stein, 1992). Changing demands are attributed to "system diversity", which refers to the variability of demand loadings on the distribution system (Rishel, 2002). Rischel declares a rule-of-thumb for selecting a pump: "Constant-speed for constant volume & constant head systems, and variable-speed for variable volume and variable head systems". Variable speeds allow pumps room to adjust for changing water demands. According to Stein (1992), with respect to variable speed pumps, "virtually an infinite number of delivery rates can be achieved within the zero to maximum design rate". Variable speed pump configurations are typically arranged as a "triplex" setup, where a small "jockey pump" handles low demands and operates constantly. Two larger pumps operate during increased demand loads. Frankel (1996) specifies that jockey pumps can handle 25% of the peak flow, while the two larger pumps can bear 50% of the maximum demand. The two larger pumps commonly operate on a "lead and lag" sequence. Lead and lag allows one pump to bear the brunt of the demand load while the other rests, and then the timing is reversed after a specified amount of time. This ensures that both pumps endure even wear (Stein, 1992). This triplex setup is shown in Figure 3.6.

Suction (or surge) tanks should be utilized as a buffer between the water main and booster pumps serving the structure. At peak operation (peak demand), the pumps are liable to induce large demands on the street main that could seriously reduce the available water pressure for surrounding buildings. Suction tanks are "filled by casual flow from the street main, independent of the building requirements” (Stein, 1992). Tank volumes are dictated by the daily demands of the system being served. Stein (1992) recommends suction tanks to be sized such that their volume is adequate to serve building demands during the day while only being filled by the “casual flow” provided from the street water main. Clearly, some amount of tank storage will be depleted throughout the day and will be refilled during times of low water use (nights). Stein (1992) also recommends utilizing suction tanks in situations where peak demands exceed 400 gpm.

3.6.3 Downfeed Systems

Downfeed distribution configurations, often referred to as “gravity tank” systems, consist of an elevated water tank(s) supplied by pumps. Harris (1990) cites that these arrangements are typically used in high-rise buildings. Downfeed distribution is based on lifting water to an elevated tank, then allowing that water to flow down through the distribution network under the force of gravity.

Cold-water downfeed applications are configured using branched-networks. Unlike pumped-upfeed systems, gravity-tank systems do not connect pumps directly to the plumbing fixtures. The gravity tanks (or “house tanks”) serve as intermediary buffers between the house pumps and the fixtures. This attribute is displayed in Figure 3.7, which has been adapted from Harris (1990).

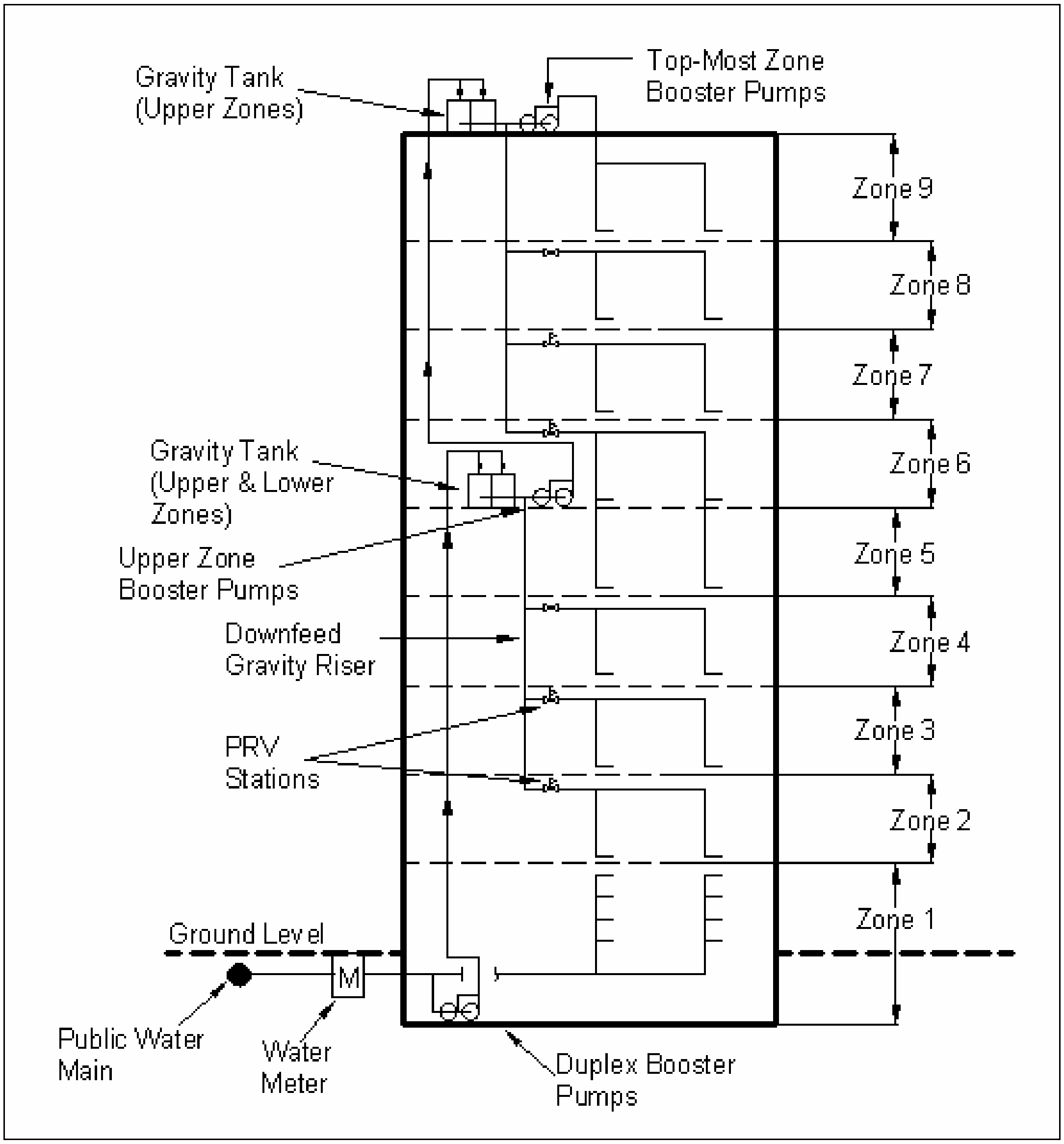

Figure 3.7: Typical Downfeed Schematic (Harris, 1990)

Figure 3.7 exhibits a typical pressure zoning arrangement for a downfeed configuration. Zone 1 is shown being served by the public water main pressure, and is independent of the downfeed network. Pressure zones 2 through 9 are arranged in a similar fashion to the pumped-upfeed system, but vertically inverted. Water is transmitted from the "house pumps" in the basement to the gravity tanks located in zone 6, which serve water demands in zones 2 through 4. The zone 6 pumps displace water from the adjacent tanks to the cisterns located in the penthouse. The upper tanks supply water to zones 5 through 9. By configuring the system with two independent supply tanks, the high-rise system is effectively cut in half. The zone 6 gravity tanks are the only common connection between the upper and lower halves. Both sections are designed completely independent of one another. The logistical operation of the system relies on the buildup of hydrostatic pressure. The “downfeed gravity main riser”, which runs from the tank to the top of the lowest zone, is directly connected to the gravity tanks. The individual branches are tapped from the riser for each zone (see in Figure 3.7). Pressure reducing valves are employed to control pressures at the top of each zone, and are referred to as “master PRV stations” (Steele, 2003). The settings on the PRVs are dictated by the minimum required pressure of the fixtures on the top zone. The vertical extent of the zones, similar to pumped-upfeed systems, is controlled by the IPC’s required maximum pressure limitation. The main risers of the system will be subjected to pressures exceeding 80 psi, but, as stated for pumped upfeed systems, is tolerated by the IPC (2000) “exception” for “main supply risers”. It is important to note that downfeed high-rise systems will employ multiple risers. A single riser configuration is not reasonable for high-rises due to the very high static heads (an 80-story building would require more than 350 psi) and large peak water demands. This is a significant difference between the aforementioned pumped-upfeed systems, which typically use a single riser for medium-rise structures.

Gravity systems typically employ constant-speed pumps to serve the demands of the water distribution system. This idea is validated by referencing Rishel’s rule-of-thumb, “Constant-speed for constant volume and constant head” (Rishel, 2002). The pumps are directly connected to the gravity tanks, and are not subject to time-variable demands of the system. In other words, the pumps are not affected by dynamic friction losses. The pumps must only provide water to the gravity tanks which hold a fixed volume and are located at a fixed elevation (i.e. constant volume and constant head). Because the pumps are used to fill the gravity tanks only, “the pumps operate at the optimum point on the pump curve for greater efficiency and less energy wastage” (Steele, 1980). Pumps are commonly configured in a “duplex” setup, where two pumps are sized to individually handle 2/3 of the peak demand. Steele (2003) states, “Each pump is sized for two-thirds of the load, so if one pump fails in the duplex set-up the other pump is capable of keeping the system in operation”. With respect to Figure 3.7, the basement pumps are sized for the total building demand, while the zone 6 boosters require a capacity for only the upper section of the building (Steele, 1980). This sizing scheme is intuitive because the basement pumps supply the “transfer/fill” gravity tanks that (directly and indirectly) feed zones 2 through 9. The topmost zone, zone 9 in Figure 3.7, cannot be sufficiently supplied with pressure from the upper gravity tanks. This is due the relatively short distance between zone 10 and the upper tanks, which therefore leads to an inadequate amount of hydrostatic pressure. Harris (1990) suggests utilizing a “tankless pumping system” to boost the static pressure to the required minimum value. Here, booster pumps draw water off the upper house tanks and directly feed the fixtures in the upper zone.

Gravity tank design is dependent on the estimated demands of the system. The volume of the tanks must be sufficient to supplement the quantity that the pump will deliver during the peak hours in buildings. The pump then continues, often for several hours, to replenish the house supply that had become partially depleted during the busy period (Stein, 1992). Gravity tanks are commonly partitioned into vertical sections. The upper division is used for potable water, while the bottom holds a reserve supply for fire protection (see Figure 3.8). As a corollary, the surface area of the tank should be large enough to allow water withdrawal (into the distribution system) with a significant decrease in the water level (Steele, 1980). A constant water level will produce, due to the hydrostatic pressure equation (Equation 1), consistent pressure values at the plumbing fixtures below.

A suction tank, as with pumped-upfeed systems, can be used as a buffer between the street main and the building's distribution network. Suction tanks are located in the basement, although Figure 3.7 does not show the implementation of such a component. Stein (1992) declares that a suction tank, utilized in a downfeed distribution application, should hold "enough reserve to allow the pumps to make up the periodic depletion in the house tanks".

3.6.4 Mixed Systems

In some cases, and especially for high-rise structures, the structure must serve different purposes for the various occupant types. For example, a high-rise structure may contain apartments, condos, hotels, office buildings, commercial stores, etc. The occupant diversity encountered in large buildings leads to complicated water situations. These complications must be addressed by the building's owner and reflected in the configuration of the distribution system. For example, the building's owner may be responsible for supplying water to the commercially zoned properties, while residentially zoned units are responsible for their own plumbing systems. These types of situations require careful solutions so that water is adequately supplied to all customers. Furthermore, the responsibilities of supplying the water should be bestowed on the correct party. Steele (2003) stresses that “no one system is best for every job”. The above simple upfeed, pumped upfeed, and downfeed configurations may be used in concert with each other to effectively supply water. A thorough study must be conducted to completely understand the demands of a building, and hence apply the best solution technique.

3.6.5 Configuration Comparisons

The above pressure boosting configurations utilize different techniques to accomplish the same objective. Each method has advantages and disadvantages. Before any design can begin, it is necessary to completely understand the attributes of each system. System characteristics are not limited to just hydraulic performance, but also include other factors such as capital expenditure, operation and maintenance, fire emergencies, and other intangibles. The strong points and weaknesses of pumped-upfeed and downfeed systems are discussed below.

I. Downfeed Configurations

  a. Advantages:

    i. These systems are relatively simple and do not require the use of complex controls to account for dynamic water demands and changing friction head values. Steele (1980) describes the simplicity of the control system by stating, “Level controls in the tank start and stop the pumps so as to maintain adequate capacity in the tank". Figure 3.8 has been adapted from Steele (1980).

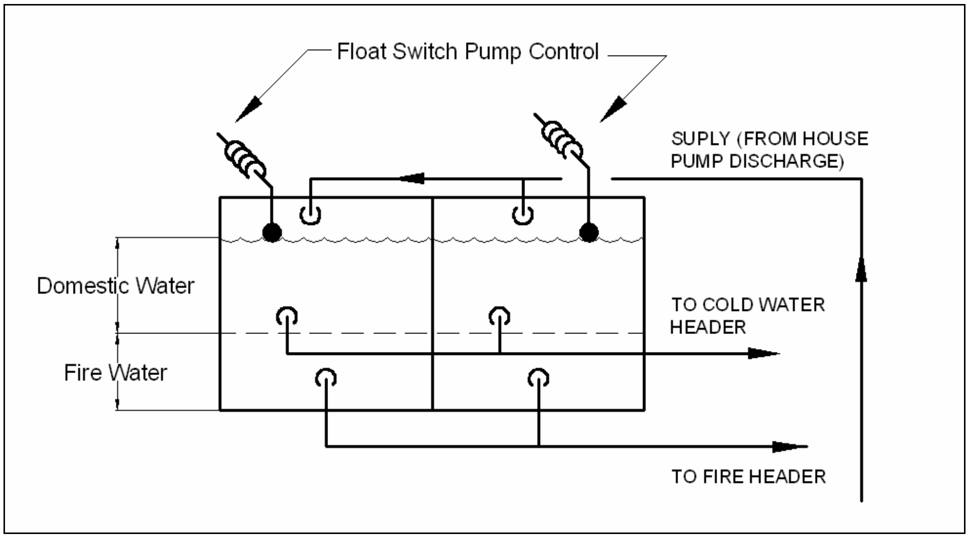

Figure 3.8: Typical Gravity (House) Tank

ii. Gravity tanks provide reserve water in case of power outages or for fire demands.

iii. The operating costs are low because the constant-speed pumps will always run at their most efficient operating point (if properly sized).

iv. Small variations of the water level in the gravity tank produce insignificant pressure variations. This leads to consistent service.

b. Disadvantages:

i. Gravity tanks are heavy and require extra structural support, which increases material costs.

1. Like suction tanks, gravity tanks are sized with respect to the total demand of the system they serve. Tank capacities may reach several thousand gallons.

ii. Large tanks require floor space that could be used for more profitable endeavors.

iii. In the case of a gravity tank rupture or failure, a large volume of water could be released into the building.

(Harris, 1990; Steele, 1980)

II. Pumped-Upfeed Configurations:

a. Advantages:

i. Less floor space is consumed by these tank-less systems.

ii. Tanks are not utilized, hence no threat of tank failure or water damage.

iii. Typically, these systems will cost less because there are no extra expenditures for tanks or extra strength structural materials.

b. Disadvantages:

i. Sophisticated controls, due to dynamic system demands, require specialists to fix and maintain the equipment.

ii. There is no reserve water in the cases of power loss or fire flow demands.

iii. These systems are less efficient because they require a pump running at even the lowest of demands. The pumps must operate outside their optimum operating point, which leads to wasted energy.

iv. Oversized pumps impose greater operating costs on the customers. Records show that pumps are oversized in more than 90 percent of all buildings installations. These non-optimized systems will run at lower efficiency, and therefore cost customers more money. (Steele, 1980)

3.7 Medium and High-Rise Building System Hot Water Configurations

3.7.1 Design Overview

Clearly hot water plumbing is an important consideration, especially for plastic pipes which may be vulnerable due to high temperature around 140° F. The limitation that "hot-water lines must not exceed 100' from the source" was not applicable in the above cold-water only sections. The temperature of cold-water is completely irrelevant with respect to plumbing codes. Simple upfeed distribution configurations utilize branched networks to supply hot water. Most times the supply pipe from the water heater to the fixtures will not exceed this 100' foot value. Figures 3.6 and 3.7 both show that medium and high-rise buildings are applications where simple dead-end networks will be insufficient. Harris (1990) states, "A long dead-end run in a hot water delivery line results in the wastage of water, since the water cools in the pipe when it is not flowing and the faucet is therefore left open until the water reaches an acceptable temperature". The IPC's regulation constraining maximum length of hot pipes to 100', coupled with the two pressure limitations, increases the complexity of these hot-water configurations. Looped circulation systems are utilized to deliver acceptable hot-water to these comparatively larger systems. When delivering hot water to multiple zones, the hot water zones should coincide with those defined by the cold water system. Like cold water arrangements, there are different circulation configurations that generate adequate hot water temperature. "A circulation-type hot-water system may be an upfeed system, a downfeed system, or some combination thereof" (Harris, 1990). The application of these various arrangements depends on the project and its individual needs and specifications. Both single and multiple pressure zone configurations are covered in the following paragraphs.

3.7.2 Upfeed System – Single Pressure Zone

One hot water arrangement, which can be used with pumped-upfeed cold water distribution, relies on an upfeed system supplying a single pressure zone. Here, a hot-water supply main (lateral) is located on the lowest floor. Multiple upfeed risers branch off of the main and distribute hot-water to the above floors. A circulating return main is located at the top of the pressure zone that collects the excess hot water from the risers. The upper main feeds the surplus hot water to a downfeed riser where it is returned to the heater. A "circulation pump" drives the flow around the circulation loop (Harris, 1990).

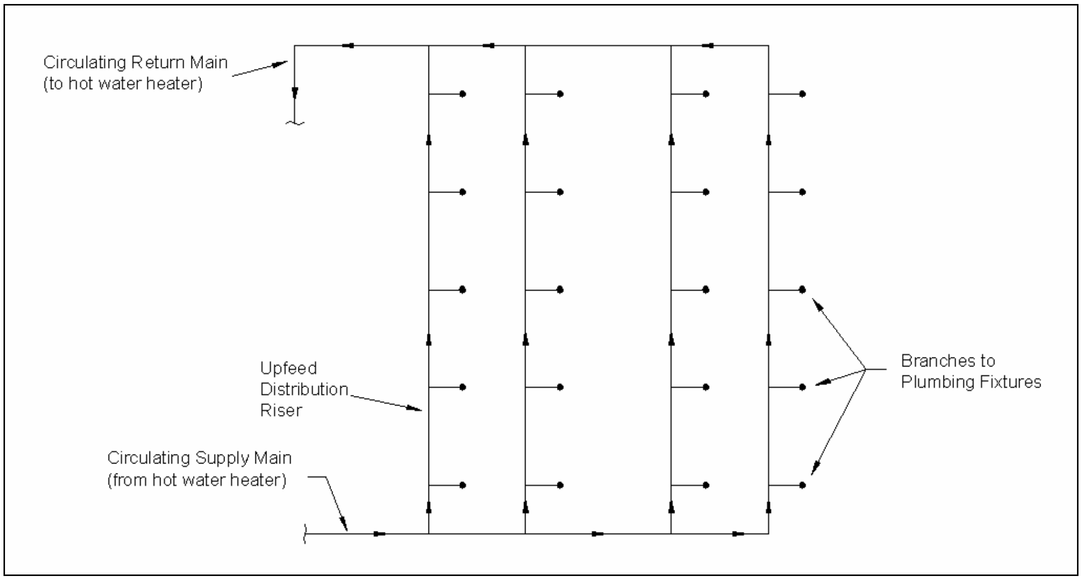

Figure 3.9: Typical Hot Water Upfeed System (single zone)

3.7.3 Upfeed/Downfeed System – Single Pressure Zone

A combination upfeed/downfeed hot water configuration can be used as an alternative to the upfeed arrangement. This design serves single pressure zones and is compatible with upfeed cold water systems. Unlike the above upfeed system, this combined arrangement locates the supply main and circulation main on the same floor. Water is fed through the hot-water supply main, via the water heater, to multiple upfeed distribution risers. The circulation loop is formed by imposing a "crossover" link at the top of the zone, which allows water to flow back through a downfeed distribution riser to serve other customers (Harris, 1990). The excess hot water is collected in a circulation main at the bottom of the zone. Water is then returned to the heater and joined with incoming cold-water. Again, the circulation process is driven by a pump.

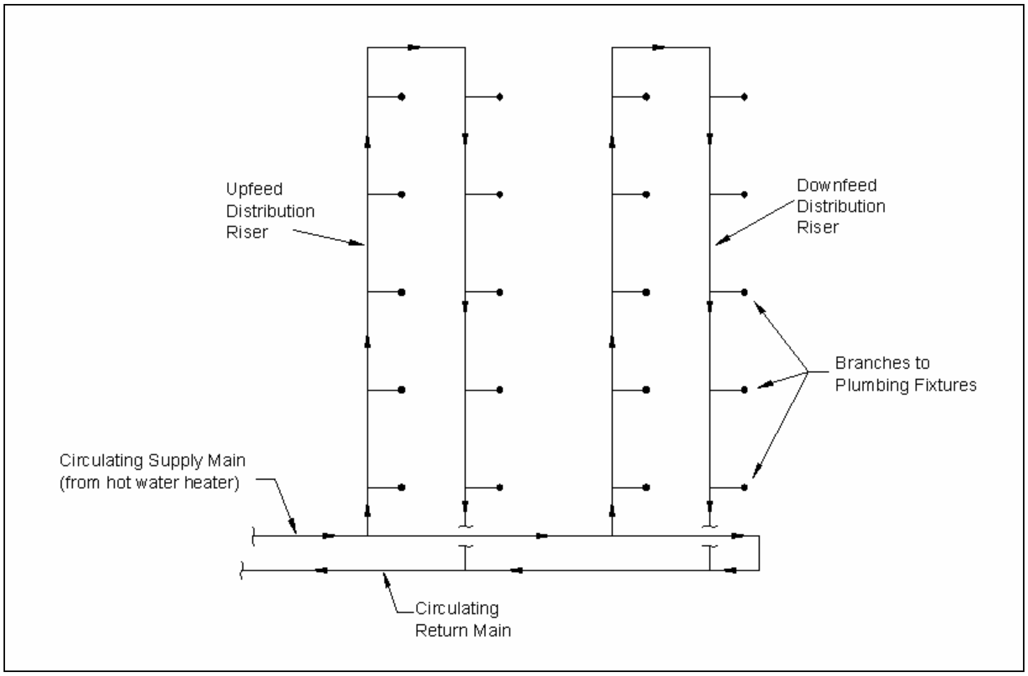

Figure 3.10: Typical Hot Water Upfeed/Downfeed System (single zone)

3.7.4 Downfeed System – Single Pressure Zone

A third arrangement used with downfeed cold water distribution, places hot water heaters below the gravity tank at the lowest points of each zone. This configuration is applicable to single zone situations. Cold water is downfed from the gravity tank to the heater. The heated water then "rises to seek its own level (discharging from the water heater and flowing through an upfeed riser) at the hot water header, becoming available there for hot water downfeed on demand" (Stein, 1992). The water is then distributed via downfeed riser(s) to the fixtures below. Special care must be taken to ensure adequate pressures at the top of zone, and to avoid excessive pressures at the bottom. Stein comments, with respect to the minimum required pressure at the top of the zone, "Commonly, 2 ½ stories, or about 35 ft, comprise the minimum pressure head above the top fixture served by any zone tank" (Stein, 1992). The excess hot water at the bottom zone is directed back to the heater where it is combined with fresh cold-water. The circulation loop is driven by a pump.

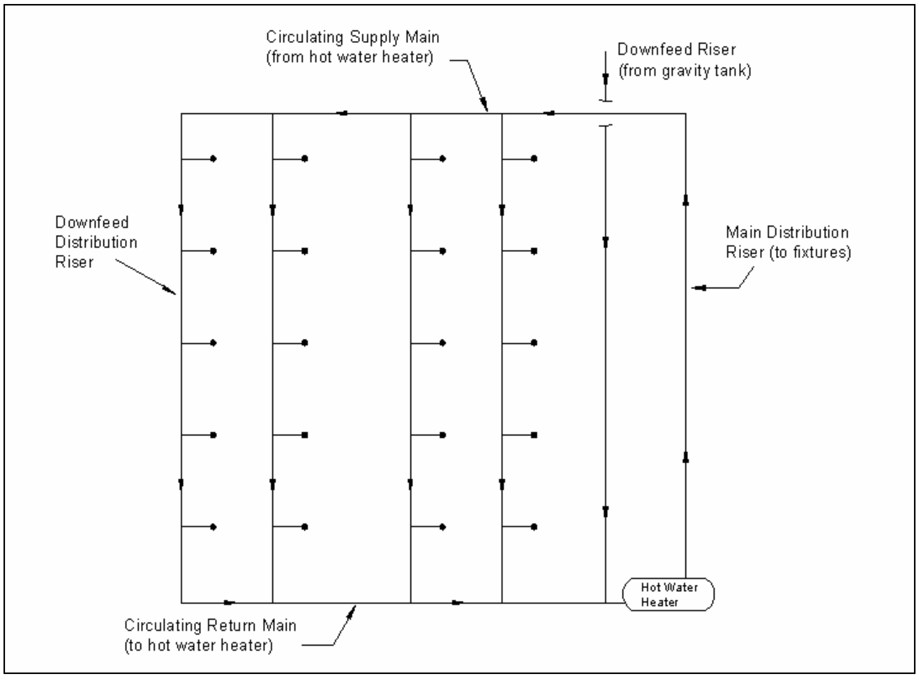

Figure 3.11: Typical Hot Water Downfeed System (single zone)

<u>3.7.5 Downfeed/Downfeed System – Multiple Pressure Zones</u>

Multiple pressure zones can be served with one hot-water circulation system. These applications utilize a "downfeed – downfeed" approach, and are typically coupled with downfeed cold-water arrangements. Here, each zone requires a pressure reducing valve, so as to reduce the pressure at the lowest floor to an acceptable level. The hot water heater is located at the top-most zone. When used in conjunction with a downfeed cold water configuration, water is pulled directly off of the gravity tank and fed to the water heater. This is ideal because heaters, which are typically expensive, are not susceptible to damages from high pressures. Steele (2003) states that water pressure at the heater should be kept below 30 psi. Hot water discharges from the heater to a main hot water riser which feeds the lower pressure zones. Each zone, except the top zone, is equipped with a PRV. The hot water is then circulated through the individual zone loop with a circulation pump, and a smaller booster heater is employed to restore any lost heat. It is important to note that the hot water is not circulated back to the main hot water riser, but is kept within the individual zone loop. These types of systems are referred to as "primary-secondary" configurations (Harris, 1990).

