PLANNING AND DESIGN PARAMETERS-DWS

DESIGN PRINCIPLES

Design Rationale:

A proposed water system has a specific amount of gravitational energy, determined by the relative elevation of points in the system. Normally, it consist of the following components: source intake, raw water transmission pipeline, treatment plant, water tank and water distribution network. Since all points in a system are immovably fixed (i.e. buried in the ground), their relative elevation cannot change. *Thus for any system, there is a fixed, specific quantity of gravitational energy available to move water.*

As water flows through the pipes and fittings, some energy is lost forever, dissipated or burned off by friction. Thus, the amount of energy at any point in the system varies according to elevation and frictional losses. Improper design may result in some points in the system having inadequate energy (low pressure), while other points may have an excessive amount of energy (high pressure).

Design Purpose:

It is the responsibility of the system designer to properly manipulate pipe sizes and thus frictional energy losses in order to move the desired flows through the system. This is accomplished by:

- Conserving energy at some points; and
- Dissipating energy or burning it off (by friction) at other points.

Design Economic Fact:

In most water supply systems, more than 50% and usually up to 75% of the total cost lies in the distribution system. It is incumbent upon the designer to plan a system that efficiently delivers clean water to the user for the least cost. Proper system design is done by careful selection of pipe sizes, efficient pipeline routing and strategic location of control valves, water tanks, and tapstands.

Design Synopsis:

There are two common approaches in the design of a new system:

(1) an iterative process, where pipe sizes are the initial parameter (assumed), and after calculating the resulting residual pressures, pipe sizes are adjusted; and
(2) the conventional process, where flow rates are the initial parameter, and calculation of required residual pressures determines pipe sizes.

PART I –PLANNING AND DESIGN PARAMETERS, CRITERIA AND SYSTEM COMPONENTS

HYDRAULICS

1.0 HEAD PRESSURE AND ENERGY

Because of the earth's gravity, fresh water weights 1g/cm^3 on the earth's surface. The weight of a column of water above a certain point could be reported as pressure on that point. In hydraulic computations, it is easier to report water pressures as the equivalent height of water column. This height is referred to as *head*, and represents the amount of gravitational energy contained in the water.

2.0 FLUID STATICS AND DYNAMICS OR WATER AT REST AND WATER IN MOTION

2.1 Hydraulic Grade Line (HGL)

Under static (no-flow) conditions, the energy (pressure or head) at any point in a pipeline is directly related to the vertical distance from that point to the level of a free water surface (open to the atmosphere). Under dynamic (flowing) conditions, the energy at any point in a pipeline is characterized by the *hydraulic grade line* (HGL).

The HGL must always come to zero whenever the piped water comes into contact with the atmosphere. Because of frictional losses, the HGL always slopes downward along the direction of flow. Its steepness is directly related to the diameter of the pipe.

When the residual head of a pipeline full of water is zero at an outlet, then the maximum flow is moving through the pipe. This is the natural flow of the pipe, and is the absolute maximum flow that can be moved by gravity (See section 2.5 for discussion of residual heads).

A non-full flowing pipe is not under any pressure in Gravity-Flow Systems. (Except where the pipe flows full in U-profiles).

If the natural flows or carrying capacity of the pipe is greater than the safe yield of the source, then the pipe will drain faster than it can be filled, and the result will be a non-full flowing pipe. For uniformly descending pipes, the HGL will lie on the surface of the water inside the pipe. In an undulating pipeline profile, non-full flowing pipes will tend to produce air blocks at high points in the system.

2.2 Friction Head Conditions

Friction losses are caused by several factors:

- Length of the Pipe
- Velocity of the Water
- "Roughness" or Resistance to flow

Velocity of water flow in a pipe follows the continuity equation: $Q = VA$

Q = Flow rate or Discharge (volume per time)
V = Velocity (length per time)
A = Cross-sectional Area (length2 – pipe)

Friction losses (head losses) are approximately proportional to the square of the velocity. Thus the higher the rate of flow through any given pipe, the greater the friction losses.

Friction losses are also proportional to the length of pipe. Thus, the longer the pipe, the greater the friction losses for any given diameter of pipe and rate of flow.

The smaller the diameter of any given length of pipe, the greater the friction losses for any given rate of flow.

The rougher the inner surface of any given pipe, the greater the friction losses for any given rate of flow. Fittings on a pipeline also increase "roughness". The greater the number of fittings and valves on a pipelines, the greater the friction losses for any given rate of flow.

Friction losses are not affected by the angular orientation of the pipe (friction losses and rate of flow will be the same whether the water flows uphill or downhill).

Friction losses are not affected by pipe water pressures.

The total friction loss of fittings on a long pipeline is considered minor when the distance between the individual fittings is at least 1000 pipe diameters.

The total friction loss from fittings must be calculated when the distance between individual fittings is less than 1000 pipe diameters. Total friction loss in this case will be greater than the individual friction losses summed for each fitting.

Frictional headloss does not depend on arrangement of pipes. Losses are the same for pipes in parallel and pipes in series.

Frictional headloss factors have been tabularized for different pipe materials. The common method is to report the amount of frictional headloss per unit length of pipe. Thus, frictional headloss from fittings can also be expressed as an equivalent length of pipe.

2.3 Air blocks.

Trapped pockets of air in a pipeline are referred to as air-blocks and can interfere with flow. Air-blocks are caused when dry pipeline are refilled and air inside has no place to escape.

No air-block would take place in a system where a tank is located at an elevation lower than the air-blocks, as long as the air-blocks are at least 10 meters below the static water level of the source (See Figure 2.3). This provision may have adverse economic impacts and may not always be possible to follow.

Serious air-blocks will occur if the residual head at any point is less than 10 meters and if the flow in the pipe is fluctuating and if the pipe diameter is oversized.

Designer can minimize air-block problem by: (1) arranging pipe sizes to minimize frictional headloss between source and first air-block, and (2) using smaller diameter pipe at the bottom of critical air-blocks sections. The last recourse is to install an air-release valve.

2.4 Flow Rates and Pipeline Configuration

A pipe fed from both ends has a capacity equivalent to that of two pipes.

Doubling the diameter of pipe will not double the flow, but could quadruple the flow in the pipe ($Q \infty \text{diameter}^2$).

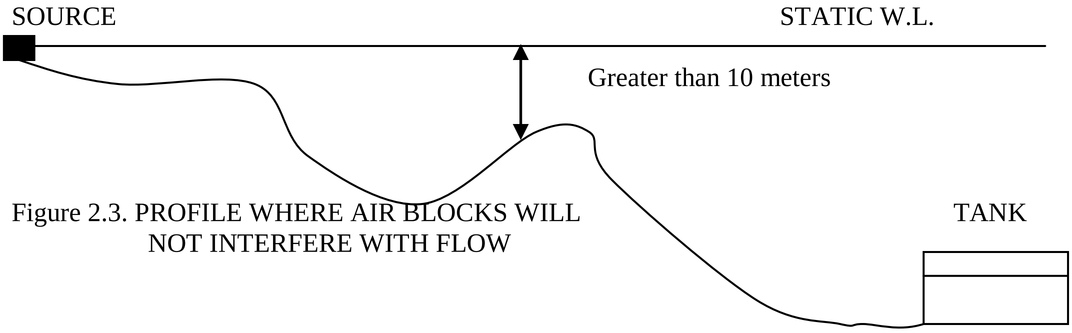
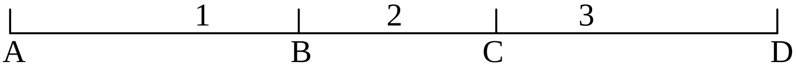


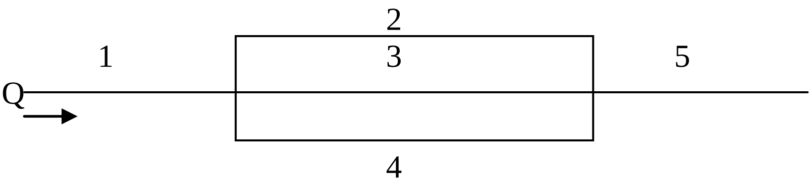
Figure 2.3. PROFILE WHERE AIR BLOCKS WILL NOT INTERFERE WITH FLOW

Pipes in series have equal discharges, but different heads.