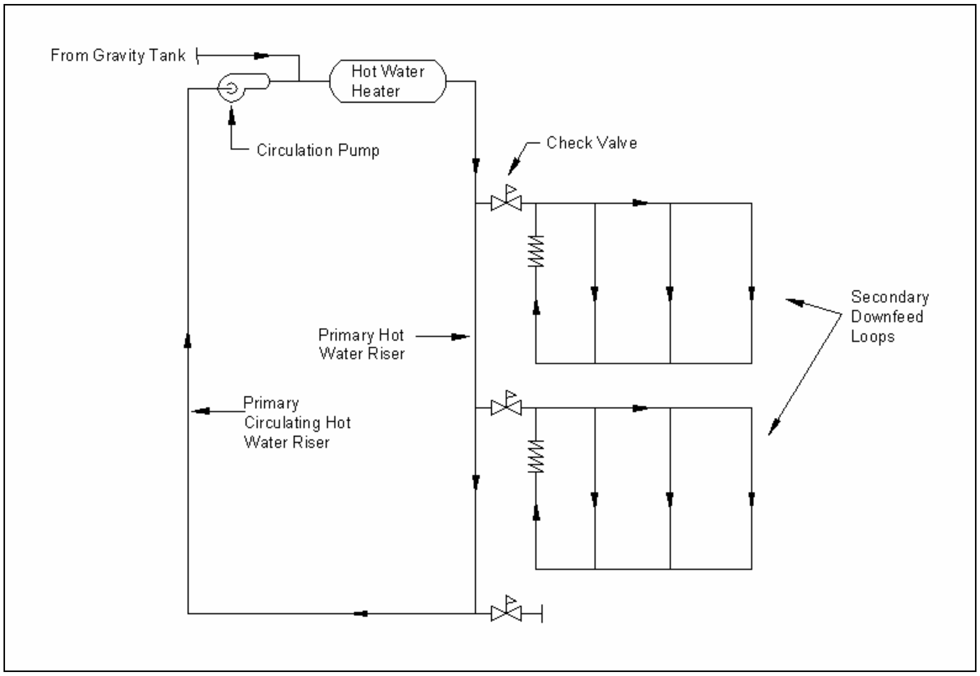

Figure 3.12: Typical Hot Water Downfeed/Downfeed System (multiple zones)

3.7.6 Conclusion

The above hot-water circulation configurations describe only a few of the feasible arrangements that could satisfy hot-water demands. The placement of circulation pumps, supply and circulation mains, and risers is up to the designer. The specified design criteria will aid in choosing the optimal application. More importantly, these looped hot-water systems satisfy the IPC's "hot-water lines must not exceed 100' from the source" requirement.

CHAPTER 4: Plumbing System Design

4.1 Introduction

The major system, typically operated by water utilities, extends from the source through the treatment plant to the street level of the minor system. The minor system is the home plumbing system. The street level pressure of the major system serves as the boundary condition in the design of the home plumbing system. It is typical practice to design the major system as a "demand driven system", in the sense that water flow must be in balance between the supply and demand nodes. Because of the pressurized network, the actual flow at nodes will depend on the energy heads at the nodes. By maintaining a minimum pressure throughout the network at all times, the requisite flow is delivered.

4.2 Role of the Energy Slope

As opposed to the major system, which is looped, the plumbing system has a branched configuration. The plumbing system design is entirely based on maintaining sufficient energy head at each node. It is achieved as follows. The hydraulically most distant point (in terms of equivalent lengths for the losses) in the plumbing system is located. This point defines the critical path, which begins at the connection point of the major and minor system, and extends to the most remote point in the minor system. By taking the energy head difference between these two nodes and dividing it by the distance, the critical slope, S_c, is obtained. Within a group of fixtures, select the fixture that has the largest operating pressure value. Theoretically, the initial slope should be the least slope among all paths. However, the traditional plumbing system approach is to take the critical slope, S_c, as that which corresponds to the most distant point.

For example, consider a street level pressure of 25 psi. The hydraulically most distant point requires a pressure of 10 psi (corresponding to the fixture with the greatest operating pressure), and is located along a 400 foot path. There is a 20 foot difference in elevation between the hydraulically most distant point and the street main (the most remote point being at a higher elevation than the street main). The critical slope is calculated as:

$$S_C = \frac{25(144) - (62.4)20 - 10(144)}{(62.4)400} = 0.037 \text{ ft/ft} \qquad (1)$$

which is also expressed in pressure unit for 100 ft lengths as:

$$S_C \text{ (pressure/100 ft)} = \frac{(0.037)62.4}{144}(100) = 1.6 \text{ psi/100 ft} \qquad (2)$$

Figure 4.1 shows a profile schematic of the example system. Point A is the street service lateral entry point at which the boundary condition pressure is maintained. Point B is the most remote point.

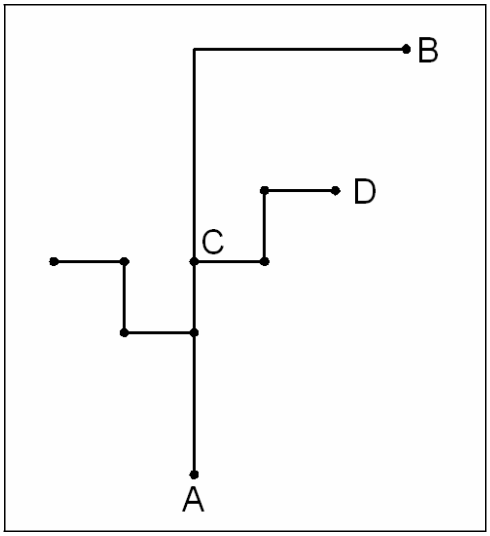

Figure 4.1: Energy Head for Initial Slope

All pipe sizes are determined using the critical slope, S_c. The reason is as follows. Because the most remote point is used in calculating S_c and the corresponding design diameter is chosen, the flow will reach from point A to point B. If every single pipe is designed using S_c, we should have the energy head at point C, H_C, as

$$H_C = H_A - (S_C)(L_{AC}) \tag{3}$$

in which, $H_A = \frac{P_A}{\gamma} + z_A + \frac{V_A^2}{2g}$ with V_A is taken as zero for a node at which two or more pipes meet. Also,

$$H_B = H_A - S_C L_{AB} = H_A - S_C L_{AC} - S_C L_{CB} = H_C - S_C L_{CB} \tag{4}$$

$$H_D = H_C - S_C L_{CD} \tag{5}$$

Point B is the most remote point and therefore, $L_{CD} < L_{CB}$, and $H_C > H_D > H_B$. Therefore, flow will take place from C to D and of course, there will be flow from C to B. By using the same critical slope, S_c, for sizing all pipes, it is granted that every node will be served. The required flow is further ascertained by choosing the suitable diameter corresponding to the flow and the critical slope.

4.3 Design of Plumbing Pipes

1. Using Table 2.6 (reproduced here for convenience as Table 4.1), assign equivalent water fixture units (wfu) for each fixture. Here there are two aspects to be considered. (a) The number of fixtures used in a building is determined by local regulations (see Table 2.1). (b) The buildings themselves are, in general, grouped as (i) residential with fixtures for household and personal care (ii) commercial including multifamily dwellings, hotels, and other small office and professional buildings (iii) large office buildings, industrial facilities, shopping centers and restaurants with fixtures determined for biological needs and (iv) schools, stadiums, transportation terminals, institutions, and similar facilities with large groups of people and specific events driven with high demands (Frankel, 1996).

Table 4.1: Fixture Units for Various Fixtures

Fixture	Occupancy	Type of Supply Control	Demand Load Values (Water Supply Fixture Units)		
			Cold	Hot	Total
Bathroom Group	Private	Flush Tank	2.7	1.5	3.6
Bathroom Group	Private	Flush Valve	6	3	8
Bathtub	Private	Faucet	1	1	1.4
Bathtub	Public	Faucet	3	3	4
Bidet	Private	Faucet	1.5	1.5	2
Combination Fixture	Private	Faucet	2.25	2.25	3
Dishwashing Machine	Private	Automatic		1.4	1.4
Drinking Fountain	Offices, etc.	3/8" Valve	0.25		0.25
Kitchen Sink	Private	Faucet	1	1	1.4
Kitchen Sink	Hotel, Restaurant	Faucet	3	3	4
Laundry Trays (1 to 3)	Private	Faucet	1	1	1.4
Lavatory	Private	Faucet	0.5	0.5	0.7
Lavatory	Public	Faucet	1.5	1.5	2
Service Sink	Offices, etc.	Faucet	2.25	2.25	3
Shower Head	Public	Mixing Valve	3	3	4
Shower Head	Private	Mixing Valve	1	1	1.4
Urinal	Public	1" Flush Valve	10		10
Urinal	Public	3/4" Flush Valve	5		5
Urinal	Public	Flush Tank	3		3
Washing Mach. (8 lbs)	Private	Automatic	1	1	1.4
Washing Mach. (8 lbs)	Public	Automatic	2.25	2.25	3
Washing Mach. (15 lbs)	Public	Automatic	3	3	4
Water Closet	Private	Flush Valve	6		6
Water Closet	Private	Flush Tank	2.2		2.2
Water Closet	Public	Flush Valve	10		10
Water Closet	Public	Flush Tank	5		5
Water Closet	Public or Private	Flushometer Tank	2		2

2. The cold water supply pipe is sized beginning with the most remote fixture from the water meter (street level lateral) and working back towards the meter. Hot water supply line sizing is also accomplished starting with the most remote hot water outlet from the water heater and working back towards the heater. For each branch calculate the branch-wise demand.

3. Calculate the critical slope, S_c. Find the pressure difference, Δp, between the street level water main and the hydraulically most distant point. The pressure

value at the most remote point is dictated by the *maximum* pressure required by any fixture at that point. For example, if a shower requiring 8 psi, a lavatory requiring 8 psi, and a flush tank requiring 25 psi are located at the most remote point, 25 psi defines the pressure. S_c is calculated as the ratio of Δp and the hydraulic length between the main and the most remote point (called the "critical path").

4. Determine the required pipe size for each branch using S_c from Step 3 and the demand from Step 2. Use the appropriate pipe material chart (Figure 4.4).
5. Use the International Residential Code's (IRC, 2000) pipe-sizing table, corresponding to the calculated pressure loss (Table 4.5), with the aid of hydraulic length and water fixture units to obtain the minimum required pipe diameters. The same hydraulic length column is used for sizing hot water pipes, as well (Ripka, 2002).
6. Use the larger of the pipe size determined between Steps 4 and 5.

4.4 Example 1:

Size the cold and hot water plumbing pipes for the system shown in Figure 4.2. The water meter is located at point A and hot water heater is located on branch CE. The most remote part of the system is point B', which is a hot water demanding node.

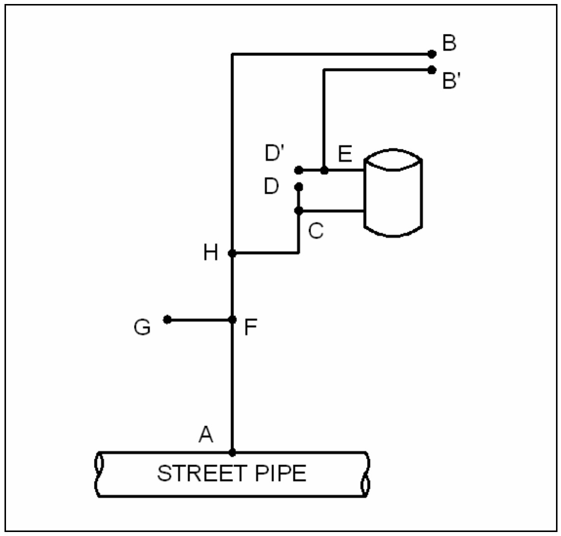

Figure 4.2: Example 1 Plumbing System

Solution (Example 1):

1. Estimate the demands.

Table 4.2: Demands for Figure 4.2 System

Demand Node ID	Fixture Type*	Cold Water (cwfu)	Hot Water (hwfu)	Total Fixture Units
B	Each Apartment 1KS, 1WC, 1BT, 1LT	1.5 + 3 + 1.5 + 1.5 = 7.5		2 + 3 + 2 + 2 = 9 @ 2 apartments = 18
	2 Apartments	15		
B'	Each Apartment 1KS, 1WC, 1BT, 1LT		1.5 + 1.5 + 1.5 = 4.5	
	2 Apartments		9	
D	1KS, 1WC, 1BT, 1LT			2 + 3 + 2 + 2 = 9 @ 1 apartment = 9
	1 Apartment	7.5		
D'	1KS, 1WC, 1BT, 1LT			
	1 Apartment		4.5	
E			13.5	
C		18		18
H		33		33
G		6		6
F		39		39
A		39		39

* KS = kitchen sink, WC = water closet, BT = bathtub, LT = laundry

2. Branch by branch cold water and hot water demands in fixture units are calculated in Table 4.3.

Table 4.3: Branch-Wise Fixture Demand

Demand Branch ID	Cold Water (cwfu)	Hot Water (hwfu)	Flush Tank Type Hunter Demand
HB	15		17.5
EB'		9	13.7
ED'		4.5	8.7
CE		13.5	16.8
CD	7.5		11.5
HC	18		18.8
FH	33		24.3
FG	6		10.7
AF	39		26

3. Required minimum pressure at B for cold water piping is 6 psi. The street level pressure is 60 psi. Therefore, the maximum pressure loss permissible for cold water piping is (60 – 6) = 54 psi. Building height is accounted for in the working pressure loss of 54 psi. The physical length AB is 50 feet. Accounting for losses, we add 20% length to obtain a hydraulic length of 60 feet. The initial slope, S_c is:

$$S_C = \frac{54}{60}(100) = 90\, psi/100\, ft$$

4. Determine the pipe sizing using S_c = 90 psi/100 ft and branch demands. Copper tubing smooth pipe Type M (Figure 4.4). Table 4.4 shows the resulting pipe diameters.

Table 4.4: Pipe Sizes – Using S_c

Demand Branch ID	S_c - Hazen-Williams Size (in)
HB	1
EB'	0.75
ED'	0.5
CE	1
CD	0.75
HC	1
FH	1.25
FG	0.75
AF	1.25

5. Use the IRC pipe-sizing table (see Table 4.5) for a pressure range of 50 – 60 psi corresponding to the 54 psi pressure loss. Use the column for a length of 60 feet. For different water fixture unit demands, read the pipe sizes from column 2 corresponding to mains and branches. Table 4.5 displays the sizing results.

Table 4.5: IRC Pipe-Sizing Table (IRC, 2000)

Meter and Service Piping (in)	Distribution Piping (in)	Maximum Development Length (feet)				
		40	60	80	100	150
0.75	0.5	3	3	2.5	2	1.5
0.75	0.75	9.5	9.5	9.5	8.5	6.5
0.75	1	32	32	32	32	25
1	1	32	32	32	32	30
0.75	1.25	32	32	32	32	32
1	1.25	80	80	80	80	80
1.5	1.25	80	80	80	80	80
1	1.5	87	87	87	87	87
1.5	1.5	151	151	151	151	151

Table 4.6: Pipe Sizes – Using IRC Pipe-Sizing Table

Demand Branch ID	IRC Table Size (in)
HB	1
EB'	0.75
ED'	0.75
CE	1
CD	0.75
HC	1
FH	1.25
FG	0.75
AF	1.25

6. Use the larger of the pipe sizes between steps 4 and 5. Except for branch ED', the diameters remain the same for all pipes between steps 4 and 5.

4.5 Example 2:

Size the cold and hot water plumbing pipes for the system shown in Figure 4.3. Note the different placement of node D. All other aspects of the system are assumed identical to Figure 4.2. The water meter is located at point A and hot water heater is located on branch HE. The most remote part of the system is point B', which is a hot water demanding node.

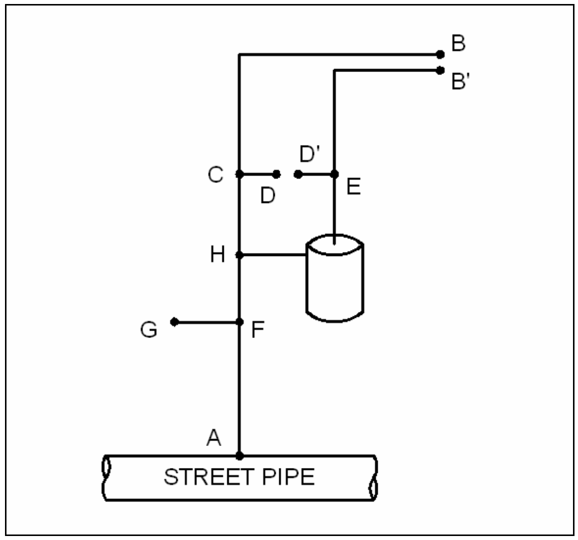

Figure 4.3: Example 2 Plumbing System

Solution (Example 2):

1. Estimate the demands.

Table 4.7: Demands for Figure 4.3 System

<table>
<tr><th>Demand Node ID</th><th>Fixture Type*</th><th>Cold Water (cwfu)</th><th>Hot Water (hwfu)</th><th>Total Fixture Units</th></tr>
<tr><td rowspan="2">B</td><td>Each Apartment 1KS, 1WC, 1BT, 1LT</td><td>1.5 + 3 + 1.5 + 1.5 = 7.5</td><td></td><td rowspan="4">2 + 3 + 2 + 2 = 9 @ 2 apartments = 18</td></tr>
<tr><td>2 Apartments</td><td>15</td><td></td></tr>
<tr><td rowspan="2">B'</td><td>Each Apartment 1KS, 1WC, 1BT, 1LT</td><td></td><td>1.5 + 1.5 + 1.5 = 4.5</td></tr>
<tr><td>2 Apartments</td><td></td><td>9</td></tr>
<tr><td rowspan="2">D</td><td>1KS, 1WC, 1BT, 1LT</td><td></td><td></td><td rowspan="4">2 + 3 + 2 + 2 = 9 @ 1 apartment = 9</td></tr>
<tr><td>1 Apartment</td><td>7.5</td><td></td></tr>
<tr><td rowspan="2">D'</td><td>1KS, 1WC, 1BT, 1LT</td><td></td><td></td></tr>
<tr><td>1 Apartment</td><td></td><td>4.5</td></tr>
<tr><td>C</td><td></td><td>22.5</td><td></td><td></td></tr>
<tr><td>E</td><td></td><td></td><td>13.5</td><td></td></tr>
<tr><td>H</td><td></td><td>27</td><td></td><td>27</td></tr>
<tr><td>G</td><td></td><td>6</td><td></td><td>6</td></tr>
<tr><td>F</td><td></td><td>33</td><td></td><td>33</td></tr>
<tr><td>A</td><td></td><td>33</td><td></td><td>33</td></tr>
</table>

* KS = kitchen sink, WC = water closet, BT = bathtub, LT = laundry

2. Branch by branch cold water and hot water demands in fixture units are calculated in Table 4.8.

Table 4.8: Branch-Wise Fixture Demand

Demand Branch ID	Cold Water (cwfu)	Hot Water (hwfu)	Flush Tank Type Hunter Demand (gpm)
CB	15		17.5
EB'		9	13.7
ED'		4.5	8.7
HE		13.5	16.8
CD	7.5		11.5
HC	22.5		20.8
FH	27		22.5
FG	6		10.7
AF	33		24.3

3. Required minimum pressure at B for cold water piping is 6 psi. The street level pressure is 60 psi. Therefore, the maximum pressure loss permissible for cold water piping is (60 – 6) = 54 psi. Building height is accounted for in the working pressure loss of 54 psi. The physical length AB is 50 feet. Accounting for losses, we add 20% length to obtain a hydraulic length of 60 feet. The initial slope, S_c is:

$$S_C = \frac{54}{60}(100) = 90\, psi/100\, ft$$

4. Determine the pipe sizing using S_c = 90 psi/100 ft and branch demands. Copper tubing smooth pipe Type M (Figure 4.4). The resulting pipe sizes are shown in Table 4.9.

Table 4.9: Pipe Sizes – Using S_c

Demand Branch ID	S_c - Hazen-Williams Size (in)
CB	1
EB'	0.75
ED'	0.75
HE	1
CD	0.75
HC	1
FH	1
FG	0.75
AF	1

5. Use the IPC pipe-sizing table (see Table 4.5) for a pressure range of 50 – 60 psi corresponding to the 54 psi pressure loss. Use the column for a length of 60 feet. For different water fixture unit demands, read the pipe sizes from column 2 corresponding to mains and branches. Table 4.10 shows the results.

Table 4.10: Pipe Sizes – Using IRC Table

Demand Branch ID	IRC Table Size (in)
CB	1
EB'	0.75
ED'	0.75
HE	1
CD	0.75
HC	1
FH	1
FG	0.75
AF	1.25

6. Use the larger of the pipe sizes between steps 4 and 5. Except for branch AF, the diameters remain the same for all pipes between steps 4 and 5.

The above two design examples highlight interesting design considerations when constructing a plumbing water distribution system. The only difference between Figure

4.2 and 4.3 is the placement of the branch that connects the cold water node D. Figure 4.2 implies that the divergence of the hot and cold water systems occurs at point C. This configuration is awkward because branch HC must carry water serving both cold water and hot water demands, while branch HB is serving only a single cold water demand at B. Figure 4.3 shows the divergence occurring at point H. This configuration effectively splits the system such that the branch extending from H to C only serves cold water demand, and the branch extending from H to E provides only hot water demands. Furthermore, a comparison of the results from Example 1 and Example 2 shows differences in both pipe sizes and total demand. Tables 4.11 and 4.12 show the differences in pipe diameters and Table 4.13 displays demand value discrepancies.

Table 4.11: Diameter Differences (S_c – Hazen-Williams Method)

Example 1: S_c Results		Example 2: S_c Results		Δ (Ex 2 - Ex 1), (in)
Branch ID	Pipe Size (in)	Branch ID	Pipe Size (in)	
HB	1	CB	1	0
EB'	0.75	EB'	0.75	0
ED'	0.5	ED'	0.75	0.25
CE	1	HE	1	0
CD	0.75	CD	0.75	0
HC	1	HC	1	0
FH	1.25	FH	1	-0.25
FG	0.75	FG	0.75	0
AF	1.25	AF	1	-0.25

Table 4.12: Diameter Differences (IRC Table Method)

Example 1: IRC Table Results		Example 2: IRC Table Results		Δ (Ex 2 - Ex 1), (in)
Branch ID	Pipe Size (in)	Branch ID	Pipe Size (in)	
HB	1	CB	1	0
EB'	0.75	EB'	0.75	0
ED'	0.75	ED'	0.75	0
CE	1	HE	1	0
CD	0.75	CD	0.75	0
HC	1	HC	1	0
FH	1.25	FH	1	-0.25
FG	0.75	FG	0.75	0
AF	1.25	AF	1.25	0

Table 4.13: Demand Differences

Example 1 Branch IDs	Demand Value (gpm)	Example 2 Branch IDs	Demand Value (gpm)	Δ (Ex 2 - Ex 1), (gpm)
HB	17.5	CB	17.5	0
EB'	13.7	EB'	13.7	0
ED'	8.7	ED'	8.7	0
CE	16.8	HE	16.8	0
CD	11.5	CD	11.5	0
HC	18.8	HC	20.8	2
FH	24.3	FH	22.5	-1.8
FG	10.7	FG	10.7	0
AF	26	AF	24.3	-1.7

Tables 4.11 and 4.12 show the pipe sizing differences between the first (Figure 4.2) and second (Figure 4.3) example. Table 4.13 shows the demand differences between the two examples. The discrepancies are attributed to the different locations of the point of divergence between the cold and hot water systems for the two piping configurations. The point of divergence is important because as Harris (1990) states, the individual “hot” and “cold” fixture unit values are typically taken as 75% of the “total” fixture unit value for a specified fixture. Figure 4.2 shows the divergence of the separate hot and cold water lines at node C. Figure 4.3 shows this divergence at node H.

It is important to note that the IRC governs construction of one and two-family dwellings only. Pipe-sizing tables, as shown in Table 4.5, are only covered in the IRC. The IPC, which governs plumbing system for all types and sizes of buildings, only endorses the use of the critical slope, S_c, and the associated Hazen-Williams figures (Figure 4.4). The following Appendix 4A provides a comprehensive step by step procedure for designing plumbing pipes. In chapter 5, we propose a “pressure-driven” formulation to evaluate the design accomplished using the critical slope procedure.

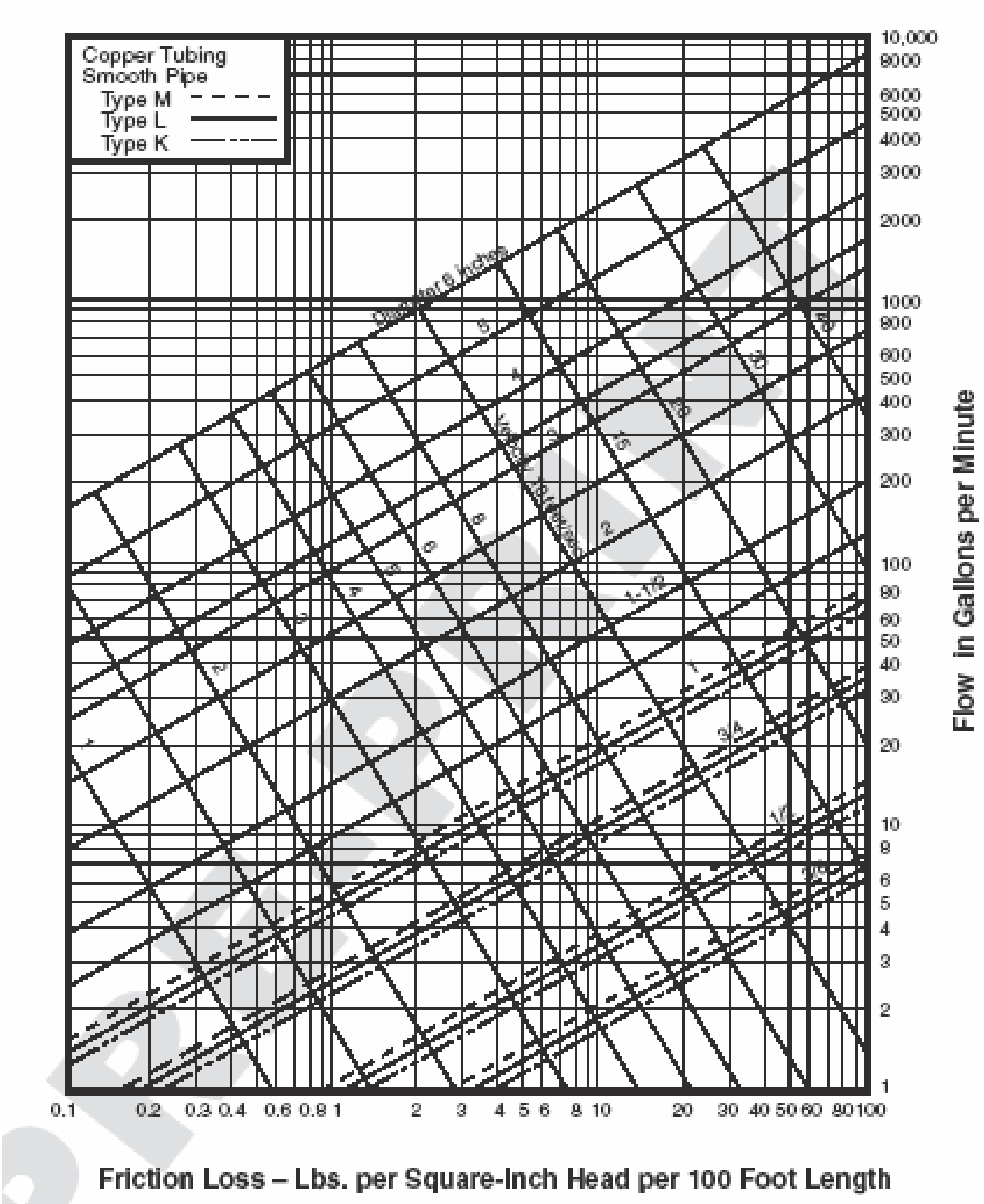

Figure 4.4: Friction Loss in Type K, L, and M Copper Smooth Pipe

4.6 Appendix 4A:

The following is a detailed description of residential water distribution design. The design should be compliant with the International Plumbing Code (IPC), which is followed throughout the majority of the Northeastern United States. The objective of designing a plumbing water distribution system is to provide the customer with potable drinking water to every fixture under normal operating conditions. This goal is accomplished by using the following steps:

1. Obtain the minimum daily static pressure in the service main.
2. Find the elevation difference between the street main and the highest fixture group within the plumbing distribution system.
    a. This will yield the maximum pressure loss due to elevation head, where 2.31 feet is approximately equal to 1 psi.
        i. Note: if the supply is located above the system, as in a downfeed distribution system, you must ADD the pressure at the lowest fixture.
3. Identify the types of fixtures being supplied by the system:
    a. Flush tanks
    b. Flush valves
    c. Blowout action fixtures
4. Define the minimum pressure that is required for the highest group of fixtures:
    a. Flush tanks
        i. 8 psi flowing

b. Flush valves

    i. 15 psi flowing

c. Blowout action fixtures

    i. 25 psi

5. Identify any fixtures that require “continuous demand”. The best examples are hose bibs, or sprinkler systems used for irrigation purposes. These fixtures will impose additional demands on the system, which differ from the random use that characterizes most other plumbing fixtures. If two hose bibs are located in a one-family dwelling, then only one bib should be included in the flow calculations, assuming that only one will be used at a time. The flow value for the fixture, in gpm, should be added directly to the calculated total demand (gpm).

6. Estimate the demand of the building for the building water service line, laterals, risers, and branches (these components comprise a typical residential branches distribution system). The water service line runs from the street water main to the water meter. Laterals are oriented in the horizontal plane and connection junction nodes. Risers are oriented in the vertical plane, and like laterals, connect junction nodes. Branches connect demand nodes (plumbing fixtures) to junction nodes. Demand estimation is completed by using the following process:

    a. Residential water distribution systems are made up of hot and cold water lines. Separate considerations for sizing should be given to each system.

        i. The IPC includes tables which provide the fixture unit values for fixtures that are serviced by both hot and cold water. The tables report “hot”, “cold”, and “total” fixture unit values for each fixture type (See Table 2.6). When designing hot and cold lines separately, the respective "hot" and "cold" fixture unit values should be used for sizing. When designing pipes that carry water for both hot and cold (i.e. the water service line), the "total" fixture unit value should be used. The individual "hot" and "cold" values are typically taken as 75% of the "total" value (Harris, 1990).

ii. A fixture unit value is defined as, "a quantity in terms of which the load producing effects on the plumbing system of different kinds of plumbing fixture are expressed on some arbitrary chosen scale" (Breese, 2001).

iii. The fixture unit values are derived from Hunter's Method.

1. A fixture unit value yields a load value (demand) in gallons per minute (gpm). This load value is "the peak instantaneous demand that will not be exceeded more than a specified fraction of the time" (Breese, 2001). The specified fraction of time is 1%, meaning that the system will function normally 99% percent of the time. The IPC differentiates load values based on public and private use, as well as flush valve or flush tank fixture type.

b. Partition the system into sections. These breaks occur at the following locations:

i. Major changes in elevation

ii. Branches to fixture groups

        iii. Equipment (pumps, water heater, water meter, etc.)

    c. Find the estimated volume of flow (demand) through each section of the system.

        i. The estimated demand is found by summing the fixture units served by each branch, and then finding the corresponding load values in gpm (using Hunter's curve). These values, along with the S_c value, are used to calculate the pipe sizes using the Hazen-Williams formula. Note: the demand estimation process is streamlined by starting at the most remote fixtures in the hot and cold lines, and working back to the water main.

7. Obtain and list the pressure losses through any special devices that are included in the system. Often, the pressure losses occurring in these devices will be given by the manufacturer (in psi). These may include, but are not limited to:
    a. Water main tap
    b. Water meter
    c. Backflow preventer
    d. Water filters/softeners
    e. Hot water heater (only on the hot water system)
8. Sum the amount of head losses due to special devices and elevation head.
9. Subtract the value from line 8 and the value from line 4 (minimum required pressure) from the minimum daily static pressure (line 1). The resulting value will be the pressure available to pipe friction loss in the system.

10. Find the critical path. This is defined by the length of the longest run of pipe in the system, which produces the path of maximum friction loss (friction losses increase with increasing pipe length). This value is defined by the hydraulically most distant point in the distribution network.

    a. The path will begin at the water main and run through the service line to the point of divergence for the separate hot and cold systems. The designation of the path will then depend on the length of the hot and cold lines, and losses associated with equipment and flow appurtenances. Typically, the hot water system will have greater losses due to heating equipment (heat exchangers, recirculation pumps, etc.).

11. Select trial pipe sizes. The IPC recommends the following steps to obtain a trial size:

    a. Determine the critical friction slop, S_c, using the following equation:

    $$S_C = (Line9)\frac{100}{Line10} \text{ psi/100 ft.}$$

    This value gives a pressure loss per 100 feet of pipe, with respect to the allowable pressure loss due to pipe friction.

    b. Next, select, for each section of the system, a trial pipe size.

        i. Refer to Hazen-Williams charts and input the flow value for each section (from Line 6) in gpm, and the pressure loss per 100 feet of pipe (Line 11a). These charts are located in Appendix E of the IPC for various types of pipe material. Chapter 4 also contains the Hazen-Williams figures for Copper pipe Types, M, L, and K in Figure 4.4.

ii. The charts will provide pipe diameters for varying flow velocities

1. Harris (1990) recommends that flow velocities do not exceed 8 feet per second. For fixtures fitted with quick-closing valves (washing machines, dishwashers, etc.), the flow velocity should not exceed 4 feet per second.

iii. If the intersection point of critical slope, S_c, and flow demand does not occur directly on a pipe size, select the next larger pipe diameter. This automatically yields pressure loss values smaller than the S_c value, hence providing a factor of safety in the design.

iv. After the trial size has been selected, find the "actual head loss value per 100 feet" of pipe in each section. This is found by finding the intersection point of the design flow value and the selected pipe diameter curve (referring to the Hazen-Williams charts).

12. Obtain the equivalent pipe lengths due to fittings and valves for each partitioned section of the system along the critical path. Equivalent pipe lengths work as surrogates for "minor loss coefficients". They account for minor losses by "adding an equivalent length of pipe for each minor loss" (Walski, 2003). The IPC furnishes two tables, one for threaded fittings, and one for soldered & recessed threaded fittings. These tables should be used to find the equivalent lengths for various fittings.

13. The values obtained from Line 12 should be added with the actual length of each pipe section. This value yields a "total equivalent length for each section".

14. Find the total friction loss in each section by multiplying the "actual head loss value per 100 feet" (Line 11-b-iv) by the "total equivalent length for each section" (Line 13). The resulting value is the friction loss (psi) in each section of the critical path.
15. The final step is to sum the friction losses in each section of the system along the hydraulic critical path. This value provides the maximum friction loss that the plumbing distribution system may incur (psi).
    a. The calculated total friction should now be compared against the pressure available to friction loss by subtracting Line 9 from Line 15. This value must be positive in order for the system to satisfactorily meet customer's demands.
        i. If the value is slightly negative, the system parameters can be changed to meet the demands. These parameters are:
            1. Trial pipe size: the pipe diameter can be increased to reduce the amount of major friction loss.
            2. Fittings: an alternative design can be drafted to reduce the amount of fittings and valves.
        ii. If the value is largely negative, the system may require a pressure boosting system.
        iii. If the value is largely positive, the pipe sizes can be reduced to minimize capital costs.

CHAPTER 5: Modeling Plumbing Systems by EPANET

5.1 Introduction:

The objective of water distribution network analysis is to obtain pipe flows and nodal heads for the entire system. In this chapter, the focus is on formulating the plumbing network as a "pressure driven" demand problem. With the aid of this formulation, energy heads at nodes and flows in pipes are determined. The pressure heads resulting from the energy heads can be checked against the minimum pressures required for proper functioning of fixtures.

EPANET is employed to aid in the solution of two pressure-driven network simulations within this chapter. EPANET is a computer program developed by the United States Environmental Protection Agency (EPA), which is publicly available for download at their website http://www.epa.gov/ORD/NRMRL/wswrd/epanet.html. The program can be used to perform steady-state and extended period simulations "of hydraulic and water quality behavior within pressurized pipe networks" (Rossman, 2004). EPANET also includes a graphical interface for editing network input data, running hydraulic and water quality simulations, and viewing program output. In this thesis, only hydraulic simulations will be performed. Most importantly, EPANET has the capability to perform pressure-driven simulations by applying emitter coefficients to model pressure-dependent nodes (i.e. plumbing fixtures). The network simulations contained within this chapter are relatively simple, but EPANET has the ability to model more complex systems (i.e. pumped-upfeed and downfeed systems). Behavior of constant and variable speed pumps, pressure regulating valves, and storage tanks can all be simulated within the EPANET program.

Plumbing distribution system demands are defined by the loads imposed by plumbing fixtures (i.e. toilets, showers, sinks, etc.), which are designed to operate at certain pressures. Because all fixtures operate under pressure, they are called “pressure driven”. In opposition to the minor systems, the major municipal systems are designed as “demand driven” systems with flow demands imposed at nodes for analysis purposes. However, operationally the basic requirement is to maintain a certain minimum pressure, p_{min}, to deliver the flow demand as shown in Figure 5.1.

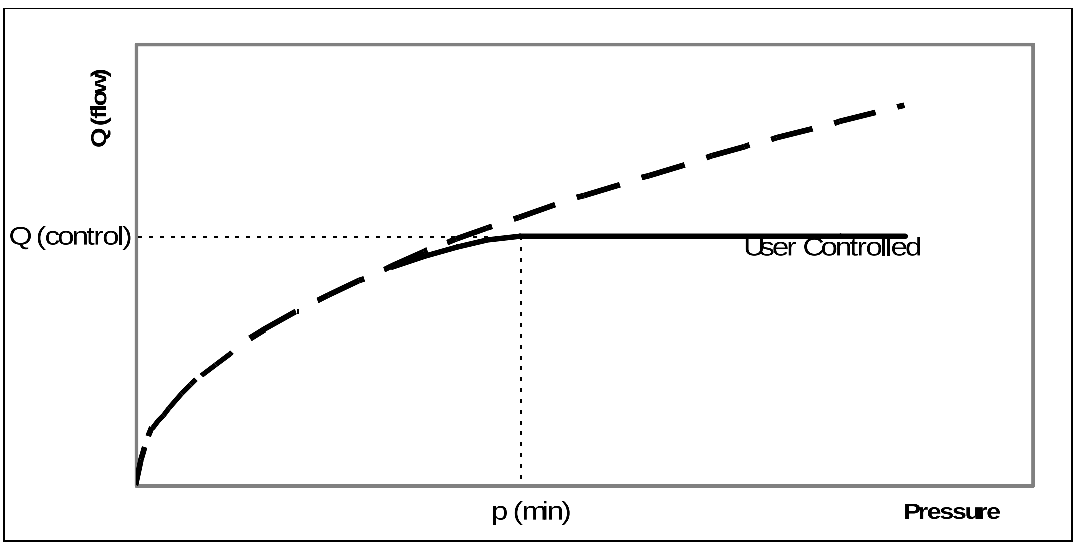

Figure 5.1: Pressure – Flow Relationship

Figure 5.1 is based on a relationship of the form:

$$Q = \begin{cases} Kp^{\gamma} & \text{for} \quad p \le p_{min} \\ Q_{control} & \text{for} \quad p > p_{min} \end{cases} \qquad (1)$$

Where, K = emitter coefficient (cfs/psi$^{-\gamma}$), Q = flow (gpm, cfs), p = pressure (lb/ft^2, psi), γ = exponent ~ 0.5, $Q_{control}$ = user controlled flow. In Equation 1, whenever p exceeds p_{min}, the pressure is capable of delivering more flow than actually needed. The user must throttle the valve to reduce the flow to $Q_{control}$.

5.2 Boundary Conditions

The primary boundary condition in a plumbing water distribution system is the pressure available at the street main. This provides a known head value, which defines a starting point for system calculations. Plumbing systems are designed to operate under worst-case conditions. For this reason, the minimum available street main pressure is used for design purposes. The street main pressure is the main source of energy for water flow through a plumbing distribution network. Depending on the type of system being supplied, this pressure may or may not be adequate to fulfill demand loadings. High static systems (high-rise buildings), high friction-loss systems, or networks with low street pressure, may require additional energy delivered by booster pumps.

Secondary boundary conditions are applied to demand nodes. The International Plumbing Code (IPC) specifies minimum pressures required to operate specified plumbing fixtures. The minimum pressures must be met for acceptable fixture operation. (see Table 3.1). These boundary conditions are critical when selecting the pipe sizes.

The difference between the street main pressure and the required minimum fixture pressure defines the amount of acceptable head loss through the piping network. A smaller pipe will yield greater friction loss than a larger diameter pipe under the same operating conditions.

5.3 Friction Loss Equations

The two widely-used equations for pipe flow are the Darcy-Weisbach and the Hazen-Williams equations. The Darcy-Weisbach equation is shown below:

$$h_f = f\frac{L}{D}\frac{V^2}{2g} = \frac{8fl}{\pi^2 gD^5}Q^2 \qquad (2)$$

Here, h_f refers to headloss [L], f is the friction factor, L is the length of pipe [L], D is the pipe diameter [L], V^2 is the mean longitudinal pipe velocity [L/T], g is gravity [L/T^2] (Bhave, 1991). The Hazen-Williams equation is an "empirical formula widely used in water supply engineering" (Bhave, 1991). The International Plumbing Code, along with other plumbing codes, reference this formula as the preferred head loss equation for plumbing water distribution systems. This preference is attributed the Hazen-Williams constant loss coefficient, C, which estimates the frictional characteristics of the pipe wall. The Darcy-Weisbach equation uses a more complex method to obtain frictional characteristics (Reynolds number and the friction factor), which make it more difficult to utilize. The Hazen-Williams equation is shown below:

$$h_L = \frac{\phi L}{C^{1.85}D^{4.87}}Q^{1.85} = \left(\frac{\alpha L}{C^{1.852}D^{1.167}}\right)V^{1.852} \qquad (3)$$

where, h_L is head loss [L], ϕ is a conversion factor depending on SI (10.66) or English (4.73) units, L is the pipe length [L], C is the Hazen-Williams coefficient, D is the pipe diameter [L], Q is the flow rate [L^3/T], $\alpha = 6.81$ when V is in meters/sec and D is in meters, and $\alpha = 3.022$ for V in ft/sec and D in feet (Mays, 2001). Bhave (1991) states that the Hazen-Williams coefficient is "dependent on the hydraulic radius, slope of the energy line, and also the flow conditions". Although this is the case, in practice the coefficient is typically assigned as a constant to specific pipe materials regardless of flow conditions.

5.4 Friction Factor and the Hazen-Williams Coefficient

The Reynolds number:

$$\mathrm{Re} = \frac{VD}{\nu} \qquad (4)$$

where, V is the longitudinal pipe velocity [L/T], D is the pipe diameter [L], and ν is the kinematic viscosity of the fluid (water) [L^2/T]. Pipe velocities are designed to range from 4 to 8 feet per second, which yields a mean velocity of 6 fps. Plumbing water distribution systems are commonly comprised of 1", ¾" and ½" pipe links (for example purposes a value of 1" is used). Water, at 50 degrees Fahrenheit, has a kinematic viscosity of 1.41×10^{-5} ft^2/s. These parameters yield a Reynolds number of approximately 3.6×10^4. Relative roughness is defined by the following equation:

$$\mathrm{Re}lative - Roughness = \frac{e}{D} \qquad (5)$$

Using e = .0015 mm or $5(10^{-6})$ feet or $6(10^{-5})$ inches for the equivalent sand roughness for Type L cooper and a pipe diameter of 1", the relative roughness is 0.00006. For such a small e/D value of 0.0006, we can treat the copper pipe as smooth and its friction factor is given by the Blasius equation

$$f = \frac{0.316}{R^{0.25}} \tag{6}$$

For a Reynolds number of $3.6(10^4)$, we obtain a friction factor of 0.023. We can relate the Hazen-Williams coefficient to the friction factor, *f*, by setting the head losses in equations (2) and (3) to be equal. That is

$$C = \left[\frac{\alpha(2g)}{(V)^{0.148}(D)^{0.167} f} \right]^{0.54} \tag{7}$$

For $\alpha = 3.022$, V = 6 ft/sec, D = 1/12 ft, $f = 0.023$, and g = 32.2 ft/sec^2, we obtain the Hazen-Williams coefficient as 143.

5.5 Emitter Coefficient

Equation 1 shows the relationship between flow and pressure. This relationship is linked with the use of an emitter coefficient. A typical application for Equation 1 is fire sprinkler system design, where the sprinklers are represented as emitters (Walski, 2003). This approach is not limited to sprinklers, and can be applied to plumbing fixtures, as well. This is a typical practice in the faucet valve industry, although the emitter coefficient is sometimes referred to as a "C.V. value". Modifying Equation 1:

$$CV = (Q)\left(\frac{1}{\sqrt{P}} \right) \tag{8}$$

Faucet design firms commonly use emitter coefficient values to hydraulically classify valves, which are used for design (Delta, 2005).

Accurate modeling will require proper emitter coefficient values. Walski (2003) lists sprinkler orifice sizes with their corresponding emitter coefficients. The following is a derivation attempting to reproduce Walski's values. Consider the orifice flow equation given by:

$$Q = C_d A\sqrt{2gh} \qquad (9a)$$

where, C_d is the discharge coefficient, A is the orifice area [L^2], g is the acceleration due to gravity [L/T^2], and h is the head loss over the orifice [L]. Using the emitter flow equation,

$$Q = K\sqrt{P} \qquad (9b)$$

in the orifice equation (Equation 9a), we have,

$$K = \frac{C_d A\sqrt{2gh}}{\sqrt{P}} \qquad (9c)$$

and writing $P = \gamma h$ we obtain,

$$K = C_d A\sqrt{\frac{2g}{\gamma}} \qquad (9d)$$

Equation (9d) applies to a consistent system of units. Using Cd = 0.667, A = $(\pi/4)(0.75/12)^2$ corresponding to ¾" opening, and p in psi, from equation (9d) we obtain

$$K = \frac{0.667\left[\pi\left(\frac{0.75}{12}\right)^2\right](12)\sqrt{\frac{64.4}{62.4}}}{4}$$

and K = 0.025. For a 1" opening K = 0.044. Also, from Equation 1, the units for K are,

$$K = \frac{(gallons)(inches)}{(\min ute)\sqrt{pounds}} \qquad (9e)$$

Using a conversion factor we have,

$$K = \left(\frac{1}{12}\right)\left(\frac{448.4}{1}\right)\left(\frac{(feet)^2 (inches)^2}{(second)\sqrt{pounds}}\right) = \left(\frac{gallons}{minute}\right)\left(\frac{inches}{\sqrt{pounds}}\right) \qquad (9f)$$

and,

$$K = \left(\frac{1}{12}\right)\left(\frac{448.4}{1}\right) C_d A \sqrt{\frac{2g}{\gamma}} \qquad (9g)$$

where, C_d is the discharge coefficient, A is the orifice area (in^2), g is gravity (ft/s^2), and γ is the specific weight of water (lb/ft^3). Equation (9g) represents the working expression for deriving emitter coefficient values based on English units. The area, A, parameter is defined by the diameter of the orifice for which the emitter coefficient is being derived. The coefficient of discharge, C_d, is a parameter that is based on the geometry of the orifice, pipe diameter, and the velocity. Below is a representation of a typical orifice in a pipe:

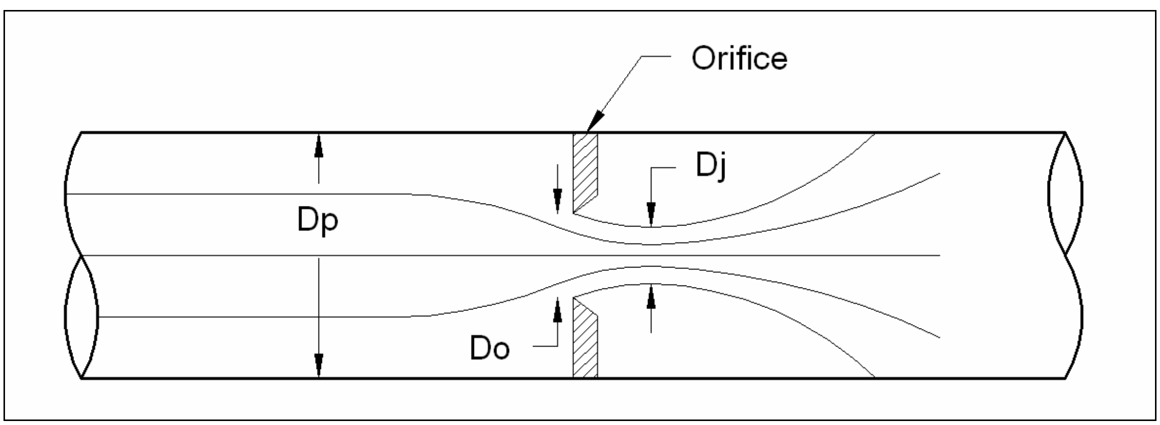

Figure 5.2: Typical Orifice Schematic

The coefficient of discharge is dependent on the ratio of the diameter of the orifice, D_o, to the diameter of the pipe upstream, D_p. C_d is represented by the following equation:

$$C_d = \frac{C_C C_V}{\sqrt{1 - C_C^2 \frac{A_O^2}{A_P^2}}} \qquad (10)$$

where, C_c represents the contraction coefficient of the orifice, C_v represents the coefficient of velocity, A_o is the area of the orifice [L^2], and A_p is the cross-sectional area of the pipe [L^2]. Both area terms, A_o and A_p are obtained from the geometry of the pipe and orifice, respectively. The velocity coefficient, C_v, is only applicable for flows where viscous effects are significant, which occurs with low to moderate Reynolds numbers (Roberson and Crowe, 1993). Fully turbulent flow, meaning higher pipe velocities and larger Reynolds numbers, nullifies the need for the velocity coefficient. The contraction coefficient, C_c, like the velocity coefficient, is a function of the Reynolds number at low velocities (Roberson and Crowe, 1993). C_c is solely a function of geometry for fully turbulent flow. The contraction coefficient is defined by the ratio of the orifice diameter to the upstream pipe diameter.