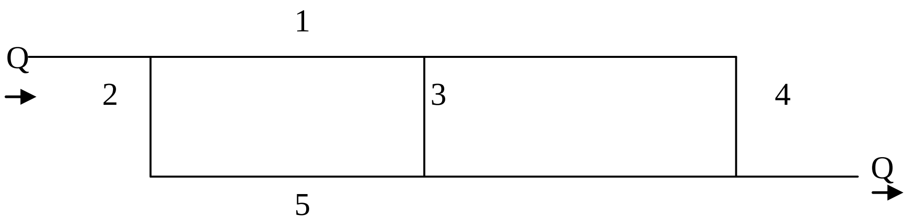
$Q = Q^1 = Q^2 = Q^3$ and $H = h^1 + h^2 + h^3$

Pipes in parallel have equal heads, but different discharges.



- $Q_1 = Q_5$
- $Q_1 = Q_2 + Q_3 + Q_4$
- $H = h_1 = h_2 = h_3 = h_4 = h_5$

Combinations of pipes in parallel and in series are more complex.



- $Q = Q1 + Q2$
- $Q1 = Q3 + Q4$
- $Q5 = Q2 + Q3$

$H = h1 + h4 = h2 + h5$
$h2 = h1 + h3$
$h4 = h3 = h5$

2.5 Design Pressures and Residual Heads

An absolute minimum static head or elevation difference of 20ft or 6m between water tank and service area is necessary for satisfactory gravity flow, even though less area may be covered.

System head (static head) available should be measured from the outlet (not the water level) of the source (spring box or reservoir) down to the point of use.

The available static head should not be fully consumed by friction headloss. There must be an amount held in reserve (residual head), to help prevent a vacuum from occurring in the transmission line.

Positive residual head indicates that there is an excess of gravitational energy or in other words there is enough energy to move an even greater flow through the pipeline. If allowed to discharge freely, a positive residual head will try to increase the flow through the pipes. As the flow increases, frictional headloss will increase until residual head is reduced to zero at the equilibrium flow rate.

The maximum ground elevation along the pipeline (especially transmission mains) must be considered in conjunction with the minimum required pressure/head at any point in the system.

For proper design, the HGL should be greater than or equal to 10 meters (5 meters absolute minimum) above the ground (or transmission pipeline) at all points in the system, except when unavoidable. Never allow the HGL to go underground. It would create suction which has 2 effects; (1) may induce contamination and/or (2) may cause pipe to collapse.

HGL may be plotted horizontally for extremely low flows in large pipes (where the headloss is less than 0.5 per 100m or pipe-length).

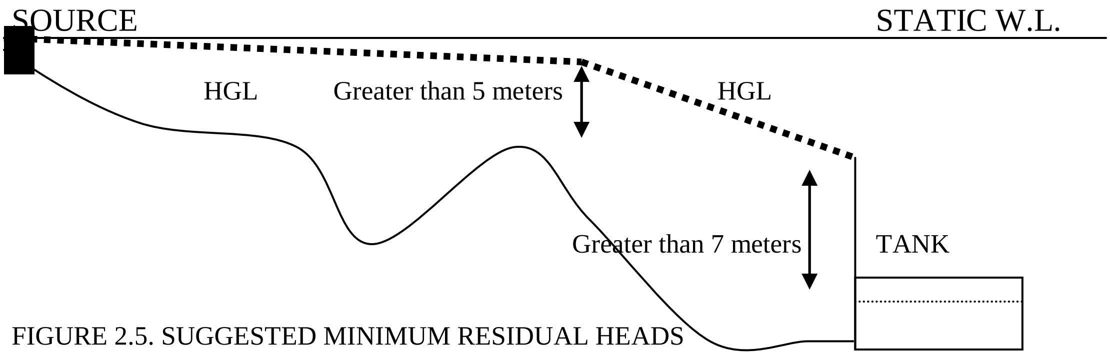


FIGURE 2.5. SUGGESTED MINIMUM RESIDUAL HEADS ALONG THE TRANSMISSION LINE

Standard Residual Heads:

At Tapstands		Discharge into tanks	
Absolute	: 7 m (urban)	absolute min	: 7 m
	: 3 m (rural)	ideal	: 10 m
Desired min	: 10 m	absolute max	: 50 m
Ideal	: 15 m		
Desired max	: 30 m		
Absolute max	: 56 m		

Minimum pressure at the remotest end of the system should be 3 m or approximately 4.26 psi (Refer Standard Residual Heads). Minimum pressure may also depend on local specifications (i.e. fire).

Note: GI pipe maximum pressure rating is 25 kg/cm2 (250 m of head). PVC maximum pressure rating is 150 m of head.

2.6 Allowable Velocities

Velocities are governed by the characteristics of the water carried and the magnitude of the hydraulic transients. If the velocity is too great, suspended particles can cause excessive erosion of the pipe; and if the velocity is too low, suspended particles may clog pipes by settling out in low areas of the pipeline. Sudden changes in velocity can cause water-hammer, which can also damage pipes.

Allowable Velocities

Min velocity	0.60 – 0.76 m/s
Max velocity	1.22 – 1.83 m/s (lower)
	3.00 – 6.00 m/s (upper)

Note: For design and investigation purposes, use: 0.60 – 1.22 m/s. Not really considered in rural design process.

Preferred Velocity of Flow in Pipes:

Main pipes	3.0 m/sec
Distribution pipes	1.5 m/sec
Distribution pipes (absolute min)	0.7 m/sec

3.0 PIPELINES

Pipe networks are used for conveying large amounts of water and can be seen in the same context as open channels or aqueducts. There are several different types of pipe.

Networks including:

- Single pipe (transmission mains);
- Branch network;
- Looped network;
- Combination of the above three.

In general, pipelines must be designed to handle the *maximum hour demand* of the area to be served.

3.1 PIPE SELECTION

For large supply networks, cast –iron pipes are the most common, with some steel and plastic pipe also used.

G.I. pipes are preferred in (1) open rocky terrain where trenching up to 0.3m deep is not possible, and (2) where traffic can be expected to cross over the pipeline.

G.I pipe should be used in all crossings:

- Crossings gullies,
- Crossings roads,
- Crossings Rivers by suspensions,
- Crossings streams.

The exposed lengths of pipe in a crossing should be sleeved or place inside a pipe of larger diameter.

Plastic pipes (P.E., P.B. and PVC) maybe used in easily trenchable upland soils (limestone, sand, gravel, clay, or shale) and in costal areas.

3.2 PIPE LAYOUT

Must generally follow the profile of the ground, but location also chosen with respect to construction cost and resulting pressures.

The most advantageous route for proposed pipelines will often follow existing or proposed roads. This practice of following quite direct routes will facilitate the protection and maintenance of the pipeline and minimize the acquisition of right-of-ways.

Pipe should be laid on a straight uniform grade as much as possible, since high spots create air-blocks, which may reduce or even stop flow.

The route for the transmission and distribution pipes should be designed and constructed away from excreta and waste water disposal systems to avoid possible cross connections.

3.3 PIPE LAYOUT WITH REGARD TO HGL

The closer the pipeline to the Hydraulic Grade Line (HGL), the lower the resulting pressures in the pipe, and in some cases, the lower the cost.