5.6 Formulation of Equations:

The objective of water distribution network analysis is to obtain the pipe flows and nodal heads for the entire system. The system constraints, unknowns, and friction formulas have been identified, along with the derivation of the emitter coefficients. All these parameters and equations must be interrelated to run a network analysis.

Plumbing distribution networks, especially for cold water lines, are usually branched-type systems. Branched networks are typically comprised of "one source, one or more intermediate nodes, and one or more sinks (plumbing fixtures)" (Bhave, 1991). There are no loops, or redundant nodes. This configuration provides only a single delivery path to each user point (plumbing fixture), and the direction of flow in all pipes is fixed. An analysis can be completed by starting at a node of known head (source node) and successively applying pipe head loss equations in concert with node continuity expressions (Bhave, 1991). The following paragraphs describe the equations to carry out a branched network analysis.

The pipe head loss relationship describes the energy dissipated due to friction between water and the inner pipe wall. Friction loss is the difference in energy heads given for each pipe x = 1, 2, …, n as:

$$h_x = H_i - H_j = R_x Q_x^n \qquad (11)$$

where, h_x is the head loss in a pipe [L], H_i is the head in the upstream node [L], H_j is the head value in the downstream node [L], R_x is the resistance coefficient of pipe x [T^2/L^5] (fundamental units for Darcy-Weisbach equation only), Q_x is the discharge in pipe x [L^3/T], and n is the flow exponent. Equation 11 can be rewritten to account for the direction of flow within the pipe. This expression is:

$$h_x = H_i - H_j = R_x \left|Q_x\right|^{n-1} Q_x \qquad (12)$$

The value of the resistance coefficient and the flow exponent are both based on the selected friction equation, and the Darcy-Weisbach formula we have:

$$n = 2 \qquad (13)$$

$$R = \frac{8fL}{\pi^2 g D^5} \qquad (14)$$

Equation (13) and (14) substituted into Equation (12) produces a head loss relationship that is based on Darcy-Weisbach friction loss formula.

The continuity equation is written as:

$$\sum_{x \to j} Q_x + q_j = 0 \qquad (15)$$

where, q_j is the supply/demand at node "j", and the pipe discharge values, Q_x, are summed for all links connected to node "j". An arbitrary sign notation must be adopted to define inflows and outflows as positive or negative values. In this thesis, inflows to the node are considered positive discharge values and outflows are negative. Pressure dependency is accomplished by substituting the emitter coefficient equation (Equation 10b) into the node-flow continuity relationship as:

$$\sum_{x \to j} Q_x + K_j \sqrt{P_j} = 0 \qquad (16)$$

where, K_j is the emitter coefficient at the demand node (plumbing fixture) and P_j is the pressure at the demand node [F/L^2]. Equation (16) represents the summation of incoming and outgoing flows, as well as demand flows defined by the pressure-dependent emitter equation. Using:

$$P_j = H_j \gamma \qquad (17)$$

where, γ is the specific weight of water [F/L^3]. This yields the working equation for the node-flow continuity relationship:

$$\sum_{x \to j} Q_x + K_j \sqrt{H_J \gamma} = 0 \qquad (18)$$

For J demand nodes and n pipes, we can write n equations of the form Equation (11) and J equations of the form Equation 18, and therefore we can solve for H_j, j = 1, 2, …., J and Q_x, x = 1, 2, …, n.

5.7 Network Example 1:

A typical branched network distribution system is shown below:

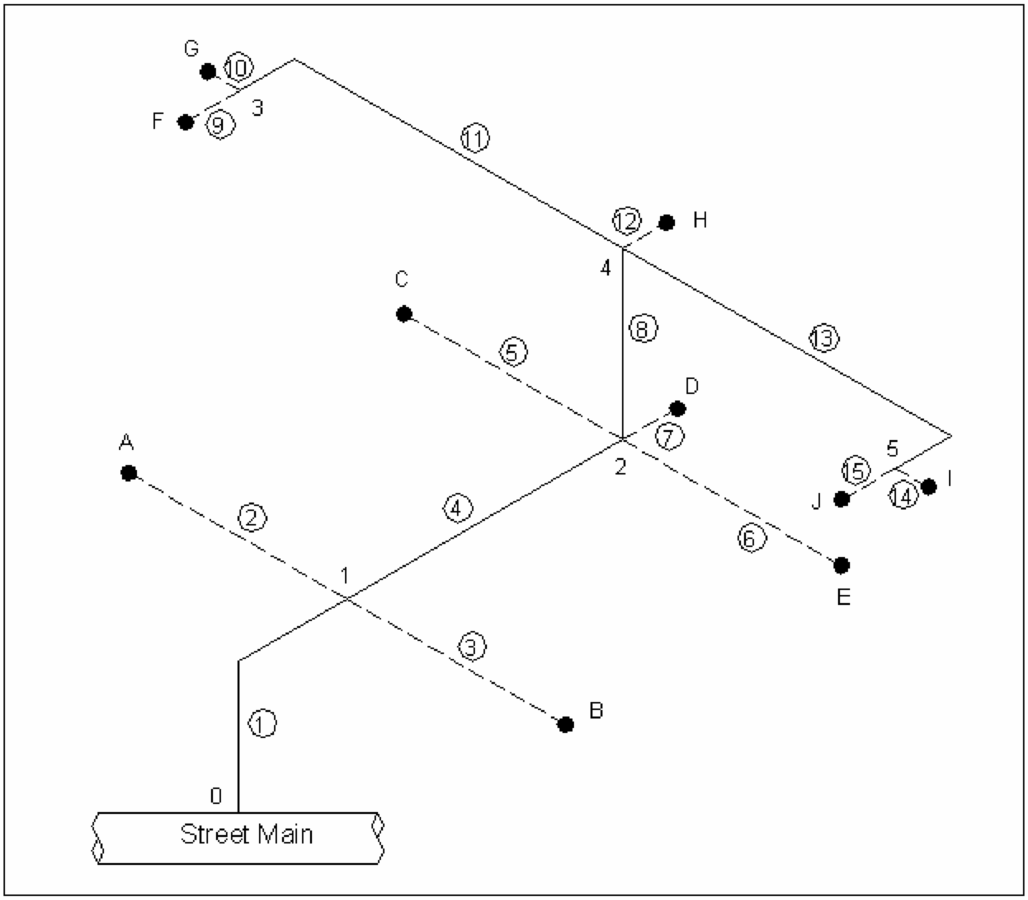

Figure 5.3: Typical Branched Network Distribution System (isometric view)

This example is used to prove the validity of the above equations and solution technique. Here, all demand nodes are displayed as large black dots. The street main represents the supply node, while all pipe intersections are junction nodes. Including the street main, there are 16 nodes, with J = 15 demand nodes omitting the street level source node whose energy value is known. The nodes are identified by numbers and letters that are not circled. There are 15 pipes connecting the nodes, and n = 15. The pipes are labeled by circled numbers. The total number of required independent equations is “n+J”, which equals 30. The given parameters for this network are:

1. Boundary Condition: Street level pressure is at 50 psi
2. Unknown Values:
    a. Head values at all demand nodes and junction nodes
    b. Flow values in all pipe links
3. Pipe Characteristics:
    a. Material: Type L Copper
    b. Diameters and Lengths:

Table 5.1: Pipe Data for Figure 5.3

Pipe ID	Length (ft)	Diameter (inches)
1	75	3
2	20	1
3	20	1
4	25	3
5	20	1
6	20	1
7	5	1
8	15	3
9	5	1
10	3	1
11	35	2
12	4	1
13	35	2
14	3	1
15	5	1

4. Emitter Coefficients (applied to each demand node). Following the discussion under equation (9d) we choose K = 0.02899 for better results.

The solution of the example network formulation has been performed using two programs. EPANET is a program developed and distributed by the U.S. Environmental Protection Agency. It is commonly used in the water resources industry, and it allows pressure-driven analysis using the emitter flow equation. Microsoft Excel was also utilized to run the network analysis. Excel has a built-in optimization tool referred to as "Solver". Numerical comparisons between EPANET and Excel provide a powerful validation tool for both solution techniques. The data is given in Tables 5.1, and Tables 5.2 and 5.3 contain the results of the analyses.

Table 5.2: Comparison of Energy Head Values:

Node ID	Head Values - Solver	Head Values - EPANET	Z	p/γ – Solver	Pressure - Solver
	(ft)	(ft)	(ft)	(ft)	(psi)
0	439.00	439	324	115	50
1	404.17	403.7	344	60.17	26
2	396.81	396.16	344	52.81	23
3	390.24	389.17	359	31.24	14
4	395.18	394.43	359	36.18	16
5	390.24	389.17	359	31.24	14
A	373.54	372.41	344	29.54	13
B	373.54	372.41	344	29.54	13
C	369.92	368.77	344	25.92	11
D	385.93	384.98	344	41.93	18
E	369.92	368.77	344	25.92	11
F	383.80	382.63	359	24.80	11
G	386.03	384.88	359	27.03	12
H	388.96	388.04	359	29.96	13
I	386.03	384.88	359	27.03	12
J	383.80	382.63	359	24.80	11

Table 5.3: Comparison of Flow Values:

Pipe Link ID	Flow Values - Solver (cfs)	Flow Values - EPANET (cfs)	Error (%)
1	1.018	0.99	2.78%
2	0.104	0.1	3.58%
3	0.104	0.1	3.58%
4	0.811	0.79	2.58%
5	0.097	0.09	7.37%
6	0.097	0.09	7.37%
7	0.124	0.12	2.90%
8	0.493	0.48	2.63%
9	0.095	0.09	5.31%
10	0.099	0.1	-0.79%
11	0.194	0.19	2.19%
12	0.104	0.1	4.27%
13	0.194	0.19	2.19%
14	0.099	0.1	-0.79%
15	0.095	0.09	5.31%

In this chapter we have presented a pressure-driven analysis for analyzing plumbing systems. The analysis is performed for given choice of diameters. Hence, by iteratively changing the diameters and assessing the results an optimal set of diameters can be chosen. This procedure will not require the design charts used in Chapter 4 and can be automated.

5.8 Network Example 2:

The above solution to network 1 verifies that either EPANET or Excel SOLVER can be used to solve pressure-driven demand problems. SOLVER, which requires that all equations and relationships be manually input, is not efficient for larger systems. The formulations represented in equations (1) through (18) are pre-built into the EPANET solution algorithm, which makes it the best choice for analyzing more complex networks. System interconnection and network parameters are the only requirements that are needed to run EPANET. EPANET also includes a graphical interface making it easier to visualize the network. Harris (1990) provides an example of a typical plumbing water distribution system of a one-family residential home. Figure 5.4 represents a schematic based on Harris' example (a larger schematic is shown in Appendix 5A Figure 5A.1).

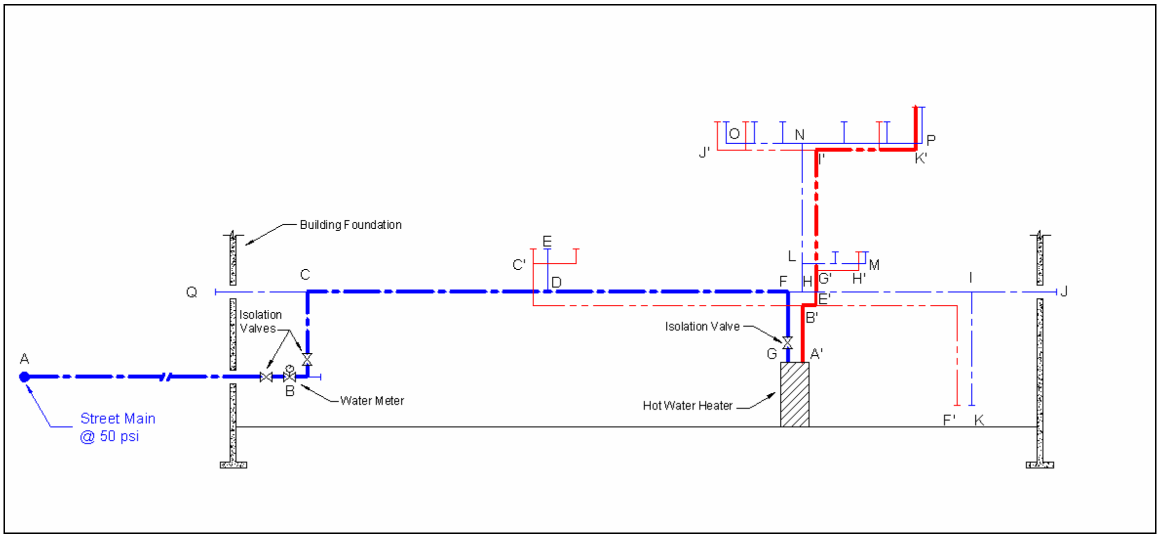

Figure 5.4: Typical One-Family Residential Plumbing System

The street main provides a minimum daily service pressure of 50 psi. The system includes fixture groups located on three floors (basement, 1st floor, and 2nd floor) and a critical path of 180'. Cold water pipes are indicated by single-dashed blue lines and hot water pipes by double-dashed red lines. The bold lines represent the critical path of the system (hydraulically longest pipe run), which includes both cold and hot water piping. Type K copper is used for the water service line and Type L copper is used for all other pipes. All hot water demand nodes are represented with an apostrophe ('). The water meter has a rated pressure loss of 6 psi at 25.9 gpm, and the hot water heater creates a 1.5 psi loss at peak demand.

The water service line (carrying cold water directly from the street water main) enters through the structure's foundation (basement level) and continues to the water meter at node B. The cold water line continues under the 1st floor branching to a hose bib at node Q and a kitchen sink at node D. Node F indicates the point at which the cold water line splits to the hot water heater at node G. Every node downstream of branch FH serves cold water demands only. All nodes downstream of branch FG serve hot water demands only. Nodes O and J' represent respective cold and hot water demands for a master bathroom. The master bathroom includes a shower/bathtub combination fixture, a water closet, and a lavatory. Nodes P and K' represent the respective cold and hot water demands for a guest bathroom, which includes a shower stall, water closet, and lavatory. Futhermore, node K' defines the most remote point in the system with an elevation of 19' with respect to the water main at 0'. Both full bathrooms are located on the second floor. Nodes M and H' represent a water closet and lavatory for the powder room (½ bathroom) located on the first floor. Also on the first floor, nodes E and C' define the cold and hot water demand points for the kitchen sink and dishwasher. Node J is a cold water hose bib, and nodes K and F' indicate an automatic washing machine located in the basement. Table 5.4 lists the elevations of each node, with respect to the street water main, as well as the fixtures that are directly connected to each node. Table 5.5 displays the quantities of fittings occurring in each branch along the critical path. Fittings are not listed on all branches because equivalent pipe length calculations (analogous to minor loss calculations) are required for only those pipes along the critical path.

Table 5.4: Network Example 2 – Node Elevations and Connected Fixtures

Node ID	Elevation	Fixtures Served (Directly Connected)		Equipment
	(ft)	Hot	Cold	
A (Street Main)	0			
A'	1			
B	0			Water Meter
B'	5			
C	6			
C'	9	Kitchen Sink, Dishwasher		
D	6			
E	9		Kitchen Sink	
E'	5			
F	6			
F'	-2	Washing Machine		
G	1			Hot Water Heater
G'	7			
H	6			
H'	9	Lavatory		
I	6			
I'	16			
J	6		Hose Bib	
J'	18	Shower/Tub, Lavatory		
K	-2		Washing Machine	
K'	19	Shower, Lavatory		
L	8			
M	9		Water Closet, Lavatory	
N	16			
O	18		Shower/Tub, Lavatory, Water Closet	
P	19		Shower, Lavatory, Water Closet	
Q	6		Hose Bib	

Table 5.5: Network Example 2 – Branch Fittings along Critical Path

	Section ID		Section Length	Fittings			
	Upstream	Downstream	(ft)	90	Tee, run	Tee, branch	Ball Valve
Hot Water Lines	I'	K'	10	2		1	
	G'	I'	10		1		
	E'	G'	10		1		
	B'	E'	1			1	
	A'	B'	4			1	
Cold Water Lines	F	G	5				1
	D	F	17			1	
	C	D	17		1		
	B	C	6			2	1
	A	B	100				1

In Table 5.5, "Tee, run" refers to a tee in which the flow does not change directions along the critical path, where "Tee, branch" refers to a tee where the flow path must change by 90°.

Node F, which indicates the point at which the cold water line splits to the hot water heater at node G, represents a critical point for estimating demand loads within the system. All pipe branches downstream of node F must be sized to carry the demand load defined by fixture unit values for individual cold or hot water demands for each fixture. Pipe links downstream of branch FG convey hot water only, and all pipes downstream of branch FH carry cold water only. All pipe branches upstream of node F, which carry the water serving both cold and hot demands, are sized using total fixture unit values for each fixture. Furthermore, the summation of individual cold and hot water fixture units does not sum to the total fixture unit value (for fixtures requiring both cold and hot water). For example, a shower/tub combination fixture is defined as having 1 fixture unit for cold water supply, 1 fixture unit for hot water supply, and 1.4 fixture units for total water supply. This corresponds to a 3 gpm demand for the individual cold and hot water lines, and a 4 gpm demand for the total fixture demand. Harris (1990) states, "The individual "hot" and "cold" fixture unit values are typically taken as 75% of the "total" value". The reasoning behind this is that a fixture (requiring both cold and hot water) may run only cold, only hot, or a combination of cold and hot water at any time. When running cold and hot water simultaneously, the water must be mixed together while flowing through the fixture valve. The mixing action limits the amount of water that can flow through the fixture, and thus limits the individual cold and hot water demands to roughly 75% of the total demand. This explains why branches upstream of node F are designed to carry only the demand associated with the accumulated "total" fixture units, and why branches downstream of node F are designed to carry only the demand associated with the accumulated "hot" or "cold" fixture units. In the case of Network Example 2, Table 5.6 displays the fixture unit values and demand value for each fixture in the building. Tables 5.7 and 5.8 show the branch-wise demands for hot and cold water lines, respectively. Notice that the demand values in branches FH and FG sum to 32.3 gpm, but the next upstream branch, DF, is only designed to carry 22.7 gpm. Branch FH and FG have been designed to carry the demand associated with the individually accumulated downstream cold and hot water fixture units, respectively. Branch DF is designed to convey the demand associated with the "total" accumulated fixture units for all downstream fixtures. This is a direct result of the above discussion, and is critical for proper pipe sizing when utilizing Hunter's curve.

The piping is sized using the IPC's recommended design procedure (Hunter's method in conjunction with critical slope, S_c), which is outlined in Appendix 4A. Table 5.6 shows fixture unit values and demands attributed to each fixture in the system. The fixture unit values are associated with a "flush-tank" dominated system, meaning that the lower part of Hunter's curve is used to derive the fixture demands (see Figure 2.4 showing Hunter's curve). Because the guest room defines the most remote point in the distribution network, it will also define the minimum required pressure used to calculate the critical slope, S_c. All fixtures in the guest bathroom, the lavatory, shower stall, and water closet, require identical minimum pressures, namely 8 psi.

Table 5.6: Network Example 2 - Fixture Units and Demands

Floor/Level	Group Description	Node ID		Fixtures	Fixtures Units			Flow Values (gpm)		
		Hot	Cold		Hot	Cold	Total	Hot	Cold	Total
Basement	Laundry Station	F'	K	Automatic Washer	1	1	1.4	3	3	4
	Hose Bibs (irrigation)		Q	Hose Bib (front)	N/A	N/A	N/A	0	5	5
			J	Hose Bib (rear)	N/A	N/A	N/A	0	0	0
1st Floor	Kitchen	C'	E	Sink	1	1	1.4	3	3	4
				Dishwasher	1.4	0	1.4	3.8	0	3.8
	Powder Room	H'	M	Lavatory	0.5	0.5	0.7	1.5	1.5	2
				Water Closet	0	2.2	2.2	0	5.3	5.3
2nd Floor	Master Bath	J'	O	Lavatory	0.5	0.5	0.7	1.5	1.5	2
				Bathtub/shower	1	1	1.4	3	3	4
				Water Closet	0	2.2	2.2	0	5.3	5.3
	Guest Bath	K'	P	Lavatory	0.5	0.5	0.7	1.5	1.5	2
				Shower	1	1	1.4	3	3	4
				Water Closet	0	2.2	2.2	0	5.3	5.3

Fixture units are not given to hose bibs because they represent continuous demands, which do not fit into Hunter's probabilistic definition of fixture usage. Continuous demands are simply added to the total demand. Also notice that only one hose bib has a defined demand value (5 gpm). It is assumed that only one hose bib will be "on" at any time. The bib furthest from the main, node J, was chosen for design over the hose bib at node Q. Node J is furthest from the supply, and therefore imposes greater friction losses through the cold water piping (friction loss increases with increasing pipe length). This selection provides a built-in factor of safety.

Demand estimates are obtained by partitioning the system into sections. Table 5.7 and Table 5.8 identify each pipe section with upstream and downstream nodes. Fixture groups are shown under the "Group Description" column in Table 5.6. Beginning at the guest bathroom, the fixture values are accumulated for each branch back to the street water main. Table 5.7 and Table 5.8 display the accumulated fixture unit and demand values for hot and cold water lines, respectively.

Table 5.7: Hot Water Accumulated Fixture Units and Demands

Section ID		Fixture Units	Demand
Upstream	Downstream		(gpm)
I'	K'	1.5	4.2
I'	J'	1.5	4.2
G'	I'	3	6.5
G'	H'	0.5	2
E'	G'	3.5	7
E'	F'	1	3
B'	C'	2.4	5.9
B'	E'	4.5	9
A'	B'	6.9	11.8

Table 5.8: Cold Water Accumulated Fixture Units and Demands

Section ID		Fixture Units	Demand
Upstream	Downstream		(gpm)
N	P	3.7	7.5
N	O	3.7	7.5
L	N	7.4	12.4
L	M	2.7	6
H	L	10.1	14.8
I	K	1	3
I	J	N/A	5
H	I	1	8
F	H	11.1	20.5
F	G	6.9	11.8
D	F	15.3	22.7
D	E	1	3
C	D	16.3	23.1
B	C	16.3	23.1
A	B	16.3	23.1

The above demand values (gpm) will be used to determine the pipe diameters. Note that these demand values differ slightly from those Harris (1990) used in his solution method (it seems that Harris may have rounded fixture unit values to the next greatest whole number).

Pipe sizes are selected using three parameters, namely, flow rate, velocity, and critical friction slope, S_c. First, the demands shown in Tables 5.7 and 5.8 provide the required volumetric flow rate values. Second, velocity must be kept to a maximum of 8 ft/s. Branches serving fixtures with "quick-closing" valves should not exceed 4 ft/s (Harris, 1990). Here, the dishwasher and automatic washing machine are limited to the 4 ft/s velocity constraint and all other branches are designed for 8 ft/s or less. Third, the critical friction slope, S_c, must be considered to guarantee hydraulic functionality of the system. The critical slope is determined by first obtaining the pressure available to friction loss. The pressure available to friction loss is calculated by subtracting the sum of the elevation head loss, the losses due to equipment, and the minimum required pressure at the most remote fixture group from the available street main pressure. It is

$$= 50\,psi - [(19\,ft/2.31) + (6 + 1.5 + 0.5)]\,psi - 8\,psi = 25.8\,psi\,.$$

The critical slope, S_c, is determined as

$$S_C = 25.8\left(\frac{100}{180}\right) = 14.3\,psi/100\,ft$$

where, 180' is the critical path. Pipe sizes are determined by referring to the Hazen-Williams curves for Type K and Type L copper. Figure 4.4 (Hazen-Williams curves for Type M, L, and K copper pipe) has been reproduced as Figure 5.5 below. The red lines have been drawn in to provide an example of how pipe AB was sized. The vertical line originates from S_c (14.3 psi/100 ft). The horizontal line corresponds to branch AB's demand value of 23.1 gpm. Notice that velocity exceeds 8 ft/s at the intersection point of the two red lines. This requires the selection of the next whole pipe size, which is 1¼". This process is repeated for each branch in the system.