The pipelines may depart from the HGL by means of:

- Canals dug through the ground,
- Flumes elevated above the ground,
- Surface aqueducts laud in balance cut and fill,
- Graded tunnels penetrating hills.

Pipelines may depart from the HGL by means of:

- Pressure aqueducts laid in balanced cut and fill at the ground surface,
- Pressurized pipe from well sources with a pump to a reservoir and having no taps in between,
- Pressure tunnels dipping beneath valleys or hills,
- Pipelines following unavoidable topography (over hill and through dale), where sometimes the pipeline may even rise above the hydraulic grade line.

3.4 PIPE SIZES

Size and shape of supply conduits are determined by hydraulics as well as by structural and economic considerations.

Suggested minimum pipe diameters:

1 ¼" Ø	When grade is over 1.0%
1 ½" Ø	When grade is between 0.5% and 1.0%
2" Ø	when grade is less than 0.5%

For rural systems, 1"Ø pipe can be considered as the minimum for grades over 1.0%, grades less than 0.2% are not recommended (gravity low systems).

Ideally, the most economic design would select pipe diameters such that all of the head available the cause flow is consumed by friction, but because of system losses, a residual head is always built into design.

3.5 DISTRIBUTION PIPES, LAYOUT

The location of the smaller diameter service pipes in a distribution network is controlled by the location of the consumers.

For very wide streets, however, it is sometimes cheaper to stall a main behind the curb on each side of the street because of the savings in service pipes and to avoid digging of street pavement.

The system should be grid ironed with connecting pipes laid on the cross streets at intervals not exceeding about 200 meters whether or not there are consumers on the cross streets.

Dead end should be avoided in order to minimize troubles from corrosion and from organic growths. In rural systems, the linear layout at most barangays requires dead ends. In this case, the lowest end of the distribution pipe should have a clean-out.

3.6 LINEAR STRUCTURAL REQUIREMENTS AND ANCHORAGES

Structurally, pipes or other close conduits must resist a number of different forces singly or in combinations:

- Internal pressures equal to the full head of water to which the conduit can be subjected;
- Unbalance pressures at bends, contractions, and closure;
- Water hammer or increased internal pressures caused by the sudden reduction in the velocity of the water either by the rapid closing of a gate valve or shut down of a pump for example;
- External loads in the form of backfill, traffic and the weight of pipes themselves when supported by piers or hangers, etc;
- Temperature -induced expansion and contraction.

Anchorages must be provided:

- To resist the tendency of pipes to pull apart at bends and other points of unbalanced pressure when the resistance of their joint to longitudinal (shearing) stresses is exceeded;
- To resist the tendency of pipes laid on steep gradients to pull apart when the resistance of their joints to longitudinal (shearing) stress is exceed;
- To restrain or direct the expansion and contraction or rigidly jointed pipes under the influence of temperature changes.

Pipe anchors and trust blocks must be provided at all elbows, dead-end mains, tees reducers, crosses, and at long descending pipe runs at regular intervals. These structures are used to prevent movement of water main parts when subjected to changing pressures and forces.

3.7 PIPELAYING

Pipes as much as possible should be laid below the ground at least 0.30m deep to prevent damage for metal and 0.60 to 0.70 for plastic.

The trench line should be free of all sharp rocks that could cut or dent the pipe, especially when using plastic pipes.

In urban areas, the pipes are usually laid in the streets at some standardize position between curbs. In the Philippines, water distribution mains are usually located on the North or East side of the road. (The South and West sides of the road are normally allocated to the drainage systems).

All water pipelines are service connections should be located at least 8.0 meters or as far as possible from any source of pollution or contamination. Extra protection in the form of concrete encasement should be provided if it will be necessary to install the pipeline close to a source of contamination.

Pipes (G.I.) should be pushed or driven beneath paved roads or railroads. However, if pushing or driving does not work, bore a hole using an auger. Breaking the pavement should be done only when all other methods have failed. Used lengths of larger diameter pipes can be used as sleeve pipes.

3.8 INSTALLING PIPES ACROSS WATERWAYS

When installing pipes across waterways of less than 6.0 meters in width, place the pipe on concrete supports located on opposite banks and strap in place. Couplings or other fittings should be located on or near the supports and out mid-span. Where width of the water way is **more** than 6.0 meters ; place the pipeline on concrete supports constructed a short distance from banks and reinforce or suspend pipes with a steel cable set at the top of the concrete supports to prevent the pipe from sagging.

3.9 INSTALLING PIPES ACROSS CULVERTS (ROAD SHOULDERS)

Ideally, pipes can be installed **above** culverts on a road shoulders if there is adequate clearance between the **top** of the installed pipe and road surface (Clearance: Plastic Pipes – 0.4m; Metal Pipes – 0.2m). When clearance is less than the minimum pipeline. In most cases, however, it is acceptable to install the pipes below culverts with the required concrete covering or encasement.

3.10 INSTALLING PIPES ACROSS CULVERTS (ROAD PAVEMENTS)

When pipes crossing culverts must be installed under the main traffic area of the road, it is necessary to have increase clearances between the pipes and the road surface (Clearance: Plastic Pipes – 0.8m; Metal Pipes – 0.6m).

3.11 BACKFILLING

Backfilling the trench should be done as soon as the pipe has been laid. This practice is encouraged to minimize exposure of the pipe to sunlight and curios villagers, both of which can be detrimental to the pipe.

A specified length (section) at each joint in the pipeline should remain accessible. This section should be only half-buried until the pipeline has been filled with water and allowed to stand at full static pressure for 24 hours. This “hydro testing” makes it easy to locate leaks.

The backfill should be screened and compacted in layers and mounded over the trench line to compensate for eventual shrinkage.

4.0 FITTING AND VALVES

4.1 AIR -BLOCKS AND WASH –OUTS (FOR UNDULATING PIPELINES)

Tap stands or air-valves must be installed at every high spot in the transmission and distribution lines to release trapped air and /or permit air to enter pipeline.

Pressure release valves, blow-off valves, or washouts (made out of “Tees” with nipple and cap) must be installed at a every low spot in the pipeline to relieve or to reduce pressure on the pipes and/or for draining the pipeline during cleaning and maintenance.

Large supply mains should be provided with air valves at high points and blow-off valves at low points.

Clean-outs (out of “Y’s” with plug) should be installed at strategic points (bends and low spots) for maintenance.

4.2 CONTROL VALVES

Gate valves should be located so that areas of the pipeline can be isolated in case of breakage or repair work (exclusive of arteries).

There should be no long lengths of pipeline without a control valve. For rural areas with small diameter pipes (6”Ǿ or less), no more than 500m=100m of pipe length should be left without the control valve. For urban areas, large supply mains (greater 6”Ǿ) should be gated once every 1to 1.5 kilometers.

In high value districts, 150m should be the maximum distanced between control valves on feeder pipe systems. Arteries or feeders should be gated so that not more than 1/3 to1/2km within the system will be affected by a break.

Control valves should be placed on all branches from feeders to mains and between feeder pipes and hydrants.

Control valves should be located at street intersections for easy access in case of pipe breakage. At intersections of large pipes, a control valve in each branch is desirable.

Ordinarily, no more than three control valves are placed at a cross and no more than two at a tee.

When there is a choice, the control valve should be installed in the smaller of two pipes at an intersection to save on cost.

A gate valve should be placed at water supply components (tank, spring box, pumping station) and at the point of use (taps). Pipe outlets that otherwise will flow freely must have a control valve at a discharge point. There should be a control valve (globe, stop cock, service cock or reduce pipe) at every tapstand, since most tapstands require some reduction of the residual head.

4.3 FITTINGS AND OTHER APPURTENANCES

Unions (preferably ground unions) or flanges (for larger pipe diameters) must be installed every 10 lengths (or to local requirements) of a pipe run. This practice provides for convenience in replacing a length of pipe at any point in the pipeline without dismantling the preceding and succeeding pipes.

Unions should be placed after (in the direction of flow) a valve, at every pipe run change in direction, at intersections, and at both sides of a crossing (rivers, roads etc.)

Cross tees should be avoided because of difficulty in dividing flows (need globe valves) and difficulty in replacing.

A check valve should be placed on the upstream side at the beginning of each rise in the pipeline to prevent reversal of flow or dangerous water hammering. (For fully pressurized pipelines or directly pumped systems.)

Expansion joints (mechanical sleeve coupling) may be required on long lines, particularly on steel pipes exposed to marked changes of temperature.

5.0 WATER TANKS

5.1 DESIGN PURPOSE

In small distribution systems (gravity flow or pumped), it is always desirable to provide a distribution tank. The main reasons are:

- To satisfy hourly variations in the consumption rate (in small systems, such variations maybe three times the average hourly consumption and sometimes more);
- To equalize pressure throughout the distribution system (pumped system);
- To reduce or break excessive pressures;

- To provide for repair of pipes between the source and the tank, without the interruption of service;
- To provide extra storage for fire protection;
- To conserve flow during low demand periods (night);
- To serve as surge tank;
- To serve as a point for system disinfection.

5.2 ADVANTAGES

Other advantages which, under certain circumstances, may assume considerable economic importance include the following:

- Where water is pumped to the tank, pumps can be operated uniformly throughout the day. Uniform pumping allows for selection of smaller capacity pumps, thus reducing costs.
- The size of the transmission pipe between the supply source and the tank can be made smaller than would be necessary if the village were fed directly from the water source.

5.3 NECESSITY

A water tank used as storage is required when:

- The safe yield of the source will not directly provide 0.225 lps for each tap (or an absolute minimum of 0.125 lps per tap);
- The daily water demand is greater than the yield of the source during the daylight hours;
- The pipeline distance from source to village is so far that it is more economical to use a smaller pipe size (transmission) and build a water storage tank near the service area.

5.4 CAPACITY

The first consideration when designing a water tank is the storage volume or capacity. Tank capacity depends, to a great extent, on the type of supply, and is influenced by two main factors: (1) necessity for supplying water during peak demand periods, and (2) necessity for providing uninterrupted flow (reserve) during normal maintenance interruption or breakdowns.

As a rule of thumb, tank should be sized to hold ½ **of the average daily use (ADU)** of water or, economy, a minimum of 1/3 of the ADU (see design process).

Tank should be sized for anticipated future demand. If this is not possible, then increase the size of the distribution pipeline (increase pipe carrying capacity) to compensate for eventual consumption increases.

5.5 LOCATION

Water tanks should be generally be located near or in the center of the service area (for elevate tanks) in order to provide sufficient same constant water pressure, or on a nearby high point in order to minimize distribution pipe costs.

6.0 SERVICE LINES (LEVEL II/III)

It is common practice in water works design to allow only one customer on a service pipe for level III systems should have only one tapstand per service pipe (see section 7.0).

6.1 SPECIFICATION

For dwellings and similar buildings the minimum desirable size of service pipes is ¾ "Ǿ, (½ "Ǿ for rural). Service pipe material include: iron steel, brass of varying copper content and plastic pipes such as PVC, PE, and PB.

In metered systems, the larger the diameter of the service pipe, the higher the water rate.

6.2 TAPPING

There are several recommended tapping methods for service lines. One method employs reducing tees during pipe installation. Another method involves drilling a hole in the distribution pipe which is then threaded for connection of a corporation cock. The third method involves drilling a hole in the distribution pipe and fitting a pre-fabricated (or special order) saddle clamp over the pipe to established the service connection.

7.0 TAPSTANDS (stand pipes) (LEVEL II- COMMUNAL FAUCET)

It is common practice in water works design to allow about 5-7 households for level II systems. The number of tapstands will depend upon:

- Geographical lay-out of the service area (centricity). Tapstand should be 25-100 meters away from users;
- Source yield;
- Number of consumers; and
- Consumption requirements.

7.1 RESIDUAL HEAD AT TAPSTAND

If residual head is too high, it will cause accelerated erosion of the interior of control valves, (globes, cocks and if too low, will result in low flows.

The static pressure when the tap is closed must not exceed the pressure rating of the tapstand fixtures and tap line.

When a tapstand will be serving few households, the flow can be reduced, and conversely the flow maybe increased for a more densely populated area by installing a double – or triple-faucet tapstand.

Regulating pressure and flow is done by installing a 1/2" Ǿ globe valve (or service cock) at the base of the tapstand and adjusting it when all taps are open until the desired flow is delivered, this valve should then be securely lock up.

7.2 LOCATION AND SIZE OF TAPSTAND

Selecting sights for tapstands is a process of compromise. The designer and the community must consider the political, cultural, social; and economic situation – no single point is likely to meet all of the requirements.

The area and its immediate surrounding must be carefully selected and attractive.

The tapstand apron area must allow room for washing clothes bathing (if these practices are too permitted) as well as collecting water. The tapstand should be sized to accommodate the several users at once.

Important government buildings (school, market, barangay hall) should have convenient access to one of the taps.

Service pipes should be sloped ***away*** from the tapstand to prevent settlement of silt at the bottom of tapstands. Otherwise, a cleanout must be provided at the bottom of tapstands.

Tapstand should be located to serve those households that will depend upon it, usually at the central point of a household cluster.

Tapstand should be located near, but not directly on a main trail. A sunny site will discourage prolonged used and thus create less wastage.

Tapstand should not be located under palm trees in order to prevent damage to tapstand or user from falling fronds or coconuts.

7.3 DRAINAGE AT TAPSTAND

Tapstand should be designed to provide adequate drainage during peak use periods to prevent unsanitary conditions (standing water).

A small water-hole to collect waste-water for animals maybe located near the tapstand, but should be at least 10m from the pipelines and tap. Over flow from the water-hole can be channeled to a nearby garden, field or soak pit.

7.4 OTHER TAPSTAND DESIGN CONSIDERATION

There must be no tapstand located along the U-profile section of the pipeline to prevent reduction in quantity and velocity of flow of succeeding taps.

A single communal faucet should serve a cluster of 5 to 7 households, 10 at most.

Ideally, a faucet should not be more than 25m from the farthest house, but this distance can be increased to a maximum of 100m where necessary.

Standard tapstands flow = 0.2254 lps (13.5 1/min) or a minimum flow of 0.125 lps. The economic design flow is 0.2 lps which will fill 5 gallon container in less than 2 minuets.

Height of the tapstands ranges from 0.50m to 1.50m based on user preference. The faucet should protrude far enough from its vertical stand (0.10-0.30m) so that water vessels can be easily filled.

8.0 WATER METERS

The bigger the service pipe diameter the bigger the diameter of the water meter. It is common practice, however, to use a smaller diameter water meter with a reducer to minimize costs. Water meter should be required on all level III systems regardless of water source and system layout.

8.1 SELECTION

In the Philippines, certain water meter brands (Arad, Kent, Preciflow) have passed test assuring accurate meter readings. Substandard meters are available at lower costs, but this meter may give readings which either underestimate water usage.

8.2 INSTALLATION

Water meters should be installed so as to be tamperproof. This is accomplished with factory-installed seals, installation seals and lockable housings. Water meters should be installed near property lines and above ground for easy access. Water meters should always be covered to prevent algae build-up, sun damage and damage to glass facing.

9.0 PIPELINE DESIGN-BASIC CONSIDERATIONS

9.1 DESIGN FLOW

To be economical, pipelines should be designed using the **smallest** possible diameter pipes that will provide adequate flows to consumers over the design period.