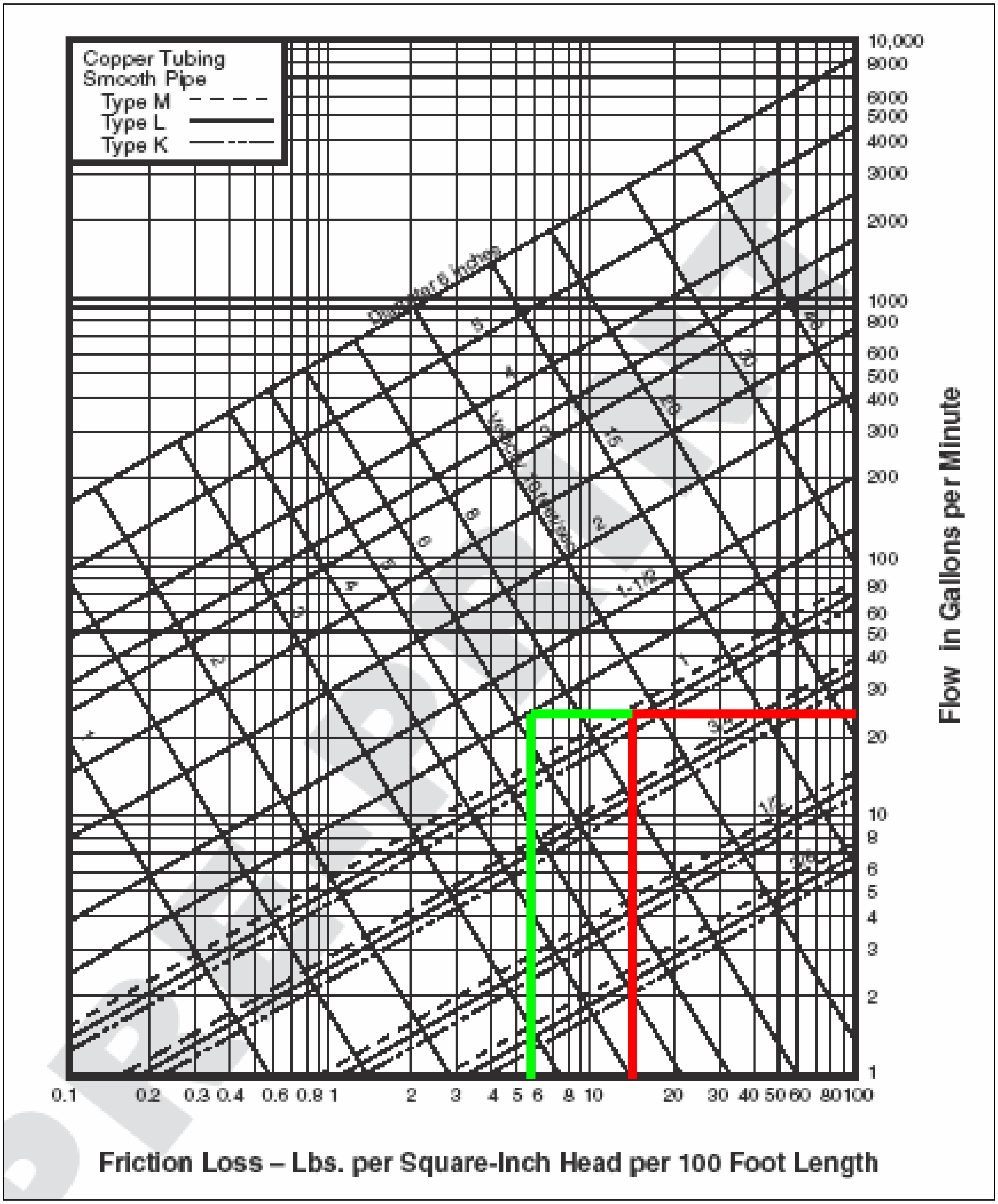

Figure 5.5: Friction Loss in Type K, L, and M Copper Smooth Pipe

Table 5.9 and Table 5.10 show the resulting pipe diameters and actual head loss values for hot and cold lines, respectively.

Table 5.9: Hot Water Pipe Diameters

Section ID		Design Velocity	Trial Pipe Size	Actual Head Loss
Upstream	Downstream	(ft/s)	(in)	(psi/100 feet)
I'	K'	8	0.5	13
I'	J'	8	0.5	13
G'	I'	8	0.75	5
G'	H'	8	0.375	10
E'	G'	8	0.75	6
E'	F'	4	0.5	8
B'	C'	4	0.75	4.5
B'	E'	8	0.75	9
A'	B'	8	1	4.5

Table 5.10: Cold Water Pipe Diameters

Section ID		Design Velocity	Trial Pipe Size	Actual Head Loss
Upstream	Downstream	(ft/s)	(in)	(psi/100 feet)
N	P	8	0.75	7
N	O	8	0.75	7
L	N	8	1	4
L	M	8	0.75	4.5
H	L	8	1	7
I	K	4	0.5	8
I	J	8	0.75	3
H	I	8	0.75	7
F	H	8	1	10
F	G	8	1	5
D	F	8	1.25	4.5
D	E	8	0.5	8
C	D	8	1.25	5
B	C	8	1.25	5
A	B	8	1.25	5

If the intersection point of the critical slope, S_c, and the flow demand does not occur directly on a pipe size, select the next largest pipe size. By selecting a larger pipe diameter, the head loss value must decrease due to Hazen-Williams inverse relationship with D (see equation (3) above). The "Actual Head Loss" column is obtained from the

Hazen-Williams curves by finding the associated head loss with the selected pipe diameter. For example, Figure 5.5 shows a horizontal green line extending from the point of intersection of the two red lines to 1.25" diameter curve. The vertical green line drops to the x-axis defining the actual friction loss of 5 psi per 100 ft of 1.25" copper pipe. Notice that none of the values in Tables 5.9 and 5.10 exceed the critical slope value of 14.3 psi/100 ft.

Minor losses are determined by using "equivalent pipe lengths", which serve as surrogate values for minor friction losses. The IPC provides tables for equivalent pipe length values for common fittings and valves. Table 5.11 shows the equivalent pipe length values along the critical path. These values were derived for the fittings listed in Table 5.5.

Table 5.11: Minor Losses in Equivalent Pipe Lengths along the Critical Path

	Section ID		Developed Section Length	Equivalent Pipe Length per Section	Total Equivalent Section Length
	Upstream	Downstream	(ft)	Per Pipe Section (ft)	(ft)
Hot Water Lines	I'	K'	10	4	14
	G'	I'	10	0	10
	E'	G'	10	0	10
	B'	E'	1	3	4
	A'	B'	4	4.5	8.5
Cold Water Lines	F	G	5	0.5	5.5
	D	F	17	5.5	22.5
	C	D	17	0.5	17.5
	B	C	6	11.5	17.5
	A	B	100	0.5	100.5

The equivalent pipe length values should be added to the developed branch lengths to obtain the total equivalent length for each section (here, developed length refers to the actual physical length of each pipe section). For each section along the critical path, multiply the total equivalent length value in table 5.11 by the corresponding actual head loss (per 100 ft) values in Table 5.9 and Table 5.10 to obtain the actual friction loss (psi) per each equivalent section of pipe. Because the IPC method is based on the worst-case scenario, this must be done for the critical path only. The resulting values for hot and cold water branches are shown below in Tables 5.12 and 5.13, respectively (blank values represent branches that are not on the critical path).

Table 5.12: Actual Head Loss per Hot Water Branch along Critical Path

Section ID		Actual Head Loss	Equivalent Section Length	Actual Head Loss per Equivalent Section
Upstream	Downstream	(psi/100 feet)	(ft)	(psi)
I'	K'	13	14	1.82
I'	J'	13		
G'	I'	5	10	0.5
G'	H'	10		
E'	G'	6	10	0.6
E'	F'	8		
B'	C'	4.5		
B'	E'	9	4	0.36
A'	B'	4.5	8.5	0.3825

Table 5.13: Actual Head Loss per Cold Water Branch along Critical Path

Section ID		Actual Head Loss	Equivalent Section Length	Actual Head Loss per Equivalent Section
Upstream	Downstream	(psi/100 feet)	(ft)	(psi)
N	P	7		
N	O	7		
L	N	4		
L	M	4.5		
H	L	7		
I	K	8		
I	J	3		
H	I	7		
F	H	10		
F	G	5	5.5	0.275
D	F	4.5	22.5	1.0125
D	E	8		
C	D	5	17.5	0.875
B	C	5	17.5	0.875
A	B	5	100.5	5.025

The final step is to sum all actual friction losses through each equivalent section of the critical path. The resulting value is the theoretical maximum pressure loss the system may incur. Here, the total is equal to 11.7 psi. This value must be compared to the pressure available friction loss previously obtained as 25.8 psi. The actual friction loss subtracted from the available friction loss yields 13.9 psi, which is the net pressure left over. This value must be non-negative for a hydraulically feasible design. Optimally, net pressure should equal zero, which means that all energy losses equal the available pressure at the street water main. In reality, net pressure should be slightly larger than zero providing a factor or safety into the design. Table 5.14 summarizes the hydraulic design characteristics for Network Example 2.

Table 5.14: Summary of Design Calculations

1	Minimum Daily Static Pressure:		50	psi
2	Max Elevation Difference (19'):		8.2	
3	Highest Pressure Req'd @ Fixture		8	psi
4	Special Equipment Losses			
	i	water meter	6	psi
	ii	hot water heater	1.5	psi
	iii	tap in water main	0.5	psi
5	Total Losses due to Special Devices and Elevation Head		24.2	psi
6	Pressure Available to Overcome Pipe Friction		25.8	psi
7	Hydraulically Most Distant Pipe Run		180	ft
8	Pressure Loss per 100' of Pipe		14.3	Δpsi/100 ft
9	Total Actual Friction Head Loss of System		11.7	psi
10	Comparison of Allowable and Actual Losses (net pressure)		13.9	psi

In cases where the net pressure results in a highly positive or highly negative number, steps must be taken to optimize the system (force the net pressure value to slightly positive). A highly negative net pressure may indicate the need for a pressure boosting apparatus, as discussed in Chapter 3. A highly positive number means that pipe sizes should be reduced to increase friction losses. Slightly negative net pressure may be corrected by increasing pipe diameters, and hence reducing the friction loss through the system. This approach can also be applied for slightly positive numbers by decreasing pipe diameters. In this case Network Example 2, 13.9 psi could be reduced. This process would require the iterative reduction of pipe diameters and a significant amount of time to adjust “actual” friction losses, and equivalent pipe lengths.

The above procedures allow for the sizing of a plumbing water distribution based on the IPC’s recommended method, which utilizes Hunter’s curve for demand estimation. As highlighted in Network Example 1, it is possible to use other approaches, such as

EPANET, to design plumbing water networks. A simulation of Network 2 is presented in the following paragraphs.

EPANET allows the use of steady-state and extended-state network simulations. Because plumbing distribution systems are designed around a 1% failure event (a single event), we chose a steady-state simulation to model Harris' one-family residential network. We also chose to induce a worst-case scenario loading upon the system, which requires that all fixtures are operated simultaneously. Although this situation is not realistic, it does provide a basis for analyzing the behavior of the system. A pressure-driven solution approach was taken with the use of emitter coefficients to represent the pressure-dependent nature of plumbing fixtures. Figure 5B.1 in Appendix 5B shows a screen capture of the EPANET network.

The appropriate selection of emitter coefficients is crucial to accurately modeling a pressure-driven simulation. In the Network Example 1 above, a K-value of 0.02899 was applied to each demand node. It is clear that the application of a single emitter coefficient is not possible in the case of Network Example 2. The system includes the use of several fixtures with differing demands. Emitter coefficient values were derived for each fixture group. K-values are based on the largest minimum required pressure for the group and the minimum required flow for each fixture from the IPC (2000). Table 5.15 shows the minimum required pressure and flows per each fixture type.

Table 5.15: Minimum Required Flows for Fixtures (IPC, 2000)

Fixture Type	IPC Minimum Require Pressure	IPC Required Flow	Hot Flow Rate	Cold Flow Rate
	(psi)	(gpm)	(gpm)	(gpm)
Shower	8	3	1.5	1.5
Shower/Bathtub	8	4	2	2
Dishwasher	8	2.75	2.75	0
Lavatory	8	2	1	1
Sillcock (hose bib)	8	5	0	5
Sink	8	2.5	1.25	1.25
Water Closet (tank)	8	3	0	3
Washing Machine	8	2.5	1.25	1.25

The IPC only provides the total required flow rate per fixture. For fixtures requiring individual hot and cold water supplies, the total flow value was simply split in half and distributed equally between hot and cold demands. Equation (9b) was used to determine the emitter coefficient by solving for K. Table 5.16 displays the K-value results for each fixture group.

Table 5.16: Emitter Coefficients, *K*, for Network Example 2 Fixtures

Floor/Level	Group Description	Fixtures	Aggregate Demand at Fixture Group		K - Coefficient	
			Hot (gpm)	Cold (gpm)	Hot	Cold
Basement	Laundry Station	Automatic Washer	1.25	1.25	0.44	0.44
	Outside Irrigation	Hose Bib (front)				
		Hose Bib (rear)	0	5	0.00	1.77
1st Floor	Kitchen	Sink				
		Dishwasher	4	1.25	1.41	0.44
	Powder Room	Lavatory				
		Water Closet	1	4	0.35	1.41
2nd Floor	Master Bath	Lavatory				
		Bathtub/shower				
		Water Closet	3	6	1.06	2.12
	Guest Bath	Lavatory				
		Shower				
		Water Closet	2.5	5.5	0.88	1.94

Pipe diameters were obtained from the above IPC design procedure (see Tables 5.9 and 5.10). Using these diameters allows for an analysis of the adequacy of Hunter's demand estimation approach. Diameter adjustments were made to correct for nominal pipe sizes provided by the IPC. For example, a ¾" pipe is not literally ¾" inside diameter, but actually .785" (Type L copper pipe). The tables below provide the nominal and actual inside diameter values for Type K and Type L copper pipe.

Table 5.17: Nominal and Actual Inside Diameters of Type K Copper Pipe

Nominal Size	Actual ID
0.375	0.402
0.5	0.528
0.75	0.745
1	0.995
1.25	1.245
1.5	1.481

Table 5.18: Nominal and Actual Inside Diameters of Type L Copper Pipe

Nominal Size	Actual ID
0.375	0.430
0.5	0.545
0.75	0.785
1	1.025
1.25	1.265
1.5	1.505

A Hazen-Williams loss coefficient "C" was chosen as 130 for the copper pipe. Typically, copper pipe is assumed to be smooth with a C-factor of 140 – 150. Reducing the value to 130 produces conservative flow and pressure estimates, and also allows for inevitable pipe corrosion and tuberculation. Table 5.19 displays the flow rate values for the steady-state EPANET simulation.

Table 5.19: Network Example 2 – Pressure-Driven Flow Values

Branch ID		Pipe Size	Flow Rate	Velocity	Head Loss
Upstream	Downstream	(in)	(gpm)	(ft/s)	(ft/100ft)
I'	K'	0.5	2.15	2.96	16.89
I'	J'	0.5	2.62	3.61	25.84
G'	I'	0.75	4.77	3.16	10.28
G'	H'	0.375	1.17	2.59	14.11
E'	G'	0.75	5.95	3.94	12.68
E'	F'	0.5	1.77	2.44	7.92
B'	C'	0.75	4.75	3.15	9.65
B'	E'	0.75	7.72	5.12	91.35
A'	B'	1	12.47	4.85	28.48
N	P	0.75	4.73	3.14	15.01
N	O	0.75	5.31	3.52	19.78
L	N	1	10.04	3.90	12.35
L	M	0.75	4.69	3.11	19.81
H	L	1	14.73	5.73	28.65
I	K	0.5	1.75	2.41	7.82
I	J	0.75	6.22	4.12	12.22
H	I	0.75	7.97	5.29	25.85
F	H	1	22.70	8.83	254.36
F	G	1	12.47	4.85	13.53
D	F	1.25	35.17	8.98	33.98
D	E	0.5	1.77	2.44	9.51
C	D	1.25	36.94	9.43	37.25
B	C	1.25	36.94	9.43	78.39
A	B	1.25	36.94	9.73	57.80

The above table provides valuable insight into the behavior of Harris' example network. The "flow rate" column shows, under our worst-case loading scenario, that pipe links located in remote areas of the system are providing with inadequate flow values. For example, the master bathroom (branch I'J') and guest bathroom (branch I'K') should be receiving 3 gpm and 2.5gpm of hot water, respectively (by adding the demands for shower/tab and lavatory for I'J', and adding demands for shower and lavatory for I'K' from Table 5.15). Table 5.19 shows that these nodes are being supplied with slightly less flow at 2.62 gpm and 2.15 gpm, respectively. Furthermore, nodes closer to the main are being provided with excessive flows. Here, branch DE (the cold water kitchen sink pipe) is being provided with 1.77 gpm, when it only requires 1.25 gpm. The hose bib located at node J is in 20% excess of its design value of 5 gpm at 6.22 gpm. Velocity values follow a similar pattern. The IPC limitation of 8 ft/s is violated in every pipe upstream of the hot/cold water split at node F. These large velocity values can lead to erosion corrosion and premature failure of pipes. These same pipes, amongst many others, also show dangerously high head loss values. This is especially apparent when compared to the S_c design value of 14.3 ft/100ft. These high levels of water-pipe interaction produce undesirable energy losses. Table 5.20 displays the resulting head and pressure values at each node in the system.

Table 5.20: Network Example 2 – Pressure-Driven Head and Pressure Values

Node ID	Demand (gpm)	Head (ft)	Pressure (psi)
K'	2.15	32.78	5.97
J'	2.62	32.14	6.13
I'	0.00	34.47	8.00
H'	1.17	34.86	11.21
G'	0.00	35.50	12.35
F'	1.77	35.42	16.21
C'	4.75	35.17	11.34
E'	0.00	36.76	13.76
B'	0.00	37.68	14.16
A'	0.00	38.82	16.39
P	4.73	32.74	5.96
O	5.31	32.47	6.27
N	0.00	34.25	7.91
M	4.69	34.49	11.04
L	0.00	35.48	11.91
K	1.75	34.62	15.87
J	6.22	34.51	12.35
I	0.00	35.24	12.67
H	0.00	38.35	14.02
G	0.00	40.21	16.99
F	0.00	40.89	15.12
E	1.77	46.38	16.20
D	0.00	46.67	17.62
C	0.00	53.00	20.36
B	0.00	57.70	25.00
A (Street Main)	-36.94	115.50	50.00

The head and pressure values shown in Table 5.20, intuitively, display similar behavioral characteristics to the flow values in Table 5.19. Demand points located in both second floor bathrooms (nodes I', J', O, and P) are all under the IPC minimum requirement of 8 psi. The total flow demand of the system, 36.94 gpm, is roughly 40% higher than the Hunter curve demand. This value is expected due to the large demand schedule imposed on the system.

The EPANET solution methodology provides a robust tool for analyzing and designing plumbing water distribution systems. EPANET outputs are based on pure hydraulics and provide the complete hydraulic behavior of a network under specified demand loadings. Plumbing fixture operation is inherently dependent on pressure, which has been verified in AWWA's M22 manual (refer to Table 2.11). Emitter coefficients capture the pressure-dependent nature of plumbing fixtures. EPANET also provides a hydraulic blueprint of network behavior for every simulation. Because each simulation provides output at each node and each pipe, a designer has the ability to make educated decisions on which pipe(s) should be changed to optimize the net pressure value. Furthermore, any problem areas (such as the extremely large friction loss values found in the last column of Table 5.19) may be addressed before construction begins. However, there are some apparent difficulties with the EPANET approach. First, the definition of emitter coefficient values is approximate. In the above simulation, fixtures located in the same location (in the same room) were lumped together under a single K-value. These values were obtained by simply taking the IPC (2000) required minimum flows and pressures (Table 5.15) and solving for K (Equation (9b)). This may not be the best method, considering individual fixtures operate with uncertainty. Emitter coefficient values must be further refined. Second, the demand loading (demand schedule) imposed on the system must be realistic. Here, we created a worst-case scenario by turning "on" all fixtures simultaneously. This situation is highly unlikely, and therefore steps must be taken to define a more probable peak demand schedule. Complete EPANET input and report files are located in Appendices 5C and 5D, respectively.

5.9 Appendix 5A

.

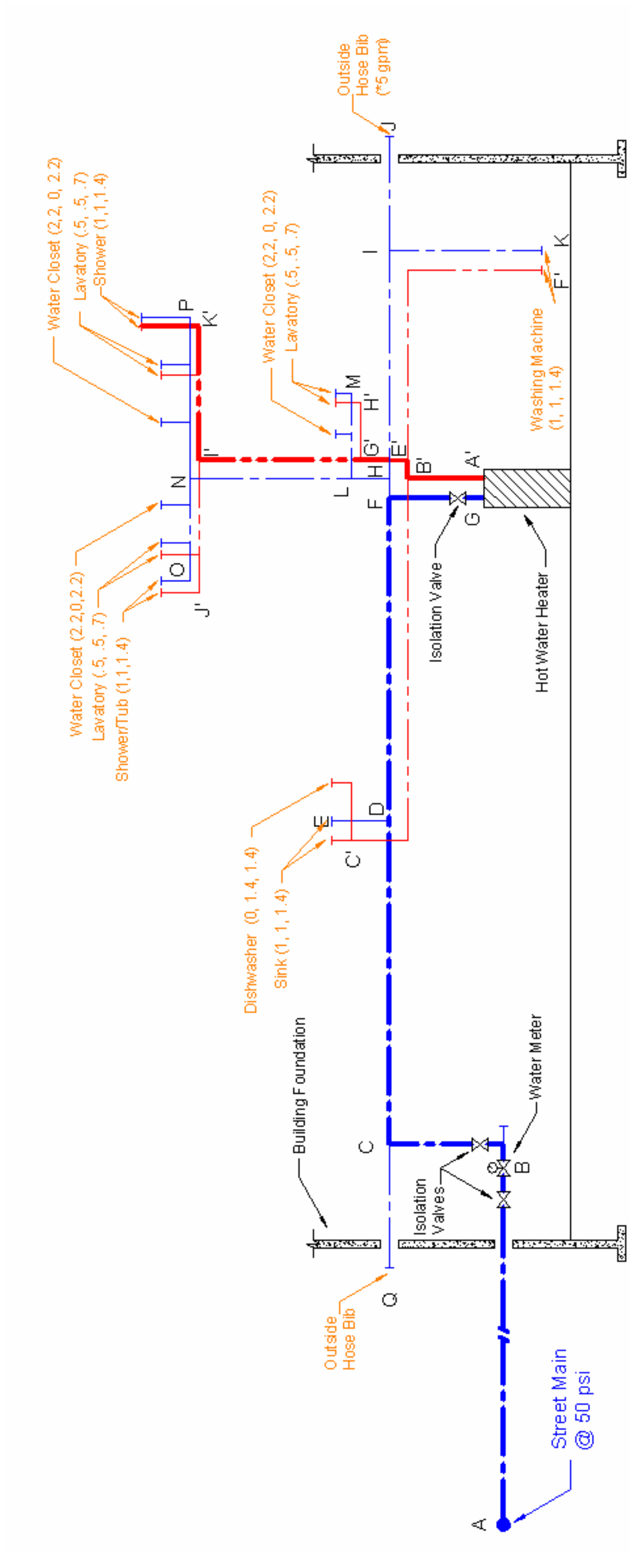

Figure 5A.1 (above) is a landscape view of Harris' typical one-family residential plumbing water distribution system. Figure 5A.1 also includes the fixture labels and their associated fixture unit values. The fixture unit values are shown in parenthesis indicating the cold, hot ,and total fixture unit values, respectively, per each fixture.