DESIGN FLOWS CAN BE OVERESTIMATED BY:

- Overestimating the per capita consumption;
- Using too large a peaking factor;
- Overestimating the populating growth rate; and
- Using too lengthy a design period.

In the case of the spring source with variable yield, the design flow for the transmission pipeline will depend on:

- The duration of the extreme values (max/min flows)
- The demand requirement.

9.2 DESIGNING PIPE DIAMETERS

If pipe material, pipe length, design flow, topography and allowable pressure values are known, pipe diameter can be calculated using he Hazen-Williams equation or head loss tables or monographs (see annexes).

9.3 BRANCHED NETWORKS

The design of branch networks basically follows the same procedures as for the design of a straight pipeline.

Determine the HGL of each path and choose the smallest diameter pipe which satisfies residual head requirements. Later, a check is made for each demand node to verify if pressures satisfy the minimum allowable.

In practice, networks are often of the combined type. A single pipe is used as a transmission main between source intake and treatment plant or storage tank. In the center of the distribution network, the pipeline can be looped while in less densely populated service areas, a branched network is normally most economical.

9.4 DESIGN ERRORS

During the design phase of a distribution network many errors can be made which will lead to a costlier and sometimes failing system.

COMMON DESIGN ERRORS

- The selected water supply service standards are too high. Many of the design parameters like peaking factor, per capita use and growth rate are only estimates (based on experience). Over-estimating these parameters to be conservative is not always justified because of the economic impact;
- The network capacity is too large (cause by a too long design period or a too high peaking factor);
- The number of worked alternatives is too few;
- Inappropriate choice of pipe materials (eg. PVC or PE vs. GI);
- Pipe length is not minimized;
- The source design flow is too high. There is a tendency to try to capture and transport 100% of spring flow regardless of demand and/or environmental impact;
- The service area is not well defined, leading to overly costly connection at boundaries;
- A socio-economic survey which helps the planning of future community expansion is not properly done.

Many system designers fail to consider all possible options for most economical system design. The reasons for considering too few alternatives include:

- Lack of imagination and experience in exploring options;
- Risk avoidance in trying something new;
- Unwilling to do extra work, especially lengthy calculations are required;
- No supports from senior staff or from organizations who make strict and conventional design standards are not applicable in all situations.

PART II – THE DESIGN PROCESS

Proper water system design considers not only the technical and economic factors, but the social and cultural factors as well. This design process will not address looped system, but rather “dead end” type of system design suitable for most rural barangays.

1.0 GENERAL SYSTEM DESIGN

1.1 *General* system design (spring box or pump, water tank, and pipeline layout)

1.2 Type of distribution facilities (level II, III combinations)

2.0 DESIGN POPULATION

The design population is used to determine water demand and thus required storage capacity.

2.1 PRESENT POPULATION

Information is available from local statistics office NEDA, DOH, POPCOM, NSO. Also consider possible water collection by people from neighboring barangays that are not part of the originally identified community. Note: average Filipino family size is 6.

2.2 PROJECT LIFE (N)

The design period for water supply projects can be 5, 10, 15, or 20 years. Rural systems are usually based on a 5 to 15 years design period, with 10 years commonly used. Where long-range water demands cannot be accurately forecasted, it is advisable to consider a shorter design period.

- Design life of G.I. pipe is 20 years, but under ideal conditions, could range up to 50 years.
- The longer the project life, the **more expensive** the system will be.

2.3 POPULATION GROWTH RATE FACTOR (F)

Growth Rate (% per year)	Project Life			
	5 years	**10** years	15 years	20 years
1	1.05	1.10	1.16	1.23
2	1.10	1.22	1.35	1.49
3	1.16	1.34	1.56	1.81
4	1.22	1.48	1.80	2.19
5	1.28	1.63	2.08	2..65

Growth rate statistics are available from NEDA, DOH, POPCOM, local statistic office. If growth rate cannot be determined, a general figure of 2% per year can be used. However, the Philippines have a growth rate of between 2.1% -2.3% (closer to 3% in some rural areas).

The Population Growth Rate Factor (F) can be calculated by the formula:

$F = (l + gr)^n$

Where, gr = growth rate (%)
n = number of years (Project Life)

2.4 Design Population (DP)

DP = Growth Rate Factor (F) x Present Population (P)

3.0 PER CAPITA USAGE (RURAL)

Facility	**Daily Demand Liters per capita per day (lpcd)**	
	Range	Average
School	7-20	10
Barangay: Level II Level III	35-50 45-70	40 60
Poblacion: Level II Level III	65-80 75-100	70 80

Note: Figures may vary from place to place due to cultural factors, local customs, educational attainment, living standard and accessibility. Realistic (local) figures should be based on a water use consumption survey. The above figures are based on Philippine studies and include all domestic uses (drinking, washing, bathing and system wastage).

4.0 AVERAGE SOURCE FLOW (ASF) OR YIELD

For spring water sources the average source flow used in design calculations is based on minimum and maximum flows as well as the duration of the minimum and maximum flows over a 5 year period. For well sources, determine the suitable well yield for continuous pumping.

5.0 AVERAGE DAILY USE (ADU, Liters per Day)

ADU = Design Population x Per Capita Usage

Note: Calculated as lpd or m^3/ day), $1m^3$ = 1,000 liters

6.0 AVERAGE DAILY FLOW (ADF, liters per second)

ADF = Average Daily Use (lpd) ÷ 86,400 sec/day

Note: Used in the design of **STORAGE CAPACITY** for systems with spring and/or well sources.

7.0 MAXIMUM DAILY USE (MDU, liters per day)

7.1 FOR SPRING SOURCES:

MDU = Average Design Flow of Spring (lpd or m^3/ day)

There is no need to calculate MDU if it is economical to transport 80-100% of spring flow to storage, (Q, lps) is then the design flow for transmission pipeline. Go to step 9.0.

If it is uneconomical to transport 100% of spring flow to storage facility, used MDU or appropriate percentage of spring flow as design flow. Go to step 8.0.

Note: For small springs ($\leq$ 1.0 lps), 80-100% of the spring flow (Q) should be transported to the water tank. For larger springs, 50-80% of springs flow consider the environmental impacts and economic constraints for both cases.

If spring flow (Q) is variable, used average flow rate. In cases where maximum or minimum flow occurs for very short periods, use flow rate that is characteristic for most of the year (more than 6 months).

7.2 FOR WELL SOURCES:

MDU = Average Daily Use x 1.3 (liters/day or m^3/day)

Note: MDU is the largest one-day water demand. Normally, this occurs during the dry season and generally on a Monday.

1.3 is a design constant obtained from project studies conducted in Philippine rural areas to accommodate increased dry season demand.

8.0 MAXIMUM DAILY FLOW (MDF, liters per second)

MDF = Maximum Daily Use (MDU) ÷ 86,400 sec/day

Notes: MDU is used in the design of **TRANSMISSION PIPELINES** (Step 14.0) for both spring and well sources.

In the cases of piped-water systems with a well as the source, the well yield must satisfy the MDU, otherwise either an additional well source must be constructed or a water tank must be provided (depends on cost and convenience).

MDU is used in determining the minimum pump capacity. (Except in Hydro-pneumatic Pressure System).

9.0 ECONOMICAL STORAGE CAPACITY

9.1 METHOD I (simple Ratio)

Calculate the ratio (R) of the average source flow (ASF) (for spring sources) over the Average Daily Flow (ADF) and refer the table below.

R = ASF (from 4.0) ÷ ADF (from 6.0)

Ratio	Storage Capacity
< 1.0	inadequate source
1.0-2.0	½ of the average daily used
2.0-3.0	¼ of the average daily used
3.0-4.0	1/8 of the average daily used
> 4.0	no storage facility required

Note: In many field cases, the ratio falls between 1 and 2 and storage capacity can often be fixed at ½ the ADU. Such figures are approximations based on field experienced and may be adjusted to fit specific, local circumstances;

For economic reasons, a storage capacity of 1/3 the ADU is often selected, but based on observe daily demand patterns (when people fetch water), simultaneous usage generally occurs during meal times or for about 20-25% of the day, thus the storage capacity can be ¼ of the ADU.

If ratio is <1, then determine storage capacity using mass curve method;

Tank size should be **no greater** than what could be filled in a 12 hour period;

In general practice, a tank should always be provided (refer to part 1, section 5.0) refer to Part 1 section 2.5 for minimum and maximum head criteria at inlet pipe of tank.

9.2 METHOD II (Mass Curve)

Make a graph with the cumulative flow on the vertical axis and time (24 hours) on the horizontal axis. Plot the cumulative supply (cumulative spring flow rate or pumping rate) over a 24 hour period (taking into account hourly demand variations). “Slide” the cumulative supply line vertically until it is tangent to the **highest** point on the cumulative demand curve and redraw the line. Again “slide” the cumulative supply line vertically until its tangent to the **lowest** point on the cumulative demand curve and redraw the line. The vertical distance between those two “slide” lines is the required storage volume. Refer to the **Water for the World** technical note on “Determining the Need for Water Storage”.

10.0 AVERAGE HOURLY USE (AHU) OR PEAK DEMAND (PD) – Level III Systems

This demand is produced when all the distribution points or taps are fully open, and accounts for 20-25% of the operating time (4-6 hrs. per day), usually around meal times. Thus, instead of water demand being spread evenly over 24 hour period, it is concentrated within 4-6 hrs. each day. Based on economic considerations, it is often impractical to size distribution pipes based on all taps open simultaneously, so we use a peaking factor instead. These factors should be used with metered taps. Adjust factors upwards for unmetered situations.

Peak Demand (PD) for Level II systems is based on actual number of taps and is calculated in step 11.0. For Level III systems and combination systems (Level II and III), PD may have to be adjusted if the peak flow based on number of taps exceeds the PD calculated using the following factors:

10.1 Urban Situation (Municipalities and Cities), Level III

AHU = 2.0x ADF (liters per second)

10.2 Rural Situation (Barangays and Sitios), Level III

AHU = 3.0 x ADF (liters per Second)

Note: Used to design **<u>Distribution Pipeline</u>** (Step 15.0).Peaking factors are based on limited Philippine field data. Also used to estimate **Minimum capacity** in Hydro pneumatic System.

11.0 NUMBER OF TAPSTANDS (or standpipes), Level II

Used a ½ "Ǿ service pipe in level II (Communal Faucet) water systems. Number of taps can be determined by: (1) code requirements, (2) code provisions, or (3) calculation of actual consumption. For small rural systems, code provisions are often most appropriate.

11.1 Code Requirements

(1) Desired Maximum flow per tap = 0.225 lps

(2) Desired Minimum flow per tap = 0.125 lps

For design purposes, an <u>ideal design flow per tap of 0.20 lps</u> is preferred. This flow allows users to fill a #5 plastic container (20 liters) in less than 2 minuets and allows for expansion (additional taps) without violating the code minimum flow per tap (2).

Number of Taps = Peak Demand ÷ desired flow per tap

11.2 CODE PROVISIONS (PREFERRED)

(1) Desired HH per tap = 5-7 (10 HH maximum).

Number of taps = Total # HH ÷ desired # HH per tap

Note: Users may be grouped based on the rustling **SPOT MAP** of the service area or by preference of the users (See Figure 12.0).

11.3 ACTUAL CONSUMPTION

Actual flow required to meet daily demand **per tapstand.** In other words calculate the average daily flow on a per tapstand basis (see steps 2.0-6.0). This approach is only used for sources which **cannot** supply the desired minimum flow per tap (0.125lps).

Projected Tap Flow = Projected # of Users x Per Capita Demand ÷ 86,400

Number of taps = Peak Demand ÷ Projected Tap Flow

Note: Design flows should always fall between the desired maximum and minimum flows. Refer to part I, Section 2.5 max/min tap flows.

11.4 Adjusted Peak Demand

<u>If the total desired tap flow from the number of taps calculated in step 11.2 is greater than the Peak Demand (AHU) calculated in step 10.0</u>, the design peak demand must be adjusted to reflect the **higher** value. This <u>value is used in designing the distribution pipeline</u>.

Adjusted PD = # of taps x flow per tap (0.2lps)

Peak Factor = ((6HH/TS)(6persons/HH)(1+gr)^n(40lps/cap/day))/86,400/0.2

11.5 CLUSTERING TAPSTANDS

Tapstands should be located based on an assessment on the political, social, and economic factors involved. Final tapstands location should be a community decision. The designer should make use of a community Spot Map to plan Level II tapstands locations. See figure 12.0.

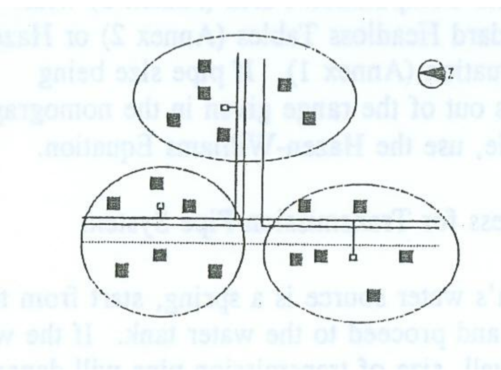


Figure 12.0 SPOT MAP.

Reminder: In the Philippines, water distribution mains are usually located on the North or East side of the road.

13.0 TANK PLACEMENT

13.1 General rules:

The nearer the storage tank to the service area: (1) the cheaper the cost (especially the distribution pipe network); (2) the easier it is to monitor (guard); and (3) the more convenient it is to operate and maintain.

The location of the storage tanks depends on:

(1) The design layout of the system.

(2) The system hydraulics or available heads.

14.0 TRANSMISSION PIPELINE (From source to Tank)

Selecting pipe sizes is a process of trial and error. Design pipeline to carry MDF (calculated in step 8.0) or average design flow (springs). Ideally the transmission pipeline should have no more than 2 different pipe sizes. Use hydraulic Computations Form (annex 3) with NWRC standard Head loss Tables (Annex 2) or Hazen-Williams Equation (Annex 1). If pipe size being considered is out of the range given in the nomograph or head loss table, use the Hazen-Williams Equation.

14.1 DESIGN PROCESS FOR TRANSMISSION PIPE SYSTEM

If the system's water source is spring, start from the top (source) and proceed to the water tank. If the water source is a well, size of transmission pipe will depend on safe yield of the well which is determined after drilling. Design well yield also depends on calculated demand and the design process will generally proceed from the bottom to the top of the system.

The smallest diameter pipe, with a calculated headloss less than the available head, should be chosen for each continuous section of the pipeline.

15.0 DISTRIBUTION PIPE SYSTEM

From storage tank to service area, pipeline must carry <u>Peak Demand or Adjusted Peak Demand</u>.

Use Hydraulic Computation Form (Annex 3) with the NWRC Headloss Tables (Annex 2), Nomographs or Hazen-Williams Equation (Annex1).

15.1 DESIGN PROCESS FOR DISTRIBUTION PIPE SYSTEM

Start with flow from water tank and proceed to service area. System can also be designed by starting with required flow at farthest end of service area and working backwards to the water tank.

For level II systems, each succeeding reach of pipeline must carry a volume of flow equivalent to the flow required for succeeding # of taps. Level III systems must be designed based on the peak factor flow for each reach or by rationing the # number of remaining HH in a reach to the total # of HH to be served.

The smallest diameter pipe, with a calculated headloss less than the available head, should be chosen for each continuous section of the pipeline. See section 2.5 of Part I for allowable residual heads.