5.10: Appendix 5B

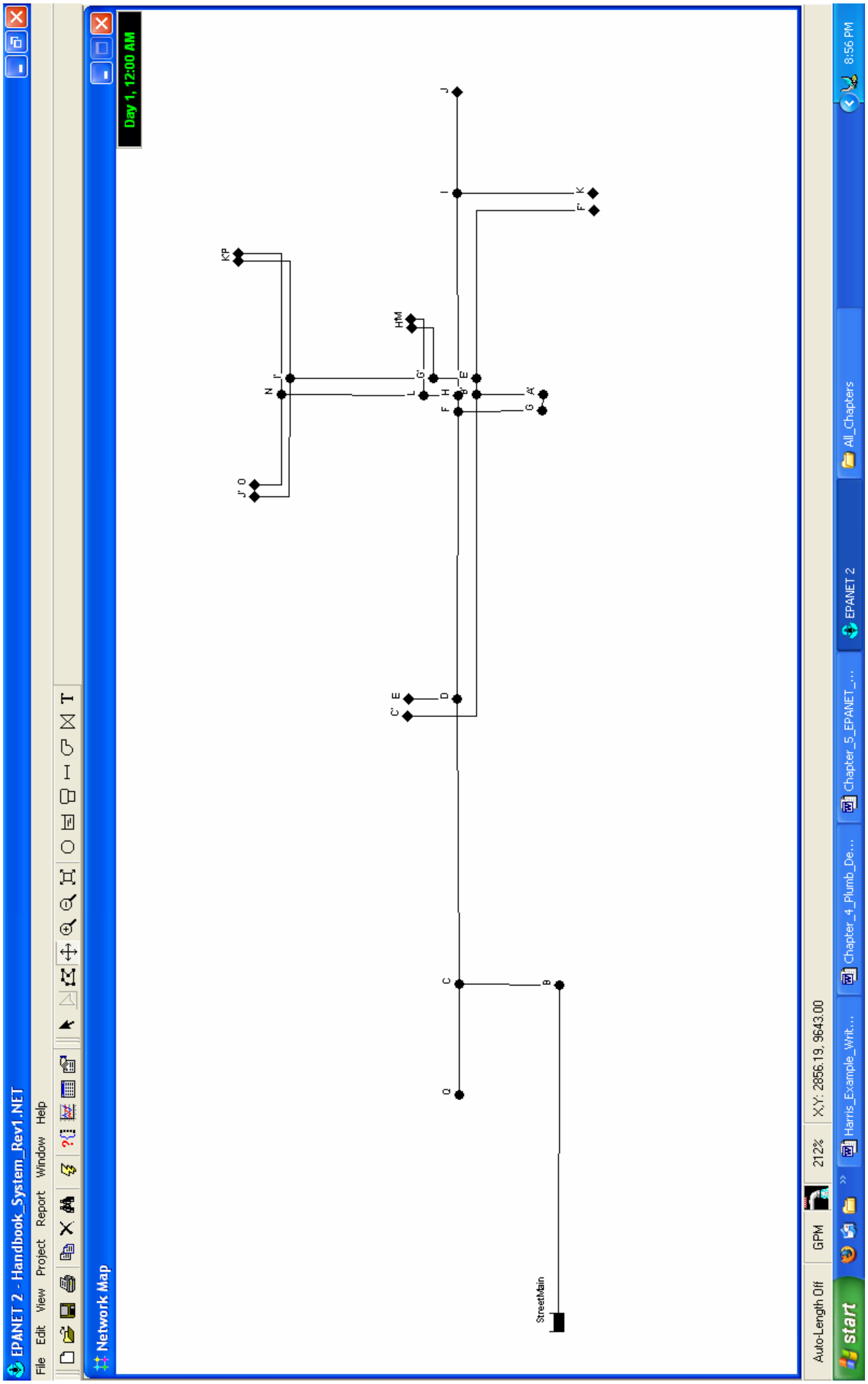

Figure 5B.1: EPANET System Interconnection

5.11 Appendix 5C

Table 5C.1 EPANET Example Network 2 Input File:

[JUNCTIONS]

;ID	Elev	Demand	Pattern
B	0	0	;
C	6	0	;
D	6	0	;
E	9	0	;
F	6	0	;
A'	1	0	;
B'	5	0	;
C'	9	0	;
E'	5	0	;
F'	-2	0	;
G'	7	0	;
H'	9	0	;
I'	16	0	;
J'	18	0	;
K'	19	0	;
H	6	0	;
I	6	0	;
J	6	0	;
K	-2	0	;
L	8	0	;
M	9	0	;
N	16	0	;
O	18	0	;
P	19	0	;
G	1	0	;
Q	6	0	;

[RESERVOIRS]

;ID	Head	Pattern
StreetMain	115.5	;

[PIPES]

;ID	Node1	Node2	Length	Diameter	Roughness	MinorLoss	Status
BC	B	C	6	1.265	130	2	Open
CD	C	D	17	1.265	130	0.6	Open
DE	D	E	3	0.545	130	0.8	Open
DF	D	F	17	1.265	130	0.6	Open
FH	F	H	1	1.025	130	1.8	Open
HI	H	I	12	0.785	130	1.8	Open
IJ	I	J	6	0.785	130	0	Open
IK	I	K	8	0.545	130	0.8	Open
HL	H	L	10	1.025	130	2.4	Open
LN	L	N	10	1.025	130	1.8	Open
A'B'	A'	B'	4	1.025	130	1.8	Open
B'E'	B'	E'	1	0.785	130	1.8	Open
E'F'	E'	F'	17	0.545	130	1.6	Open
B'C'	B'	C'	26	0.785	130	3.8	Open
E'G'	E'	G'	10	0.785	130	0.6	Open
G'H'	G'	H'	4.5	0.43	130	1.6	Open
G'I'	G'	I'	10	0.785	130	1.8	Open
FG	F	G	5	1.025	130	0.2	Open
GA'	G	A'	1	1.025	130	3.5	Open
AB	StreetMain	B	100	1.245	130	15.5	Open
CQ	C	Q	6	0.785	130	0	Closed
NO	N	O	9	0.785	130	5	Open
NP	N	P	10	0.785	130	5	Open
I'J'	I'	J'	9	0.545	130	5	Open
I'K'	I'	K'	10	0.545	130	5	Open
LM	L	M	5	0.785	130	4.2	Open

[EMITTERS]

;Junction	Coefficient
E	0.44
C'	1.41
F'	0.44
H'	0.35
J'	1.06
K'	0.88
J	1.77
K	0.44
M	1.41
O	2.12
P	1.94

[TIMES]

Duration	0		
Hydraulic	Timestep	1:00	
Quality	Timestep	0:05	
Pattern	Timestep	1:00	
Pattern	Start	0:00	
Report	Timestep	1:00	
Report	Start	0:00	
Start	ClockTime	12	am
Statistic	None		

[OPTIONS]

Units	GPM	
Headloss	H-W	
Specific	Gravity	1
Viscosity	1	
Trials	40	
Accuracy	0.001	
Unbalanced	Continue	10
Pattern	1	
Demand	Multiplier	1
Emitter	Exponent	0.5
Quality	None	mg/L
Diffusivity	1	
Tolerance	0.01	

5.12 Appendix 5D:

Table 5D.1: EPANET Example Network 2 Report File:

Link	-	Node	Table:	

Link	Start	End	Length	Diameter
ID	Node	Node	ft	in

BC	B	C	6	1.265
CD	C	D	17	1.265
DE	D	E	3	0.545
DF	D	F	17	1.265
FH	F	H	1	1.025
HI	H	I	12	0.785
IJ	I	J	6	0.785
IK	I	K	8	0.545
HL	H	L	10	1.025
LN	L	N	10	1.025
A'B'	A'	B'	4	1.025
B'E'	B'	E'	1	0.785
E'F'	E'	F'	17	0.545
B'C'	B'	C'	26	0.785
E'G'	E'	G'	10	0.785
G'H'	G'	H'	4.5	0.43
G'I'	G'	I'	10	0.785
FG	F	G	5	1.025
GA'	G	A'	1	1.025
AB	Street Main	B	100	1.245
CQ	C	Q	6	0.785
NO	N	O	9	0.785
NP	N	P	10	0.785
I'J'	I'	J'	9	0.545
I'K'	I'	K'	10	0.545
LM	L	M	5	0.785

Page	2			
Node	Results:			
--				
Node	Demand	Head	Pressure	Quality
ID	GPM	ft	psi	
--				
B	0	57.7	25	0
C	0	53	20.36	0
D	0	46.67	17.62	0
E	1.77	46.38	16.2	0
F	0	40.89	15.12	0
A'	0	38.82	16.39	0
B'	0	37.68	14.16	0
C'	4.75	35.17	11.34	0
E'	0	36.76	13.76	0
F'	1.77	35.42	16.21	0
G'	0	35.5	12.35	0
H'	1.17	34.86	11.21	0
I'	0	34.47	8	0
J'	2.62	32.14	6.13	0
K'	2.15	32.78	5.97	0
H	0	38.35	14.02	0
I	0	35.24	12.67	0
J	6.22	34.51	12.35	0
K	1.75	34.62	15.87	0
L	0	35.48	11.91	0
M	4.69	34.49	11.04	0
N	0	34.25	7.91	0
O	5.31	32.47	6.27	0
P	4.73	32.74	5.96	0
G	0	40.21	16.99	0
Q	0	53	20.36	0
Street Main	-36.94	115.5	0	0

Link	Results:			
------------------------------------- ---------------------------------				
Link	Flow	VelocityUnit	Headloss	Status
ID	GPM	fps	ft/Kft	
------------------------------------- ---------------------------------				
BC	36.94	9.43	783.94	Open
CD	36.94	9.43	372.51	Open
DE	1.77	2.44	95.06	Open
DF	35.17	8.98	339.79	Open
FH	22.7	8.83	2543.63	Open
HI	7.97	5.29	258.52	Open
IJ	6.22	4.12	122.16	Open
IK	1.75	2.41	78.19	Open
HL	14.73	5.73	286.53	Open
LN	10.04	3.9	123.46	Open
A'B'	12.47	4.85	284.78	Open
B'E'	7.72	5.12	913.52	Open
E'F'	1.77	2.44	79.24	Open
B'C'	4.75	3.15	96.53	Open
E'G'	5.95	3.94	126.81	Open
G'H'	1.17	2.59	141.06	Open
G'I'	4.77	3.16	102.81	Open
FG	12.47	4.85	135.26	Open
GA'	12.47	4.85	1397.13	Open
AB	36.94	9.73	577.98	Open
CQ	0	0	0	Closed
NO	5.31	3.52	197.81	Open
NP	4.73	3.14	150.13	Open
I'J'	2.62	3.61	258.38	Open
I'K'	2.15	2.96	168.89	Open
LM	4.69	3.11	198.12	Open

CHAPTER 6: Fire Demands

6.1: Introduction

This chapter provides a detailed overview of contemporary fire protection systems used in buildings. These systems are constructed with the sole purpose to protect life and property. With the advent of robust design techniques, these systems are highly effective. Post (2004) reports that since 1970 only 17 buildings 4 stories or taller, not including the World Trade Center, have suffered structural damage due to fire. The majority of fire suppression and fire control systems use water as the medium for protection, and therefore have implications on the total water demand of a building. Schulte (1999) states, "Today, virtually every multi-occupant residential and commercial building is fitted with an automatic sprinkler system and standpipe system". A discussion of fire systems, as an addition to domestic water systems, is requisite for fully encapsulating the complete water demand for a building. Water-based sprinkler systems and standpipe systems are the two most common types of suppression systems used in the U.S. (Harris, 1990). This chapter presents these two fire protection systems and their role in water consumption for buildings.

The operation of fire suppression systems differs from plumbing water distribution systems due to a fundamental difference. A fire system is activated (when functioning properly) only during times of emergency. They can be considered to be "on call" at all times, and remain static when not in use. Fire systems are designed to supply a large quantity of water over a relatively short period of time. Conversely, a plumbing water distribution system is in a continuous state of use. The water volume demanded by plumbing fixtures fluctuates over time. Furthermore, both the fire protection and plumbing water distribution systems are, in most cases, connected to the municipal water main. Differences relating to the volume, duration, and time of use creates design difficulties when designing and constructing the municipal water system.

6.2: Fire Water Supply

Mahoney (1980) states that domestic water demands are broken into three classes. These are: (1) average daily consumption, (2) maximum daily consumption, and (3) peak hourly consumption. He also states that a municipal water system should be designed to satisfy the required fire flow concurrently with the maximum daily domestic demand. This approach ensures that the municipal distribution system will be capable of handling fire flow requirements at all times of domestic use.

An overview of water distribution networks will provide background information on water supply to buildings for fire and domestic water consumption. Municipal water distribution systems are large interconnected piping networks. These systems are comprised of three pipe classifications. “Primary feeders” are pipes that connect the water source to “secondary feeders”. Primary feeders must be sized such that they can handle maximum daily water consumption rates to all built-up areas of a community. “Secondary feeders” tie the gridded distribution system to the primary pipes. Secondaries are typically looped to provide for redundancy and reliability within the network. They allow for the concentration of fire flow at any point within the distribution grid. “Distribution mains” are those pipes that run directly to fire hydrants and domestic water connections. These pipes are cross-connected with each other to create a grid (Mahoney, 1980). Water must then be supplied from the municipal water system to the building to satisfy requirements for domestic water and fire systems. Discussion of water service lines, which run from the public main to the building's domestic water connection, are not covered within this chapter. The connection between the public main and a building's fire system is accomplished with a "Private Fire Service Main". The National Fire Protection Association (NFPA) defines a private fire service main as, "that pipe and it appurtenances on private property (1) between a source of water and the base of the system riser for water-based fire protection systems, (2) between the source of water and the base elbow of private hydrants or monitor nozzles, and (3) used as fire pump suction and discharge piping, (4) beginning at the inlet side of the check valve on a gravity or pressure tank" (NFPA 24, 2002). NFPA 24 covers private fire service main requirements in great detail, and should be referenced for any questions.

Walski (2003) states that fire protection demands, with respect to the municipal distribution system, will depend on "size of the burning structure, its construction materials, the combustibility of its contents, and the proximity of adjacent buildings". The Insurance Services Office (ISO) has developed a rating system, termed "the Fire Protection Rating System", which is used in the U.S. to determine required fire flow demands. Mahoney (1980) defines this required fire flow as "the amount of water needed for fire fighting purposes in order to confine a major fire to the buildings within a block or other group complex". The ISO's Protection Rating System produces a "Needed Fire Flow", *NFF*, that can be used for design of a municipal distribution system. The *NFF* is predefined for one & two-family residences and the flow value is based on the distance between buildings (Walski, 2003). The table is shown below.

Table 6.1: Needed Fire Flow, NFF, for One and Two-Family Residences (Walski, 2003)

Distance Between Buildings (feet)	Needed Fire Flow, NFF (gpm)
More than 100	500
31 to 100	750
11 to 30	1000
Less than 11	1500

The formula below used to calculation the *NFF* for commercial and industrial buildings:

$$NFF = 18F\sqrt{A}O(X + P) \qquad (1)$$

where, *NFF* = need fire flow (gpm), *F* = the class of construction coefficient (dependent on the type of material used for construction), *A* = the effective area (ft^2), *O* = occupancy factor (referring to the type of occupant residing in the building (i.e. residential, commercial, etc.)), *X* = exposure factor (distance to and type of nearest building), *P* = communication factor (types and locations of doors and walls) (Walski, 2003). ISO defines a minimum fire flow of 500 gpm, and a maximum of 12,000 gpm. The duration of flow is also governed by ISO. "According to ISO (1998), fire requiring 3500 gpm or less are referred to as receiving "Public Fire Suppression", and those requiring greater than 3500 gpm are classified as receiving "Individual Property Fire Suppression". For fire requiring less than 2500 gpm or less, a two-hour duration is sufficient; for fire needing 3000 to 3500 gpm, a three-hour duration is used; and for fires needing more than 3500 gpm, a four-hour duration is used along with slightly different rules for evaluation" (Walski, 2003). Walski also states that providing adequate water via the municipal distribution system reduces the insurance rates for citizens connected to the system.

The positioning of fire hydrants is imperative to effectively fighting a fire. The NFPA (1986) states that, "hydrant spacing is usually determined by fire flow demand established on the basis of the type, size, occupancy, and exposure of structures". It is also advised that hydrants spacing should not exceed 800 feet, although 500 feet is typical for built-up areas. Hydrants should be placed at all street intersections (Mahoney, 1980).

The actual flow capacity of public water distribution systems, depending on the location within the network, is a relatively uncertain value. Urban and suburban expansion can strain existing systems. The capability of a public network is tested by running fire hydrant flow tests. These flow tests are conducted such that "the pressure of the system is monitored as relatively large amounts of water flow from the hydrants" (Harris, 1990). Mahoney (1980) state that a system is deemed adequate when, "it can deliver the required fire flow for the required duration of hours while the domestic consumption is at the maximum daily rate". Flow tests are typically completed by using two test hydrants. One hydrant is designated as a "test" hydrant, while the other is deemed the "flow" hydrant. The test hydrant is used to record the available static pressure during periods of domestic use only (as stated above, these measurements should be taken at time of maximum daily consumption). Next, the flow hydrant is opened and a pitot tube is used to measure the velocity pressure at the outlet. This open hydrant simulates a fire flow. At the same time, pressure at the test hydrant is measured to obtain a value of the residual pressure in the system (while domestic use and fire flow are occurring simultaneously). This flow testing procedure yields three pressure values.

The outlet flow rate from the flow hydrant is converted from velocity to flow rate by the following equation:

$$Q = 29.83cd^{2}\sqrt{p} \qquad (2)$$

where, Q = the flowrate in gpm, c = the hydrant outlet flow coefficient, d = the inside diameter of the hydrant outlet (inches), and p = the velocity pressure (psi). This flow value provides information necessary to compute a water supply curve at the tested hydrants. The static pressure is paired with a flow rate of 0 gpm and the residual pressure is coupled with the flow value derived from Equation 2. These four values create two points that are used to define the water supply curve. A water supply curve is plotted on semi-log paper where pressure is plotted versus flow raised to the 1.85 power. This water supply curve is used to determine flow capacities of the public water main (Schulte (PDF), 1999).

6.3: Fire Sprinkler Systems

The governing code for fire sprinkler systems is the National Fire Protection Association's NFPA 13 document. NFPA also produces separate codes for fire systems in one and two-family residential buildings (NFPA 13D) and in residential structures not exceeding four stories in height (NFPA 13R). This chapter deals primarily with NFPA 13. Puchovsky (1999) states, "the purpose of this standard is to provide a reasonable degree of protection for life and property from fire through standardization of design, installation, and testing requirements for sprinkler systems, including private fire service mains, based on sound engineering principles, test data, and field experience". There are a multitude of different sprinkler designs that can be used to suppress or control fires.

The designs are categorized under two classes, namely, wet-pipe or dry-pipe sprinkler systems. Dry-pipe sprinklers are used in cases where freezing becomes an issue (cold climates, refrigerator sprinklers, etc.). Wet-pipe sprinkler systems are the most commonly specified. A wet-pipe sprinkler system is defined as, “a sprinkler system employing automatic sprinklers attached to a piping system containing water and connected to a water supply so that water discharges immediately from sprinkler opened by heat from a fire” (Puchovsky, 1999). NFPA 13 (1999) allows many different types of water supplies to service the wet-pipe sprinkler systems. Supplies include, but are not limited to, the municipal water system, pressure tanks, gravity tanks, penstocks, flumes, rivers, or lakes. Harris (1990) states that “the most common and efficient source of water supply for fire sprinkler and standpipe systems is the public water main”. Gravity tanks, pressure tanks, or natural supplies (rivers, lakes, ponds, etc.) may be used if a building is not supplied with municipal water. Typically, these approaches are not used. Roy (2005a) mentions that, due to the large pressure requirements of standpipe hose connection (up to 175 psi), gravity tanks are no longer used within the United States for fire system supply. The following paragraphs highlight the main points of wet-pipe fire sprinkler system design.

Implementation and construction of sprinkler systems is based on local codes and regulations. The International Fire Code (IFC) requires the use of automatic sprinkler systems in various classes of buildings. These buildings types are presented in list form below:

1. Group “A”: These buildings represent any structure that is used to assemble large groups of people. Some examples are movie theaters, churches, sports arenas, and amusement parks.
2. Group “E”: This group encompasses buildings used for educational purposes.
3. Group “F”: This class includes factory and industrial buildings.
4. Group “H”: These are high hazard buildings that may contain materials that constitute a physical or health hazard.
5. Group “I”: This group includes institutional facilities such as, medical institutions or mental health facilities.
6. Group “M”: This group covers buildings that display and sell merchandise.
7. Group “R”: These buildings include boarding houses, hotels, assisted living facilities, permanent residences, etc.
8. Group “S”: This group includes all storage facilities, including high-piled storage.
9. Group “U”: These are buildings that fall under utility and miscellaneous. They may include sheds, garages, greenhouses, stables, etc. (IFC, 2000)

Each building group within the IFC includes different requirements and exceptions. The IFC should be referenced when determining what type of fire protection a particular building requires.

A sprinkler is a hydraulic device used to discharge water in order to control or suppress a burning fire. Contemporary fire sprinkler systems are designed to engage automatically when during a fire situation. These systems are intuitively named “automatic sprinkler systems”. Automatic sprinklers are defined by the NFPA as, “devices for automatically distributing water upon a fire in sufficient quantity either to extinguish it entirely or to prevent its spread in the event that the initial fire is out of range of sprinklers” (NFPA, 1986). The most commonly used type of sprinkler is the upright or pendant spray type. These devices are relatively simple, and are typically comprised of a frame, a flow deflector, an orifice cap, and an operating element. The flow deflector functions to disperse the discharging water into a spray, which covers a specified amount of area. The operating element is temperature sensitive. It functions to hold the orifice cap in place during times of non-emergency (Schulte, 1999). High temperatures will compromise the operating element, which then allows pressurized water to flow freely through the sprinkler orifice. The orifice construction of a sprinkler creates a pressure-dependent flow situation. Discharge from a sprinkler is modeled using the following equation:

$$Q = K\sqrt{p} \qquad (3)$$

where, Q = flowrate (gpm), K = the emitter/discharge coefficient of the sprinkler, and p = the pressure (psi). Equation 3 is derived from the general orifice flow equation. The value of K is obtained through testing, and the average value is given by the manufacturer of the sprinkler. Although there are numerous sprinkler types, the standard upright/pendent sprinkler will be concentrated upon within this chapter.

Sprinkler systems may be designed with two different methods in mind. The first being fire control, and the second being fire suppression. Fire control is the typical method used in most buildings. Puchovsky (1999) defines fire control as, “Limiting the size of a fire by distribution of water so as to decrease the heat release rate and pre-wet adjacent combustibles, while controlling ceiling gas temperatures to avoid structural damage". Fire control systems are not designed to completely extinguish a fire. Manual application of water through hoses and standpipe systems are needed for completely extinguishment. Fire control systems are the most commonly constructed systems for most building types (Schulte, 1999). Fire suppression is defined as, "Sharply reducing the heat release rate of a fire and preventing its regrowth by means of direct and sufficient application of water through the fire plume to the burning fuel surface" (Puchovsky, 1999). In other words, fire suppression is utilized to control and completely extinguish a fire in progress. These systems are usually implemented in storage areas, where the design of the system is primarily dependent on the type of storage, its height, and configuration. It is important to note that suppression systems must be designed with only Early Suppression Fast Response (ESFR) type sprinklers (Puchovsky, 1999). Only fire control systems will be covered in this chapter.

The primary piping constituents of a sprinkler system are:

1. <u>Branch Lines</u>:

   Pipes that distribute water directly to the sprinklers

2. <u>Cross Mains</u>:

   Pipes that are supplying the branch mains

3. <u>Feed Mains</u>:

   Pipes that supply the cross mains

4. <u>Risers</u>:

   Any vertical supply pipe within the sprinkler system

5. System Risers:

The above ground horizontal or vertical pipe between the water supply and the main (cross or feed) that contains a control valve and a water flow alarm device.

Wet pipe sprinkler systems may use three different network configurations to supply sprinklers with water, namely, branched, looped, and gridded systems. The simplest configuration is a branch network where cross mains are not interconnected, and all sprinklers are located on a dead-end branch line. Figure 6.1 below shows a typical branched network sprinkler system.

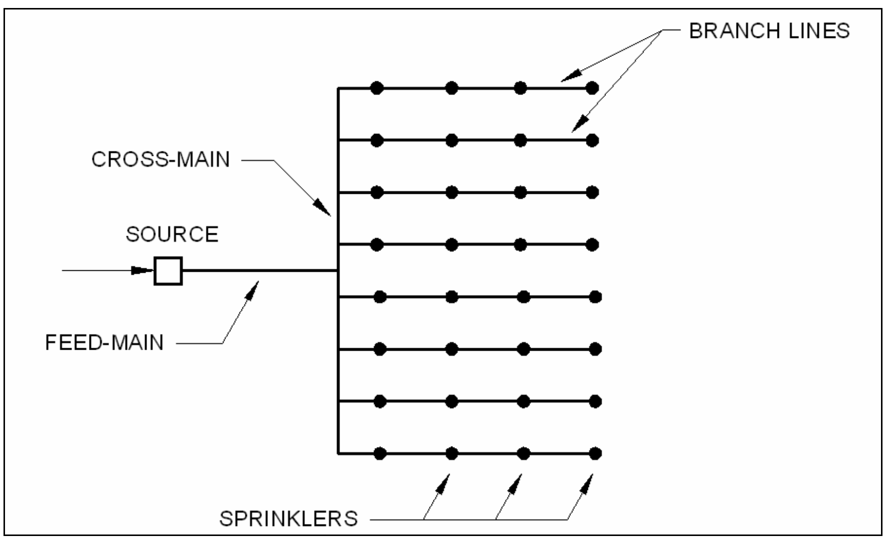

Figure 6.1: Typical Branched Sprinkler System (Plan View)

Looped systems connect multiple cross mains (perpendicular and parallel) to serve sprinkler demand points. This increases the reliability of the sprinkler system through redundancy, although the sprinklers are still located on dead-end paths. Figure 6.2 displays a looped network.