16.0 TECHNICAL PROPOSAL

Every Proposal should include the following technical components:

(1) Project General Data

(2) Computations (Hydraulic and Structural)

(3) System Components and Appurtenances

(4) Design Drawings

(5) Specifications

(6) Materials List

(7) Labor Requirements

(8) Schedule of Work

(9) Summary of Costs

Note: Final format will depend on funding agency formats.

ANNEX 1

1.0 FORMULAS FOR TRANSMISSION PIPE SIZING

Calculation of pipe diameter is based on empirical flow equations. Several different empirical equations have been derived, but results do not vary significantly. One commonly used flow equation is the Hazen-Williams equation.

1.1 FORMULA USING HAZEN-WILLIAMS (METRIC)

(1)
$$H_f = 6.78 \frac{L}{D \cdot 1.65} \cdot \left(\frac{V}{C}\right)^{1.85}$$

- or -

$$V = 0.355 \; C \cdot D^{0.63} \left(\frac{H}{L}\right)^{0.54}$$

(2)
$$H_f = \frac{10.597 \; L \cdot Q^{1.85}}{C^{1.85} \; D^{4.865}}$$

(3)
$$Q = 0.278 \; C \cdot D^{2.63} \left(\frac{H}{L}\right)^{0.54}$$

$$D = \left[\frac{Q}{0.278 \; C \left(\frac{H}{L}\right)^{0.54}}\right]^{\frac{1}{2.63}}$$

Where,

H_f = Head loss due to friction (m/100m)

V = Velocity (m/s) for design purpose use:

0.60- 1.22 m/s

Q = Flow rate (m/sec)

D = Diameter (*inside*) of pipe (m)

C = H-W coefficient (depends on pipe diameter & condition)

H = Head available or elevation difference (m)

L = Total pipe length or ground distance (m)

(4) $$Q = 0.0918 \ C \cdot D^{2.63} \left(\frac{H}{L}\right)^{0.54}$$

Where,

Q = Flow rate (m^3/sec)

D = Diameter (inside), (cm)

Note: This formula is sufficiently accurate for pipe sizes of 150 mm upwards and for values of C not substantially below 100. It is more accurate for larger diameter pipes and for pipe velocities on the order of 1m/s.

Pipe Material	Hazen-Williams "C"		
	New	Design	Aged
Plastics/Glass/Fibers	150	140	130
Copper or Brass	140	130	120
Asphalt, Mortar Lined	140	120	100
CI OR Ductile Iron			
Asbestos Cement	130	120	110
Steel or G.I	130	110	90
Concrete	130	110	100
Linen Fire House	110	090	080

*** For pipe diameters 75 mm (3") and larger.**

Accurate estimate from the Hazen-Williams Equation depend on the use of the actual inside diameter of the pipe (not just nominal diameter), and a judicious choice of the friction coefficient "C". The preceding table lists typical "C" values for various kinds of pipe. The H-W equation is intended for used only with cool water and pipes of 75mm (3") or larger diameter. It maybe "stretched" for smaller size by subtracting 5 from the "C" values for 50-63mm (2-2 ½") pipes and subtracting 10 for 25-38 mm (1-1 ½") pipes. Headloss estimates in smaller sizes should be considered very crude.

“New” values are an average of those reported by various manufacturers. “Design” values are representative of conditions over the first 5-10 years of service allowing for a certain amount of scale, slime, deposition, etc. for corrosive water; reduce “C” value further. “Aged” values area an estimate of typical conditions after 15-20 years of service.

(5) Hazen-Williams nomographs for C = 80-150

(6) Hazen-Williams tables (most common) for frictional losses based on C = 100 for new GI pipes and C = 150 for plastic pipes. In instances where desired pipe size is outside the range given in the table, use Equation (2) Section 1.1 to find corresponding headloss.

Note: The constants in the preceding formulas are directly related to the units used in the formula. Therefore when using these formulas, the units specified are of critical importance.

1.2 Energy Equation (Bernoulli Equation)

This equation applies only to steady water flow within a “Streamline” and to flow where no energy is lost through friction. In order to apply the same equation to flow within the pipe as a whole, the energy equations for each of the separate streamlines must be summed because the velocities are not all the same.

$$\frac{V_1^2}{2g} + \frac{P_1}{w} + Z_1 + H_p = \frac{V_2^2}{2g} + \frac{P_2}{w} + Z_2 + H_l$$

Where,

$\frac{P}{w}$ = Pressure head (ft or m)

$\frac{V^2}{2g}$ = Velocity head (ft or m)

Z = Elevation head (ft or m)

H_L = Head loss between points 1 and 2 (ft or m)

H_P = Pump head; only when energy is added by pump (ft or m)

1.3 Darcy - Weisbach (Full-flowing pipes, Metric)

(1) $h_f = \frac{f \cdot L\, V^2}{D\, 2g}$

(2) $h_f = \frac{0.0826\ f \cdot L\, Q^2}{D^5}$ *using flow rate, Q*

(3) $h_f = \frac{0.001653\ L \cdot Q^2}{D^5}$ *when* $f = 0.02$

Where,

f = Darcy’s coefficient (friction factor)

L = Length of pipe (m)

Where,

f =Darcy's coefficient (friction factor)

L =Length of pipe (m)

V =Velocity of flow (m/s)

D =Diameter (inside) of pipe (m)

***H*ی** = Headloss due to friction (m)

g = Gravitational constant, 9.81 m/sec/sec

Q =Flow rate (m^3/s)

Darcy-Weisbach (non-full flowing pipes. Metric)

(1) $$Q = \frac{1}{n} A \cdot R^{\frac{2}{3}} \left(\frac{H}{L}\right)^{\frac{1}{2}}$$

(2) $$V = \frac{0.59}{n} D^{\frac{2}{3}} \left(\frac{H}{L}\right)^{\frac{1}{2}} \qquad n = 0.011 - 0.013$$

Where:

Q = Flow rate (m^3/sec)
H = Elevation difference (m)
L = Total pipe length or ground distance (m)
A = Area (cross-sectional) of flow (m^2)
R = Radius (hydraulic, m) (Cross-sectional Area ÷ Wetted Perimeter)
D = Diameter (inside) of pipe (m)
n = Roughness coefficient, (0.010-0.015)

Mannings Formula (metric)

(1) $$h_f = \frac{6.35\ n^5\ L \cdot V^2}{D^{\frac{4}{3}}} \quad \text{where}$$

$$V = \frac{0.397}{n} d^{\frac{2}{3}} \left(\frac{H}{L}\right)^{\frac{1}{2}}$$

(2) $$h_f = \frac{10.29\ n^2\ L \cdot Q^2}{D^{\frac{16}{3}}}$$

$$h_f = \frac{0.001245\ \ L \cdot Q^2}{D^{\frac{16}{3}}} \quad when\ \ n = 0.011$$

$n =$ Roughness coefficient

Note: This equation is used for lengths of pipeline involving many fittings whose effect is appreciable and must be accounted for. This equation is more accurate than the Hazen-Williams (H-W) formula for estimating high flows, or flows in old, rough-surfaced pipes where the H-W "C" value is well below 100.

2.0 DISTRIBUTION PIPE SIZING – ALTERNATIVE METHODS

(1) Sectioning Method (looped systems)

(2) Circles Method (looped systems)

(3) Pipe equivalent Method (preliminary check, networks)

(4) Hardy Cross Method (looped systems)

(5) Computers (dead-end and looped systems)

(6) Computations (Headloss Tables for pipes and fittings)

ANNEX 2

Q	**PIPE SIZES (mm.)**									
LPS	**13**	**19**	**25**	**31**	**38**	**50**	**63**	**75**	**100**	**150**
0.01 0.02 0.03 0.04 0.05	0.20 0.80 1.60 2.80 4.20	0.22 0.38 0.60								
0.06 0.07 0.08 0.09 0.10	6.00 8.0 10.00 12.60 15.20	0.62 1.00 1.30 1.64 2.12	0.20 0.26 0.34 0.44 0.52	0.15 0.18						
0.11 0.12 0.14 0.15 0.16	18.20 21.40	2.35 3.00 4.00 4.20 5.00	0.62 0.72 0.96 1.10 1.24	0.22 0.26 0.34 0.36 0.44	0.13 0.15 0.16					
0.18 0.20 0.25 0.30 0.40		6.20 7.60 11.60	1.54 1.88 2.84 4.00 6.80	0.54 0.64 0.96 1.34 2.30	0.202 0.262 0.400 0.46 0.94	0.70 0.10 0.14 0.24				
0.50 0.60 0.70 0.80 1.00			10.20 14.40	3.48 4.80 6.40 8.20 12.60	1.42 2.00 2.66 3.40 5.20	0.36 0.50 0.66 0.84 1.28	0.12 0.17 0.22 0.28 0.42	0.70 0.91 0.117 0.177		
1.20 1.40 1.50 1.60 1.80				17.60	7.20 8.80 9.80 11.00 14.70	1.78 2.40 2.70 3.04 3.75	0.60 0.80 0.88 1.02 1.28	0.248 0.330 0.374 0.422 0.524	0.104 0.129	
2.00 2.50 3.00 3.50 4.00					16.80	4.60 7.00 9.90 13.90 18.40	1.54 2.40 3.30 4.38 5.00	0.640 0.96 1.36 1.80 2.30	0.157 0.238 0.332 0.442 0.368	
4.50 5.00 6.00						23.70	7.20 9.00 12.40	2.86 3.48 4.88	0.706 0.858 1.200	0.12 0.17
7.00 8.00 10.00							17.20	6.40 8.30 13.00	1.60 2.40 3.10	0.22 0.28 0.42

FRICTION HEAD LOSS IN METRES PER 100 METERS GALVANIZED IRON PIPE (GIP)

FRICTION HEAD LOSS IN G.I. PIPES

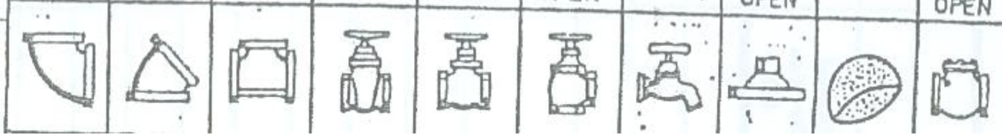
Q	PIPE SIZES (mm.)									
LPS	13	19	25	31	38	50	63	75	100	150
.01 .02 .03 .04 .05	0.1 0.4 0.8 1.4 2.1	0.11 0.19 0.30								
.06 .07 .08 .09 .10	3.0 4.0 5.0 6.3 7.6	0.41 0.50 0.65 0.82 1.06	0.10 0.13 0.17 0.22 0.26							
.11 .12 .14 .15 .16	9.1 10 .7	1.18 1.50 2.00 2.10 2.50	0.31 0.35 0.48 0.55 0.62	0.11 0.13 0.17 0.18 0.22						
.18 .20 .25 .30 .40		3.10 3.80 5.80	0.77 0.94 1.42 2.00 3.40	0.27 0.32 0.48 0.57 1.15	0.101 0.131 0.20 0.23 0.47	0.12				
.30 .60 .70 .80 1.00			5.10 7.20	1.74 2.40 3.20 4.10 6.30	0.71 1.00 1.33 1.70 2.50	0.18 0.25 0.33 0.42 0.64	0.11 0.14 0.21	0.089		
1.20 1.40 1.50 1.60 1.80				8.80	3.60 4.40 4.90 5.50 7.35	0.89 1.20 1.35 1.52 1.88	0.30 0.40 0.44 0.51 0.64	0.124 0.165 0.187 0.211 0.262		
2.00 2.50 3.00 3.50 4.00					8.40	2.30 3.50 4.95 6.95 9.20	0.77 1.20 1.65 2.19 3.00	0.320 0.48 0.58 0.90 1.15	0.079 0.119 0.155 0.221 0.184	
4.50 5.00 6.00						11.85	3. 60 4.50 6.20	1.43 1.74 2.44	0.353 0.429 0.60	0.06 0.09
7.00 8.00 10.00							8.60	3.20 4.15 6.50	0.80 1.20 1.55	0.11 0.14 0.21

FRICTION HEAD LOSS IN METERS PER 1OO METERS

PLASTIC PIPE- PVC, PE, PB C = 150

FRICTION HEAD LOSS IN PLASTIC PIPES

RESISTANCE OF VALVE AND FITTINGS*



EQUIVQLENT LENGTH STRAIGHT PIPE METERS

NOMINAL DIA. IN MILLI-METERS	90° ELBOW	45° ELBOW	STAND. T	GATE VALVE FULLY OPEN	GLOBE VALVE FULLY OPEN	ANGLE VALVE FULLY OPEN	FAUCET FULLY OPEN	FOOT VALVE FULLY OPEN	STRAINER	CHECK VALVE FULLY OPEN
13	0.55	0.24	1.04	0.11	4.88	2.56	4.88	1.22	3.05	1.16
19	0.69	0.30	1.37	0.14	6.40	3.66	6.40	1.52	3.66	1.58
25	0.84	0.41	`1.77	0.18	8.23	4.57		1.83	4.27	1.98
32	1.14	0.52	2.29	0.24	11.28	5.49		2.13	4.88	2.74
38	1.36	0.61	2.74	0.29	13.71	6.71		2.44	5.49	3.35
50	1.62	0.76	3.66	0.38	16.76	8.54		2.74	6.10	4.27
63	1.98	0.91	4.27	0.43	19.61	10.06		3.05	6.71	5.18
75	2.50	1.16	4.88	0.53	25.90	12.80		3.66	7.62	5.79
100	3.35	1.52	6.71	0.70	33.54	12.80		4.57	9.15	7.62
150	5.03	2.04	9.76	1.01	48.78	24.39		6.42	12.21	11.59

*WHEN THE LENGTH OF PIPE IS GREATER THAN 1000 TIMES ITS DIAMETER, THE LOSS OF HEAD DUE TO VALVES AND FITTING MAYBE DISREGARDED.

EQUIVALENT LENGTH OF PIPE FITTINGS, VALVES, ETC.

HYDRAULICS COMPUTATION FORM

Project: ______________________________ Prepared by: ______________________________

Location: ______________________________ Checked by: ______________________________

GIVEN					REQUIRED							
1	2	3	4	5	6	7	8	9	10	11	12	13
Reach	# HH	Design Flow, Q (lps)	Length, L (m)	Available Head, H (m)	Nominal Diameter Ǿ (mm)	Headloss Factor, H_f (per 100 m)	Headloss H_f x L +100 (m)	Residual Head, H_f (m)	HGL Pt. Elevation (m)	Ground Elevation (m)	Station	Remarks

ANNEX 3

Table 11-2 typical head losses (m) in service connections with water meters [a]

Flow 1pm	Service Connection Length (m)					
	0[b]	5	10	15	20	25
5 (0.083lps)	0.3	0.6	0.9	1.2	1.5	1.8
10 (0.162lps)	1.6	2.4	3.2	3.9	4.7	5.5
15 (0.25lps)	3.0	4.8	6.5	8.3	10.1	11.9
20 (0.314lps)	5.1	7.9	10.7	13.5	16.3	19.1

[a]Based on smooth ½ inch (13mm) pipe and 5.8-inch (16mm) meter.

[b]Source: Adapted from American Water works Association distribution Manual