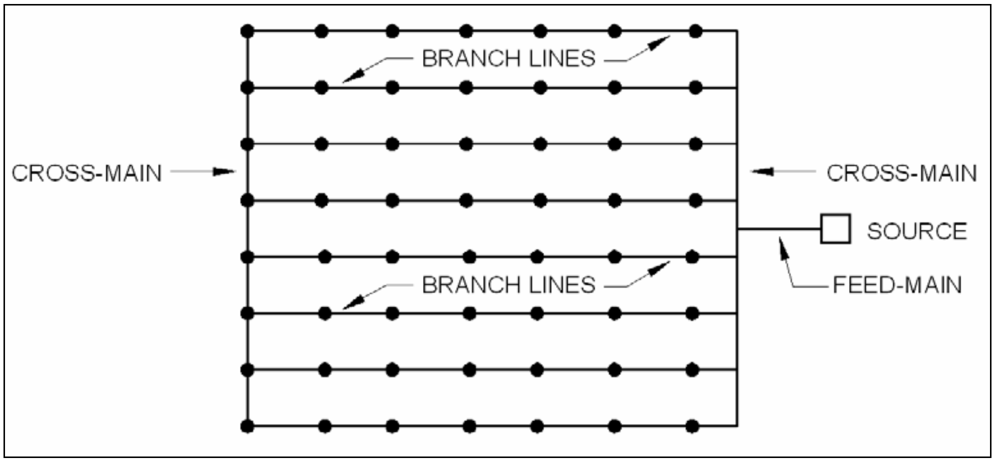

Figure 6.2: Typical Looped Sprinkler System (Plan View)

Gridded configurations connect parallel cross mains with branch lines. “An operating sprinkler will receive water from both ends of its branch line while other branch lines help transfer water between cross mains” (Puchovsky, 1999). Gridded systems are the most hydraulically complex of the three configurations. The complex flow scenarios occurring in a gridded configuration necessitate the NFPA 13 requirement that these systems be “peaked”. This is discussed in further detail under Step 2 of the hydraulic design approach for sprinkler systems. Figure 6.3 shows a typical gridded network schematic.

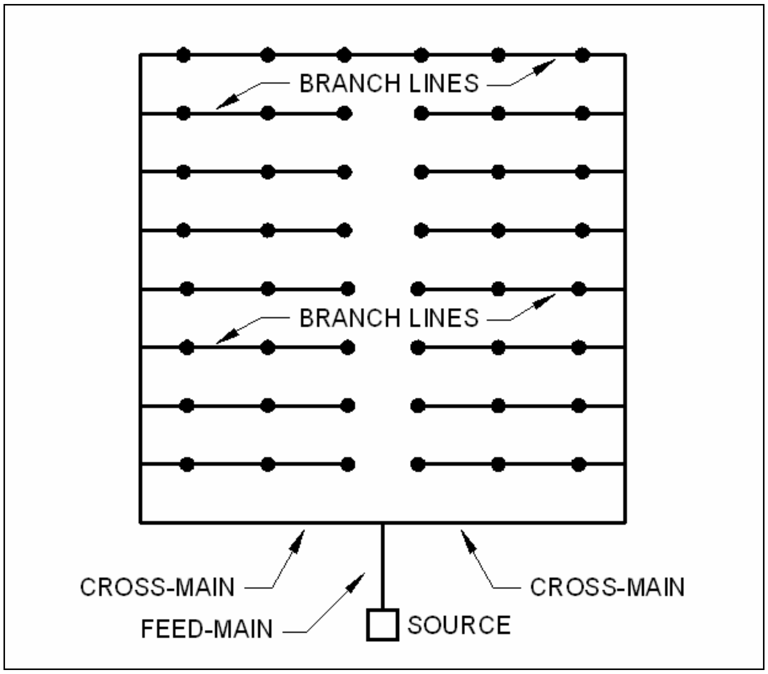

Figure 6.3: Typical Gridded Sprinkler System (Plan View)

Fire sprinkler systems are designed on the severity of the fire that could occur in a building (Harris, 1990). The severity of a fire is described by the "occupancy hazard" of a building. Occupancy hazard classifications provide a convenient means of categorizing the fuel loads and fire severity associated with certain building operations. The classifications also present a relationship between the burning characteristics of these fuels and the ability of a sprinkler system in controlling the associated types of fires" (Puchovsky, 1999). The occupancy hazard design approach is the umbrella under which all design procedures fall, and will ultimately determine the fire water demand of a building. As Schulte (1999) states, sprinkler system design "really involves only two basic engineering decisions – the available water supply at the site and the hazard classifications of the building". Occupancy hazard classifications are categorized below:

1. <u>Light Hazards</u>:
    a. These hazards are characterized by occupancies where the combustibility of contents is low and fires with relatively low rates of heat release are expected.
        i. Churches
        ii. Residential
        iii. Offices
        iv. Hospitals
2. <u>Ordinary Hazards – Group 1 & Group 2:</u>
    a. Group 1 hazards include light manufacturing and service industries where the use of flammable and combustible liquids or gases is either nonexistent or very limited
        i. Bakeries
        ii. Canneries
        iii. Dairy products manufacturing
    b. Group 2 hazards represent more severe fires, and require more demand from the sprinkler system to achieve fire control
        i. Chemical plants
        ii. Distilleries
        iii. Feed mills
3. <u>Extra Hazard – Group 1 & Group 2:</u>
    a. Group 1 hazards are those buildings where the quantity and combustibility of contents is very high and dust, lint or other materials are present. This

increases the probability that a rapidly developing fire with high rates of heat release will occur.

   i. Plywood manufacturing
   ii. Aircraft hangers

b. Group 2 hazards have moderate to substantial amounts of flammable or combustible liquids

   i. Flammable liquids spraying
   ii. Plastics processing

4. Special Occupancy Hazards:
   a. This type of hazard typically has multiple occupancy classifications, and therefore requires a specialized approach.

5. High-Piled Storage Hazards:
   a. This hazard group refers to warehouses that have commodities stored at heights of 12 feet or greater. Commodities are broken down into a class system scale. Class 1 represents non-combustible goods and Class 4 describes commodities that are high in plastic content (highly-combustible). (Puchovsky, 1999)

6.4: Fire Sprinkler Design Approaches

Designing a sprinkler system can be completed using one of two methods, namely the pipe schedule approach and the hydraulically designed approach. NFPA 13 recognizes both procedures, although Puchovsky (1999) states, “a sprinkler system designed using hydraulic analysis is preferable over those systems designed using a pipe schedule approach". The pipe schedule procedure will be briefly mentioned, but this chapter will primarily focus on the hydraulic method.

The pipe schedule method, until recently, was the traditional approach for determining the sprinkler system water demand. The 1999 edition of NFPA 13 restricts the use of pipe schedules to relatively small applications (under 5000 square feet) and only in light and ordinary hazard classifications. The use of this method is also limited to sprinklers containing ½" orifice diameters only. NFPA 13 lists copper and steel as the only options for piping material. Pipe schedules, themselves, are tables that show the maximum number of sprinklers that may be served by a specified diameter of pipe. For example, in a light occupancy hazard classification only two sprinklers may be served by a 1" diameter steel pipe. 5 sprinklers can be served by 1 ½" pipe and 10 sprinklers by 2" pipe and so on. Risers in the system are sized to supply all sprinklers on any one floor in the building. The sizing procedure starts from the most distant sprinkler on a floor and continues back to the riser connection. The number of sprinklers is accumulated from the most distant sprinkler to the riser connection. This approach can lead to friction loss problems in pipes having long runs or a large number of fittings. NFPA 13 recommends upsizing the pipe diameter to compensate for these large losses.

The hydraulic design approach is based on energy relations. There are three steps within the hydraulic design approach. The first is to determine the area of sprinkler operation, the sprinkler discharge densities, and the hose stream demands. This is a relatively simple that procedure is based on the occupancy hazard classification of the structure. The second step is to employ hydraulic calculations to ensure the sprinkler system will discharge the required volume of water over the specified area. The third is to optimally size the sprinkler piping. The entire process accomplished through the use of either the "area/density" method or the "room design" method. The following paragraphs overview the hydraulic design approach.

<u>Step1</u>

Schulte (1999) states the three main design criteria for a hydraulically designed sprinkler system are (1) assumed area of sprinkler operation, (2) the sprinkler discharge density, and (3) the hose stream demand. Quantifying the assumed area of sprinkler operation, *A*, is aids in the determination of the number of sprinklers that are assumed to operate. The sprinkler discharge density, *q*, refers the amount of water being discharged from a sprinkler per square foot. This value determines the minimum flow required at each sprinkler which is assumed to operate. The hose stream demand, q_h, is the volumetric flow rate of water required allocated to hose demands (separate from a standpipe system) in the building (Schulte, 1999).

Determining *A* and *q* begins with defining the occupancy hazard classification of the building. NFPA 13 encapsulates five occupancy hazard classification curves for light hazard, ordinary hazard group 1, ordinary hazard group 2, extra hazard group 1, and extra hazard group 2 in the plot of "area/density" curves. These curves represent the "area of sprinkler operation (ft^2)" versus "sprinkler discharge density (gpm/ft^2)". Intuitively, required discharge densities increase as the occupancy becomes more hazardous. For example, a 2500 ft^2 area requires 0.085 gpm/ft^2 for a light hazard occupancy, while an extra hazard (group 2) occupancy requires 0.40 gpm/ft^2 for the same area of sprinkler operation. Puchovsky (1999) displays the area/density curves on page 503 of his

“Automatic Sprinkler Systems Handbook”. Once the occupancy hazard classification and the area of sprinkler operation, *A*, have been identified, the sprinkler demand, *q*, (gpm) can be calculated. It is simply the sprinkler discharge density multiplied by the area of sprinkler operation. The resulting demand value represents the flow rate required by the system without any head losses present.

The area/density method is one approach for finding *A* and *q*. To use this method, only a single point must be chosen from the appropriate occupancy hazard classification curve (as discussed in the previous paragraph). For example, if a building is classified as a light hazard, the designer would use the light hazard curve. The chosen point is up to the discretion of the designer. Puchovsky (1999) cites some rules of thumb for choosing an appropriate point. A smaller area of assumed sprinkler operation, *A*, will yield a greater required discharge density, *q*, and hence greater system pressure. A larger area produces a lower density, but will require an increased total water demand. He declares, “(a smaller area of sprinkler operation) is generally considered superior in terms of fire control and is expected to confine the fire to a smaller area, reducing the total number of operating sprinklers”. NFPA 13 also states that for actual sprinkler areas (the physical area of the room or building) less than the minimum values indicated on the area/density curves, the minimum values shall be used for design. For example, a light hazard occupancy of 1000 ft^2 requires a minimum design area of 1500 ft^2, which is the smallest area represented on the area/density curves. The sprinkler discharge density, 0.1 gpm/ft^2, associated with a 1500 ft^2 area must be used for design.

The room design method is the second approach for determining the assumed area of sprinkler operation, *A*, and discharge density, *q*. Here, water supply requirements are based on the room that requires the greatest water demand and pressure (Puchovsky, 1999). The term "room" has a loose definition. If a large room communicates to a smaller room through unprotected corridors, then the two rooms and the corridors must be considered as a single room. Here, protection refers to walls that are required to have a fire-resistance rating equal to the water demand duration as stated in Table 6.2. The assumed area of sprinkler operation is analogous to the area of the most demanding room (or series of rooms and corridors). This area value corresponds to a single point along the appropriate area/density curve, which automatically yields the required sprinkler discharge density. As with the area/density method, if the most demanding room's area is below the minimum area in the area/density curve, then the minimum area and associated sprinkler discharge density shall be used for design (NFPA 13, 1999).

The hose stream demand for hydraulically calculated fire sprinkler systems is given in tabular form in NFPA 13. This table is shown below.

Table 6.2: Hose Stream Demand and Water Supply Duration Requirements (NFPA, 13)

Occupancy Classification	Inside Hose	Total Combined Inside and Outside Hose	Duration
	(gpm)	(gpm)	(mins)
Light Hazard	0, 50, or 100	100	30
Ordinary Hazard	0, 50, or 100	250	60 - 90
Extra Hazard	0, 50, or 100	500	90 - 120

The total sprinkler system water demand is determined by adding the requirements of the hose stream and the sprinklers. This value may change a small amount after the hydraulic calculations have been completed.

Step 2:

Puchovsky (1999) states, with respect to the hydraulic calculations for a sprinkler system, "A combination of factors must be integrated into the design, including the hazard, the spacing of sprinklers, the type of pipe, and the type of sprinkler system". Calculations typically begin at the hydraulically most distant sprinkler in the system and work back to the water supply. The procedure is outlined in the following paragraphs.

First, identify the hydraulically most demanding area within the building. This area corresponds to the point in the fire sprinkler system which requires the greatest amount of water flow and water pressure. By beginning the analysis at this area, all intermediate sprinklers within the system are guaranteed to be supplied with adequate flow and pressure. Gridded systems present complex hydraulic interactions. When using these types of networks, NFPA 13 requires that a minimum of three separate calculations must be performed to verify the most demanding area is used. This procedure is known as "peaking".

Second, configure the design area of the most hydraulically distant area within the building. From Step 1, the area of assumed sprinkler operation, *A*, is already known (from either the area/density method or the room design method). When using the area/density method, NFPA 13 places restrictions on the dimensions of the design area, *A*. Equation 4 guarantees that the resulting assumed area of sprinkler operation, *A*, will be a rectangle:

$$L \geq 1.2\sqrt{A} \qquad (4)$$

where, *L* is the distance parallel to the branch lines (ft). Puchovsky (1999) states, "the rectangle is required to be longer in the dimension parallel to the branch lines because this arrangement accounts for the possibility that a fire could spread in this direction and open multiple sprinklers on a single branch line before opening sprinklers on other branch lines". When using the room design method, the dimensions of the assumed area of operation, *A*, are pre-defined by the floor area of the room and its communicating spaces.

Third, find the area of discharge per sprinkler, A_s. This value is defined by the horizontal area between the sprinklers on the branch line and the adjacent branch lines (NPFA 13, 1999). A_s is determine using the following equation:

$$A_S = S * L \qquad (5)$$

where, A_s is area of sprinkler operation (ft^2), *S* is the distance between sprinklers on a branch line (ft), and *L* is the perpendicular distance between branch lines (ft). NFPA 13 constrains the maximum protection area of any sprinkler to be less than 400 ft^2. For standard pendent and upright spray sprinklers, which are focused on within this text, the maximum area of protection, A_s, and sprinkler spacing distance, *S*, are based on the occupancy hazard classification and design approach. The values for hydraulically designed systems are shown in Table 6.3.

Table 6.3: Protection Areas and Maximum Spacing for Standard Pendent and Upright Spray Sprinklers (NFPA, 1999)

Occupancy Class	Protection Area "A_s"	Distance "S"
	(ft^2)	(ft)
Light	225	15
Ordinary	130	15
Extra (sprinkler discharge density ≥ 0.25)	100	12
Extra (sprinkler discharge density < 0.25)	130	15

The lengths S and L are left to the designer, but they must fit within the constraints shown in Table 6.3. These values are applicable to both area/density and room designed systems.

Fourth, calculate the number of sprinklers, N, within the assumed area of operation, A. This value is calculated using the following formula:

$$N = \frac{A}{A_S} \qquad (6)$$

Clearly, the value of N will not always be a whole number. For all values of N that result in a decimal, the number must be rounded up to the next whole sprinkler.

Fifth, the number of sprinklers along a branch line, N_b, and the number of branches within the assumed area of operation, B, must be calculated. The following equation is used to obtain this value,

$$N_b = \frac{1.2\sqrt{A}}{S} \qquad (7)$$

Again, any resulting decimal values must be rounded up to the next whole sprinkler. B is calculated by simply dividing N by N_b.

Finally, determine the amount of water and pressure required to meet the demands stipulated in the above steps. Calculations begin by finding the required pressure at the most distant sprinkler within the assumed area of operation, A. The required discharge density per sprinkler, q (gpm/ft^2), has been defined by the area/density curves. The required flow rate per sprinkler is obtained by multiplying the density, q, with the sprinkler's protection area, A_s (ft^2). The required pressure value is calculated using the emitter equation (Equation 3) $Q = K\sqrt{P}$. Here, the K-value is obtained from the sprinkler manufacturer. The resulting pressure, P, is the required pressure at the outlet of the most distant sprinkler within the assumed area of sprinkler operation, A. The hydraulic calculations proceed by working upstream towards the water supply. Pressure losses must be calculated along branch and main lines to obtain the additional flow values demanded by upstream sprinklers within the assumed area of operation, *A*. NFPA 13 requires that friction losses must be determined using the Hazen-Williams formula. NFPA 13 presents the equation in the following notation:

$$h_f = \frac{4.52Q^{1.85}}{C^{1.85}d^{4.87}} \qquad (8)$$

where, h_f is head loss (ft/100ft), Q is the flow rate (gpm), C is the Hazen-Williams friction loss coefficient, and d is the internal pipe diameter (in). The loss coefficient is dependent on the pipe material. The inside diameter, d, is an unknown parameter. Puchovsky (1999) suggests that "an initial pipe sizing can be based on the pipe schedule tables (for the specified occupancy hazard classification)". Using the discharge value at the most distant sprinkler, along with the loss coefficient and initial pipe diameter, a friction loss value can be obtained for the pipe link connected to the upstream sprinkler. The head loss value, h_f, is multiplied by the sprinkler spacing, S, to obtain the actual pressure loss, ΔP, between the two most distant sprinklers. Note that all elevation differences and minor losses must be included in the ΔP calculation between each sprinkler. This is shown in the following equation:

$$\Delta P = h_f * S \qquad (9)$$

where, ΔP is the pressure loss between successive sprinklers (psi). The flow value at the upstream sprinkler is obtained by using the emitter equation (Equation 3), where the pressure value is now ΔP added to the pressure at the downstream sprinkler, P. The K-value is a constant so long as the sprinkler type and orifice size are identical. The computed upstream sprinkler flow is added to the downstream value to obtain the total required flow at the upstream sprinkler. This procedure is repeated for all sprinklers, N_b, on a branch line. Once the demand for an entire branch has been computed (meaning that the hydraulic calculations have been run from the most distant sprinkler on a branch back to the cross-main), a new K-value can be determined for the entire branch. The emitter equation (Equation 3) is used to obtain this value by inputting the total demand for that branch and the associated pressure at the cross-main/branch line node. All identical branches can use the new K-value to determine their individual flow values. It is common to have a branch line that does not have all its sprinklers within the assumed area of operation, *A*. In this case, the sprinkler(s) closest to the cross-main should be included in the summation of water demand values (NFPA 13, 1999). Hazen-Williams equation is then run from the area of operation, *A*, back to the water supply source to determine the required pressure, P_R for the sprinkler system. The total water supply value, *Q*, is obtained by adding all the sprinkler demands and the hose stream demands.

Step 3:

The hydraulic calculation procedure allows for the sprinkler system to be optimized. The water supply curve generated from hydrant flow tests provides the available pressure and capacity of the municipal distribution system. A sprinkler system demand curve can be similarly constructed. First, the total elevation difference between the water source and the sprinklers must identified. The first point of the demand curve, P_0, occurs at a demand of 0 gpm and a pressure value equal to the total elevation difference (1 psi is roughly equivalent to 2.31 feet of head). The second point, P_1, of the curve is defined by the total water demand, Q, for the sprinkler system and the required pressure, P_R. Connecting the two points yields the fire sprinkler system demand curve. This curve can be superimposed onto the water supply plot. The resulting hydraulic graph directly compares the system demand requirements to the available street main water supply. Puchovsky (1999) states, "A hydraulic analysis demonstrates if the water supply is adequate for the sprinkler system demand". A sample curve is displayed on page 668 in Puchovsky's text. Adjustments to optimize the sprinkler system can be made based upon the resulting demand and supply curves. For example, if the demand curve runs above the suppler curve, then steps must be taken to reduce the friction loss of the piping system. This can be done by iteratively increasing the size of the piping. In extreme cases, such as in high-rise buildings, fire pumps may be required to boost the pressure of the system. If the demand curve falls below the water supply curve, pipe sizes may be decreased. This is an ideal situation because smaller diameter pipes will reduce the initial monetary investment of the owner.

6.5: Fire Standpipe Systems

A fire standpipe system is defined as, "an arrangement of piping, valves, hose connections, and allied equipment installed in a building or structure, with the hose connections located in such a manner that water can be discharged in streams or spray patterns through attached hose and nozzles, for the purpose of extinguishing a fire" (NFPA 14, 2003). Cote (1986) asserts that these systems provide a means for manually applying water to a fire through the use of hose. He also states that standpipe systems are designed to "provide quick and convenient means for obtaining effective fire streams on large low buildings, or the upper stories of high buildings". A "standpipe" refers to the portion of system piping that delivers water supply to the hose connections (NFPA 14, 2003). Standpipe system design and construction is regulated by the National Fire Protection's NFPA 14 document entitled "Installation of Standpipe and Hose Systems". As with fire sprinkler systems, there are various standpipe system designs that can be utilized. "Wet standpipe" systems contain pressurized water at all times. An automatic wet pipe system, "is attached to a water supply capable of supplying the system demand at all times and that requires no action other than opening a hose valve to provide water at hose connections" (NFPA 14, 2003). Harris (1990) states that these are the most commonly employed systems, and therefore will be concentrated upon within this chapter.

Standpipe systems can be constructed under three different classifications. The classification depends upon the intended personnel who will use the standpipe system. For example, some systems are designed specifically for trained fire-fighters only. Other systems are classified such that an untrained tenant may use them. NFPA 14 defines the three classifications as:

Class I Standpipe Systems: provides 2 ½” hose connections to supply water for use by fire departments and those trained to handle heavy fire streams.

Class II Standpipe Systems: provides 1 ½” hose connections for use by building tenants and fire departments for initial fire response.

Class III Standpipe Systems: Provides 1 ½” hose connections for tenant use, and 2 ½” connections for fire department use.

Implementation of a specified classification is dependent on local codes and regulations. The International Fire Code (IFC) allows the use of all three classes, subject to the building type in which the standpipe system is being installed. The 2000 edition of the IFC states that Class III standpipe systems shall be installed “throughout buildings where the floor level of the highest story is located more than 30 feet above the lowest level of the fire department vehicle access, or where the floor level of the lowest story is located below the highest level of fire department vehicle access”. Class I systems are required for buildings that exceed 10,000 square feet in area per floor. These systems must be installed “where any portion of the building’s interior area is more than 200 feet of travel, horizontally or vertically, from the nearest point of fire department vehicle access” (IFC, 2000). In addition to IFC regulation, Roy (2005c) states, “Class I systems should be provided in all new high-rise buildings, although some designs still utilize a Class III system”. The code should be consulted for exceptions to the above rules. It is important to note that the Class II systems are not included under the IFC’s “required installations” section. Furthermore, the IFC defines the locations of fire hose connections in each of the three standpipe class systems. In general, hose connections should be located such that a hose stream can reach any point within the building. Stairwells are a common place to find standpipe hose connection valves. Below are pictures taken in a stairwell at the west end of Durham Hall, located on the Virginia Tech campus in Blacksburg, Virginia. Figure 6.4 shows the standpipe riser on the right-hand side (the large diameter pipe). Figure 6.5 displays the associated fire hose connection valve, which is located about 3' above the floor.

Figure 6.4: Fire Standpipe Riser

Figure 6.5: Hose Connection Valve

NFPA 14 states that "the design of a standpipe system is governed by building height, area per floor occupancy classification, egress system design, required flow rate and residual pressure, and the distance of the hose connection from the source(s) of the water supply". As with any system, there are constraint values that must be adhered to. The maximum pressure at any point in a standpipe system is limited to 350 psi. NFPA 14 (2003) states, "350 psi was selected because it is the maximum pressure at which most system components are available, and it recognizes the need for a reasonable pressure unit". The maximum pressure within a system is most likely to occur, due to large elevation head values, at the bottom of a riser or at the discharge of a fire pump. NFPA 14 (2003) also limits the pressure values occurring at the hose connections. When residual pressures at 1 ½" connections exceed 100 psi, a pressure regulating device must be installed to limit the pressure to 100 psi. When static pressures exceed 175 psi, regulating devices must reduce 1 ½" connections to 100 psi (static and residual) and 2 ½" connections to 175 psi (static and residual).

The three classes of standpipe systems induce different water demands on the water supply. Class I and III systems, which include 2 ½" hose connections, demand more water than Class II systems having smaller 1 ½" connections only. NFPA 14 includes an outline for calculating the minimum flow necessary for each system classification. Class I and III systems must have a minimum of 500 gpm at the hydraulically most remote standpipe. Each additional standpipe in the system must draw a 250 gpm demand. The total demand limit for fully sprinklered buildings is 1000 gpm, and 1250 gpm for unsprinklered or partially sprinklered structures. Class II standpipes require a minimum demand of 100 gpm at the most distant hose connection. No additional flow demands are required for systems two or more hose connections. Furthermore, maximum demand values limit 2 ½" connections at 250 gpm, and 1 ½" connections at 100 gpm.

6.6: Fire Standpipe Design Approaches

NFPA 14, like NFPA 13, allows for the use of either a pipe-schedule method or a hydraulic calculation method for system sizing and design. The pipe-schedule tables provide sizing requirements based on "total accumulated flow" and "total distance of piping from farthest outlet" (NFPA 14, 2003). The pipe-schedule method is not considered any further for standpipe sizing. For hydraulic calculations, Class I and III standpipes must be at least 4" in diameter. No minimum diameter for Class II standpipes is stated within NFPA 14. The number of required standpipes within a building is equal to the number of individual exit stairways. Standpipes must be interconnected when two or more are installed in a single structure. For systems that are supplied by tanks on the top of the building/zone (a rare situation within the United States), the standpipes must be interconnected at the top. Hydraulic calculations are carried out in a similar fashion to sprinkler systems. First, the design demand values must be determined. NFPA 14 requires that each standpipe within a Class I or III system be designed such that it will provide 250 gpm to the two most distant hose connections and 250 gpm at the at the topmost outlet of each of the other standpipes. The pressure at these connection points must be consistent with the aforementioned minimum residual pressure values (100 psi for 2 ½" connections, 65 psi for 1 ½" connections). Supply piping, which interconnects standpipes to the water supply, must be sized to service all standpipes up to a total demand flow of 1250 gpm. Class II systems require that each standpipe must be sized to provide 100 gpm at the most distant hose connection. The minimum residual pressure at 1 ½" inch connections is 65 psi. Supply piping should be sized to carry 100 gpm. The hydraulic calculations begin at the most distant hose connection and continue back to the water supply. Any losses due to friction, flow perturbations (minor losses), and elevation changes must be accounted for in the calculations. "Any extra demands given by either standpipes or sprinklers must be added where they occur along the hydraulic design path" (NFPA 14, 2003). The hydraulic design path refers to the run of piping from the most distant connection to the water supply source. Major friction losses should be computed with the Hazen-Williams formula, and any minor losses should be converted to equivalent pipe lengths (NFPA 14 provides a table for equivalent pipe lengths of common standpipe fittings). Hydraulic calculations yield the operating pressure range of the system, or in other words, the standpipe system has a complete map of all pressure values within the system. This information is vital to the correct placement of pressure reducing devices, and to ensure that all points within the system satisfy pressure requirements. Most importantly, the hydraulic calculations provide the system water demand and required pressure at the supply source. Comparing the water demand requirements to the water supply curve derived from the hydrant flow test, standpipe systems may be iteratively resized to determine the most optimal piping configuration (similar to sprinkler systems).

NFPA 14 allows various water sources for supply including, the municipal water system, pressure tanks, and gravity tanks. Similar to sprinkler systems, the public water main is the most convenient and reliable source of water for standpipes (Harris, 1990). NFPA 14 defines acceptable water supply sources when "the pressure available at each supply source exceeds a standpipe system's pressure demand at the designated flow, the design is acceptable". In addition, NFPA 14 also requires that water supply sources for Class I, II, and III systems must be able to provide design flow and pressure for a period of no less than 30 minutes. Acceptability is determined by superimposing the standpipe demand curve on the water supply curve (developed by hydrant flow tests). This procedure is identical as that described for fire sprinklers. If a system is deemed unacceptable, then steps must be taken to adjust the piping design or the water supply. High-rise buildings, due to relatively large elevation head values, will often create unacceptable water supply scenarios. These situations are solved by utilizing pressure boosting equipment such as a fire pump. Extremely tall buildings may cause further complications related to the maximum pressure constraint of 350 psi. For example, a 100 story building (at 10 feet per floor) would require a standpipe fire pump to move water against 1000 feet of head, or roughly 435 psi. 2 ½" hose connections require an extra 100 psi residual pressure, and frictional losses would require even more energy. Clearly a single zone standpipe system would be design folly, requiring large number of pressure reducing devices and extra-strength pipes and equipment. A vertical zoning configuration is the best solution to these types of situations. Here, break tanks and booster pumps can be strategically placed within the system to meet all flow and pressure requirements. NFPA 14 requires that, "each zone requiring pumps shall be provided with a separate pump". This type of approach is very similar to pressure boosting application in plumbing water distribution systems.

6.7: Combined Systems

Combined systems utilize common piping between the fire sprinkler and the fire standpipe systems. NFPA 13 and NFPA 14 both allow for combined systems. Roy (2005b) states that these shared piping arrangements dramatically reduce the cost of installation and operation and maintenance, which therefore makes reliable fire protection economically feasible for a greater range of buildings. He also notes that a combined system, when designed properly, will not reduce the reliability of the system. Upon further inspection of the standpipe system located in Durham Hall on the Virginia Tech campus, it can be deduced that this system is, in fact, a combined standpipe and sprinkler arrangement. Figure 6.6 below, which is a blown up view of Figure 6.4, shows two branches originating at the vertical standpipe and traversing horizontally along through the building. Notice the upper tag reads "standpipe riser" and the lower tag reads "2nd floor sprinklers". Construction of Durham Hall was completed in 1998. These pictures illustrate how contemporary fire engineering procedures are utilized to provide fire protection and simultaneously reduce costs.

Figure 6.6: Durham Hall Combined Sprinkler/Standpipe System

Intuitively, NFPA 13 and NFPA 14 must be used in conjunction with one another to achieve an acceptable combined system design. The following paragraphs highlight the design procedure required to construct a combined sprinkler/standpipe system. As a note, it is assumed that the building is fully sprinklered.

The water demand for a combined system is determined by comparing the flow values for the sprinkler system and the standpipe system, individually. Harris (1990) states, “if a building is completely equipped with fire sprinkler system protection, the water supply for a combined fire sprinkler and standpipe system need only be sized to accommodate the larger of the separate fire sprinkler or standpipe system requirements”. NFPA 14 cites that the demand value determined for the standpipe system can also supply the sprinkler system, but in cases where the sprinkler demand exceeds the standpipe value, the larger demand must be provided. NFPA 13 reiterates by stating, "the water supply (for wet pipe sprinkler systems) shall not be required to be added to standpipe demand as determined from NFPA 14". It also includes that if the sprinkler system demand and hose stream allowance exceeds NFPA 14 standpipe requirements, "this higher demand should be used". Unfortunately, the wording and nomenclature of the two NFPA documents do not identically match each other. It is important to note that NFPA 14 requires that combined systems must have a minimum standpipe diameter of 6 inches, although 4 inches pipes are allowed when hydraulic calculations (as opposed to pipe schedule methods) have been used. All system constraints and limitations cited by NFPA 13 and 14 must be adhered to for combined systems (see above sections). All applicable hydraulic calculations must be followed as previously stated.

Correctly designing a combined system is reliant on knowing what code, NFPA 13 or 14, to apply. Roy (2005b) states, "As a designer, I would use NFPA 14 for light and ordinary hazard occupancies and use the standpipe piping for sprinkler water distribution". Roy (2005b) provides the following advice for designing a shared pipe system. It is important to note that Roy has stated this design procedure based on the assumption that standpipe system will typically yield a greater demand value than a sprinkler system. His assumption is supported by Harris (1990) who says, "Generally, the standpipe system has the larger water supply requirement". First, begin the design process with NFPA 14. Determine the location and number of risers for the system, while satisfying the aforementioned minimum pipe diameter limitations. Use NFPA 14 to determine both capacity and residual pressure requirements for the standpipe system (using Hazen-Williams as the applicable friction loss equation). Now, locate and size sprinklers based on NFPA 13. If the water supply is municipal main, design the water supply piping according to NFPA 24. Determine the adequacy of the water supply be comparing the water supply curve to the combined system demand curve. If the municipal main is inadequate, size a fire pump to meet the pressure and flow requirements.

CHAPTER 7: Summary and Recommendations

The key contributions of this thesis are the following. To our knowledge, for the first time, a complete synthesis of plumbing design methodology is presented along with the supporting theory. The intricate details pertaining to the probabilistic nature of demands are presented. Because of the much smaller travel times occurring in plumbing distribution systems, small diameter pipes are sufficient to provide for the intermittent demands incurred by only a subset of fixtures. The estimate of the probability of fixture use as the ratio of the duration to time between uses should be evaluated over a peak usage period.

The lag times between fixture uses are relatively long. These facilitate long contact durations between the pipe wall and the water throughput, which promotes water quality related activities. Edwards (2004) states that lead leaching and corrosion can occur (under certain circumstances) in plumbing systems, and these actions are dependent on the contact time between water and pipe material

We have also suggested modifying the minimum pressure criterion for the most hydraulically distant fixture. Because the underlying principle is to provide water supply to each node, the idea of critical slope, which is the least energy slope, is crucial. We suggest calculating this slope from the maximum required pressure at the hydraulically most distant fixture group. Once the process is automated by enumerating each path, the critical path with the least energy slope can easily be found.

We have also proposed the pressure-driven formulation as the means to numerically model the plumbing systems. Both Excel SOLVER and EPANET can be used. Estimating the emitter coefficient does pose some difficulties. The formulation lends itself both for analysis and design. By changing pipe diameters, say, within the framework of a Genetic Algorithm, an optimal set of diameters can be calculated. A useful appendix, namely, “Primer on EPANET”, is included to provide a detailed overview explaining all steps required to a run a pressure-driven simulation in EPANET.

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APPENDIX 1: EPANET Primer

The following is a document that sequentially lists steps for running an EPANET pressure-driven model of a plumbing distribution system. It is vital to note that this is not a typical “demand-driven” application. Plumbing fixtures (demand nodes) are represented with emitter coefficients. An emitter coefficient is a value that relates pressure and discharge by

$$Q = K\sqrt{P} \qquad (1)$$

where, Q is the flowrate in gallons per minutes (gpm), K is the emitter coefficient, and P is the pressure differential over the emitter in pounds per square inch (psi). This equation is a direct derivation from a fluid mechanics orifice equation. Emitter coefficients have been derived for specific plumbing fixtures. These values are based on International Plumbing Code flow requirements for said fixtures.

1. It is highly recommended to first create a scaled drawing of the water distribution network. An elevation view or isometric view can be used. This can be accomplished through the use of AutoCAD, or other drawing programs. AutoCAD has the capability to export a drawing file (*.dwg) as a bitmap file (*.bmp). EPANET allows bitmap files to be imported as backdrops for the drawing surface. This is very useful for sketching the pipe network in EPANET due to program’s limited drawing capabilities.

a. Converting an AutoCAD drawing file to a bitmap file:

   i. Once the drawing has been completed in AutoCAD, go to the "File" dropdown at the top left hand corner of the screen

   ii. Select the "Export" option

      1. The "Export Data" dialogue box will pop up.

      2. Using the "Save in" scroll menu at the top of the dialogue box, choose the destination where the file will be saved.

      3. At the bottom of the screen, input the desired file name into the "File Name" scroll menu.

      4. In the bottom-most scroll menu, "Files of Type", select "Bitmap (*.bmp)" format.

      5. Click the "Save" button

   iii. Now the original drawing will appear and the cursor will be displayed as a small square (this signifies that the program is waiting for the user to select something)

      1. Select the drawing by clicking on a corner (outside the drawings extent) and dragging diagonally across the drawing to the opposite corner (outside the drawing's extent). It is important to select the entire drawing, which may require some zooming before you select the image.

      2. Then hit "Enter"

      3. The bitmap file will now appear at the location where the drawing was saved.

2. Now open EPANET and add the bitmap file as a backdrop:
    a. Once in EPANET, select the "View" dropdown from the options at the top of the screen
    b. Select the "Backdrop" option (which will have an arrow to more options)
    c. Select the "Load" option
        i. A dialogue box labeled "Open a Backdrop Map" will appear.
        ii. Search for the bitmap file saved from the AutoCAD drawing and select it by double-clicking.
        iii. The backdrop will appear in the EPANET drawing display.
            1. *Note: you may have to use the "zoom" and "pan" options of EPANET to correctly center the drawing.
            2. You can hide the backdrop by selecting the "View" dropdown, the "Backdrop" options, and then "Hide".
3. Inputting Nodes:
    a. There are three types of nodes available for use:
        i. Junctions: These can serve a few different purposes
            1. Connecting node: connect 2 or more pipes
            2. Dead-leg: can define a dead-end run of pipe
            3. Demand node: user will predefine a demand at the node
            4. Emitter node: an emitter value can be associated with the node for pressure dependent demand

ii. Reservoirs: These will typically serve as the water supply or water main for the system. They are defined by head, not pressure, so the conversion from psi to feet must be made first.

iii. Tanks: these are nodes with storage capacity. These are important for systems that may require house tanks for downfeed water distribution, or other storage applications. *These will not be covered in this primer.

Below is a screen capture of the main menu located at the top of the EPANET interface. Labeled are those options which allow junctions, reservoirs and tanks to be input into a network.

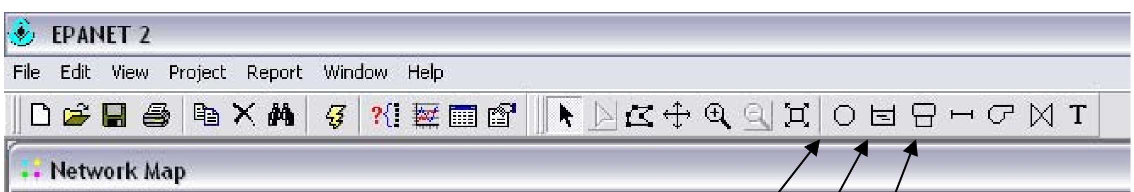


Figure A1.1: EPANET Main Menu (Node Selection)

Junctions:

Reservoirs:

Tanks:

Once a node is selected, the cursor will show a bullseye, and the node can then be placed into the EPANET display. The backdrop will aid in placing the node in its correct location with respect to the network layout.

4. Data Browser:

   a. The data browser is an interface in EPANET that allows the user to access the attributes of each object (Junctions, pipes, etc.) used for analysis. The data browser is typically placed at the right hand side of the display and is shown below:

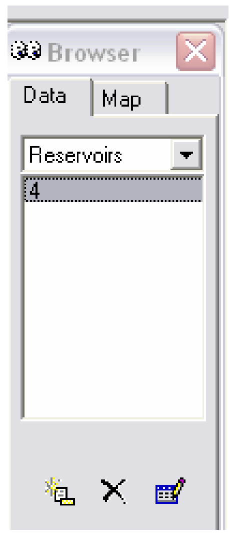


Figure A1.2: EPANET Data Browser

   b. Notice the dropdown (here "reservoir" is selected). All network objects can be chosen by clicking on the dropdown and selecting the desired option.

   c. When an object is added to a network, the date browser will automatically assign identification values (ID) in chronologic order. This ID number will appear in a list form in the Data Browser. In the above screen capture, there is a single reservoir which has an ID of "4". The ID of any object can be changed by double clicking on object in the network display. A dialogue will pop up (referred to as the "attribute window"), and the ID can be change by manually typing the desired value in the "ID" prompt line. For the reservoir referenced above, the attribute window will look like this:

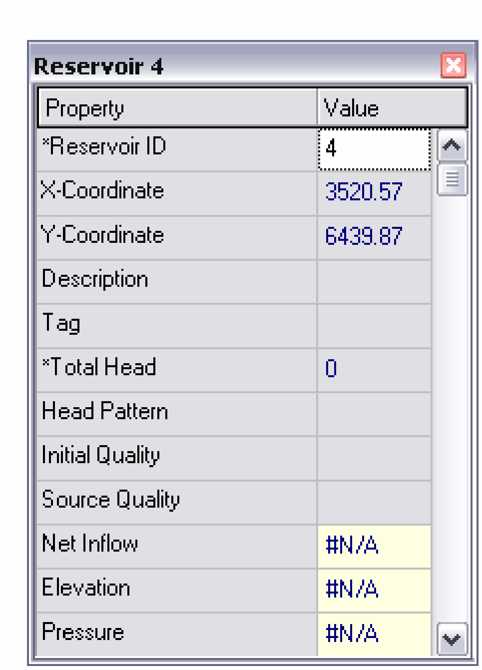


Figure A1.3: EPANET Attribute Window (Reservoir)

i. All other attributes associated with an object can also be modified by using the same technique (double-clicking the object and manually typing the desired value in the adjacent prompt line). Here, the total head in the reservoir can be changed by clicking on the "0" value adjacent to the "Total Head" line, and manually typing the correct head value.

5. Defining Node Attributes:

a. <u>Junctions</u>: As previously mentioned, junctions connect two or more pipes together. Junction elevations must be defined for EPANET to calculate the head loss due to elevation changes. This is accomplished by defining the “Elevation” value in the junction attribute window.

i. Note that you will have to select an elevation datum for the elevation values of the junctions to be meaningful. It is common to assign the street main an elevation of zero feet. Below is a typical junction attribute window:

ii. Emitters: A junction can act as a demand junction (water will be withdrawn from the junction) by placing an emitter coefficient value in the “Emitter Coeff” prompt line. As stated in the introduction, emitter coefficients can be roughly approximated for plumbing fixtures based on their required flow as stated by the International Plumbing Code. When defined, these values will allow a junction to be treated as a demand node dependent on the pressure drop over the fixture.

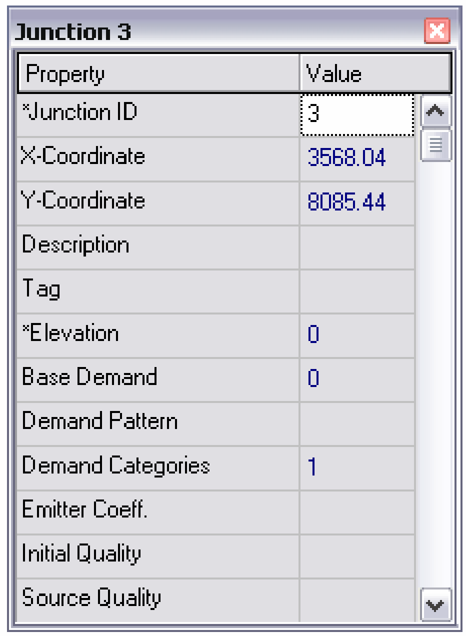


Figure A1.4: EPANET Attribute Window (Junction)

b. Reservoirs: Because plumbing systems are designed with respect to the minimum available pressure in the street main (Hunter's method), the reservoir should be assigned a head value that corresponds to this value. This is accomplished by multiplying the minimum main pressure by a conversion factor of 2.31. This leads to a head value in feet. The value is input in the line titled "Total Head".

   i. Figure A1.3 shows an attribute window for a reservoir.

6. Inputting Pipes:

   a. Once the nodes have been drawn and assigned elevation values, pipes can be added. The below window shows the pipe option in the EPANET interface:

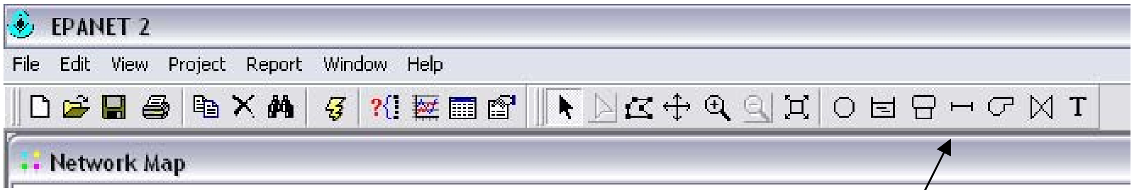


Figure A1.5: EPANET Main Menu (Pipe Selection)

Pipes:

   b. Pipes are drawn in by clicking on the downstream node and then the upstream node. Again, EPANET will automatically assign an ID value to each pipe.

7. Defining Pipe Attributes:
   a. The following is a list of attributes that must be defined for each pipe before running a hydraulic analysis:
      i. Diameter – inches
      ii. Length – feet
      iii. Roughness Coefficient – depends on the head loss equation being used (values can be found in the EPANET help index under “roughness coefficient”). Plumbing systems are typically modeled by the Hazen-Willaims equation:
         1. Darcy-Weisbach – “ε” (inches)
         2. Hazen-Williams – “C”
            a. To correctly run Hazen-Williams, the flow units must be set to GPM. This is accomplished by clicking on the “Project” dropdown and selecting “Analysis Options”. Click on the first prompt line

titled “Flow Units” and select “GPM” from the dropdown menu.

3. Mannings – “n”

iv. Loss Coefficient – this is the summation of all minor loss coefficients associated with a particular pipe. Minor losses are associated with flow appurtenances, such as elbow, tees, valves, etc. Values can be found in the EPANET help index under “Minor Loss” – “Coefficients”.

8. Special Equipment:

a. Often, manufacturers will state the pressure drop of a piece of plumbing equipment (water meter, water heater, water softener, etc.) in psi at a certain flow rate. EPANET does cannot directly work with such a loss. This problem can be solved by deriving a minor loss coefficient “K” from the minor loss equation given as:

$$h_L = K\frac{V^2}{2g} \qquad (2)$$

where, h_L is the head loss in feet, V is the pipe velocity and g is the gravitational constant. The head loss given by the manufacturer must be converted from psi to head (ft) and used for the value of h_L. The velocity should be taken as the maximum design velocity through the pipe, which is typically 8 fps. The corresponding K value then yields the largest feasible minor friction loss associated with the piece of equipment.

b. The derived K-values for each piece of equipment should be added to the minor loss values associated with their respective pipe sections.

9. Running a Model Simulation:

   a. Once all the above network characteristics have been entered, the system should be ready for analysis. Click the “Run” button on the main tool bar to run the analysis. Below shows the correct button:

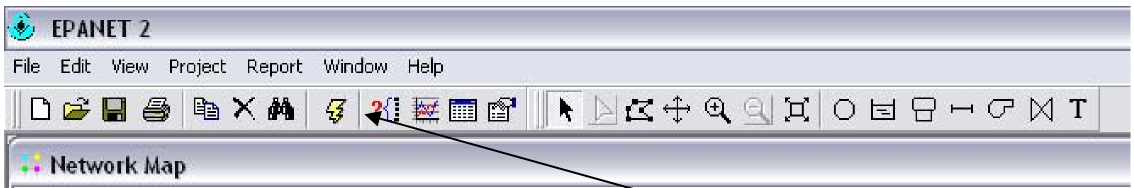


Figure A1.6: EPANET Main Menu (Run Simulation)

Run Option:

   b. EPANET will automatically pop-up a dialogue stating that run was successful. If the run with unsuccessful, it is necessary to retrace the above steps and check for errors.

   c. It is important to note that the simulation has been run under steady-state conditions for constant supply pressure.

10. Checking Results:

   a. After a successful run, the hydraulics results can be viewed in table format. Choose the “Table” option on the main menu. This options is shown below:

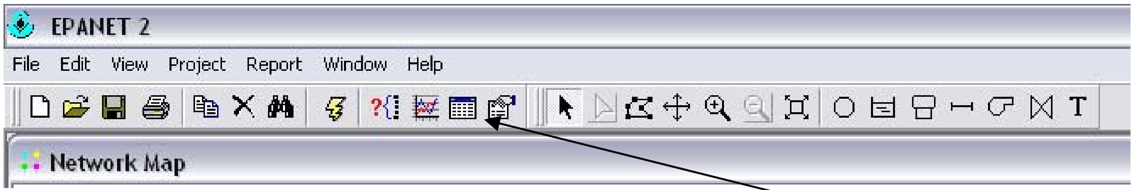
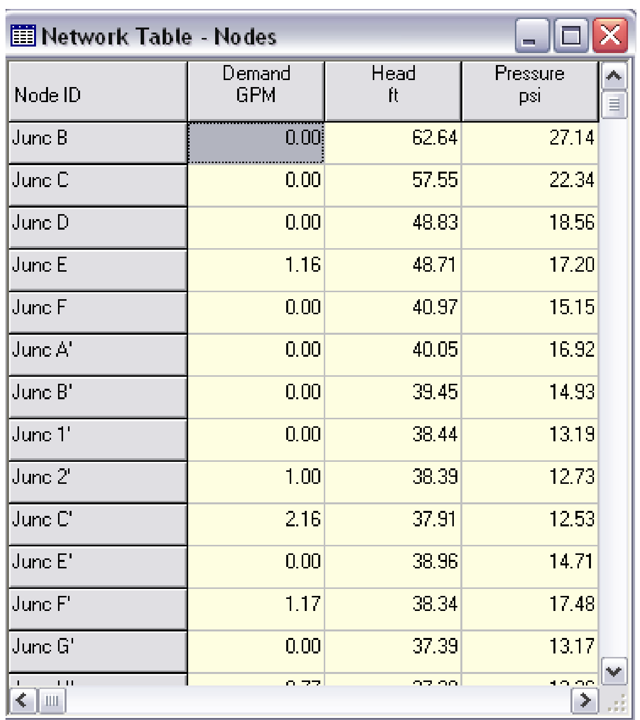


Figure A1.7: EPANET Main Menu (Results Table):

Table Option:

b. A dialogue will appear with the option for "Network Nodes" or "Network Links". The "nodes" option will yield head values, pressure values, and flow demands at each node. The "links" option produces values for flow, velocity, head loss, and friction factor in each pipe. A sample of each is shown below:



Network Table - Nodes

Node ID	Demand GPM	Head ft	Pressure psi
Junc B	0.00	62.64	27.14
Junc C	0.00	57.55	22.34
Junc D	0.00	48.83	18.56
Junc E	1.16	48.71	17.20
Junc F	0.00	40.97	15.15
Junc A'	0.00	40.05	16.92
Junc B'	0.00	39.45	14.93
Junc 1'	0.00	38.44	13.19
Junc 2'	1.00	38.39	12.73
Junc C'	2.16	37.91	12.53
Junc E'	0.00	38.96	14.71
Junc F'	1.17	38.34	17.48
Junc G'	0.00	37.39	13.17

Figure A1.8: EPANET Node Output Table

Network Table - Links

Link ID	Flow GPM	Velocity fps	Unit Headloss ft/Kft	Friction Factor
Pipe BC	22.15	8.61	849.16	0.063
Pipe CD	22.15	8.61	512.64	0.038
Pipe DE	1.16	1.60	42.84	0.049
Pipe DF	20.99	8.16	462.81	0.038
Pipe FH	12.81	4.98	1051.12	0.233
Pipe HI	5.31	3.52	129.64	0.044
Pipe IJ	4.14	3.79	126.37	0.031
Pipe IK	1.17	1.60	36.55	0.042
Pipe HL	7.50	2.92	78.73	0.051
Pipe L1	2.08	1.91	89.69	0.088
Pipe 12	1.29	1.78	65.64	0.061
Pipe 1M	0.79	1.09	27.61	0.068
Pipe LN	5.41	2.10	42.25	0.052
Pipe N3	2.72	2.49	212.05	0.123

Figure A1.9: EPANET Pipe Output Table