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RESEARCH ARTICLE

A STUDY ON HYDRAULICS OF UNDERGROUND BURIED PIPE SYSTEM

[*,1]Radhakrishna, A. R., [2]Ravikumar, A. S. and [3] S. B. Dugad.

[1]Department of Agricultural Engineering, GKVK, UAS, Bangalore, Karnataka, India
[2]Department of Civil Engg., UVCE., Bangalore, Karnataka, India
[3]Senior Manager (Technical) and Staff of Jain Irrigation System, Jalagaon, Maharastra, India

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ABSTRACT

The design of pipe system for water supply through buried pipes and for efficient irrigation, it is necessary to know the hydraulics of the pipe systems. The hydraulics of the buried pipe distribution system consists of flow of water in the pipe, frictional head loss, other fitting losses, velocity, etc. The material selected for water distribution system is made of PVC pipes confirming to the Bureau of Indian Standard Specifications IS:4985, 1981 (as amended from time to time) or the standard prescribed by other national standard organization. The procedure, in the design consists of deciding the pipe diameter for different discharges and for different area of the plot. The design also consists of pressure tolerance of the pipe system. The long history of hydraulics of the pipe system is referred. The extension of reviewed paper is presented. In designing the pipe system, the hydraulics of pipes plays the major role in the contest of hydraulics.

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INTRODUCTION

The irrigation and distribution system conveys water to the fields from the source or from the point of supply. In most of the tank system the conveyance of water is in open channel system. In open channel system there are conveyance losses like evaporation, percolation loss etc, to avoid the losses an alternate technology of underground multiple outlet buried pipe system has been introduced. The main quantifiable benefits of buried pipe system over the open channel alternatives are: reduction in water transit and distribution losses. Reduction in the land area taken up by the distribution system. Reduction in the maintenance and operating costs of the irrigation system. Short transmit times effectively reduce the distance between the farmers and the water source, decreasing the magnitude and importance of tail-end problems and water stealing. Elimination of suitable habitants for disease like Malaria from stagnate water in the command area Campbell (1984) conclude that pipe system in Northern India assured flow delivery at the design discharge to the farthest irrigator with a minimum losses and unauthorized diversion enroute Economic benefits were generated by the higher level of agricultural development (growing of high value crop), which was made possible by the improved flow deliveries. Gisselquist (1989) similarly reports successful crop diversification on buried pipe distribution system in Bangladesh, benefiting from the system flexibility, To design multiple outlet pipe system it is very much necessary to know the hydraulics of buried pipe system Hence, The history of hydraulics of pipe distribution system consists of flow of water in the pipe, its frictional head loss, other fitting losses, velocity, etc are reviewed.

****Corresponding author:*** **Radhakrishna, A. R.**
Department of Agricultural Engineering, GKVK, UAS, Bangalore

Details of study area

The study area chosen for the present study was Chunchadenahalli tank, Kolar district. Kolar district is one of the drought affected districts in Karnataka and has more number of irrigation tanks (Raju et al., 2003). Chunchadenahalli tank is situated adjacent to Chunchadenahalli village, Vakkaleri hobli, Kolar taluk, Kolar district. The catchment and command area of the tank geographically lies between 13o 5' 40" and 13o 8' 15" N latitude and 78o 1' 30" and 78o 4' 5" E longitude. The tank is covered in Survey of India Toposheet No. 57 K/4 on 1:50,000 scale. Hydrogeologically the tank catchment area is situated in granite gneiss and it is moderately weathered. Large area of the catchment is hilly with red sandy loam soil and the depth of the soil varies from 7.5 to 22.5 cm. The Chunchadenahalli tank has an independent catchment area of 544 ha and water spread area of 12.5 ha. The command area contains 23.37 ha, which is owned by 75 farmers. The total length of the tank bund is

776.84 m. There were five main irrigation channels and ten sub channels. The total volume of water in the tank at full tank level was found to be 158210.85 m^3 where as it was 244353.81 m^3 at maximum water level.

MATERIALS AND METHODS

Design of Buried Pipe Distributary system

The main advantages of buried pipe line system are saving of land, elimination of seepage losses, and relatively little maintenance. The efforts made on implementation of buried pipeline system in the command area. In supportive to present study, Campbell (1984) reported that pipe systems in northern India assured flow delivery at the design discharge to the furthest irrigator with a minimum losses and unauthorized diversions en route. Adoption of buried pipeline distributary systems has lead to the reduction in water transit and distribution losses, reduction in the land area taken up by the distribution system and reduction in the maintenance and operating costs of the irrigation system. Murthy (2002) also opined that buried pipe systems were used for conveying irrigation water on the farms have worked efficiently. The entire command area of the tank was divided into five sections so that water can be given to each section once in five days and also to reduce the cost of pipe system considering 148 existing plots consisting of 65 farmers in the command area. The salient features of the command area and the existing land profile, the main channels and sub channels were considered while designing the buried pipeline system. The information on the outlets of buried pipe system for individual plots has been considered and the rate of water discharge in the pipe system for individual plot has been worked out. The buried pipe distributary system was designed based on the rate of water discharge in the pipe system for individual plots, crop water demand of the command area and cropping pattern. The friction losses in the pipe were estimated by Hazen-Williams equation. The design of pipe system for water supply through buried pipes and for efficient irrigation, it is necessary to know the hydraulics of the pipe systems. The long history of hydraulics of the pipe system is referred. The extension of reviewed paper is presented. In designing the pipe system, the hydraulics of pipes plays the major role in the contest of hydraulics

Hydraulics

Curt Reynolds et al., 1995 elaborately discussed the hydraulics in designing the low bubbler irrigation system and describe about the rigid PVC lateral, in reducing friction loss which is critical factor for designing the pipe system ,The major loss due to friction and minor losses are discussed in elaboration. The energy equation or Bernoulli's equation is the primary hydraulic equation used for pipe design described as;

$$\frac{v_1^2}{2g} + \frac{p_1}{w} + z_1 = \frac{v_2^2}{2g} + \frac{p_2}{w} + z_2 + h_f + h_l \qquad \text{........(1)}$$

where,

p = unit pressure, w = unit weight of water, v = mean velocity of flow, z_1 and z_2 elevation head, p/w = pressure energy, $v^2/2g$ = velocity energy, h_f = head loss by friction, h_l = all loss in pressure head other than friction.

The energy equation is useful to fix the pipe diameters of a system by determining the piezometric heads for the upstream and downstream ends of the pipe system. The piezometric heads at the upstream and downstream ends of the system are determined from the elevation of the water source and the field layout. The difference between the upstream and downstream piezometric heads is the total allowable head loss through the system. The friction loss through pipe components of Piping system will comprise a certain amount of the total allowable head loss. Initially the pipe diameter will be assumed the velocity heads and minor losses are assumed as zero but they will be accounted for later when calculating the delivery heights. The diameter of each pipe can be determined by substituting the assumed flow rate, known pipe length, and calculated allowable head loss for each pipe component into the friction loss equation and solving in terms of the diameter. The Darcy Weisbach's equation is the most universal formula used for computing head loss in all types of pipes (Brater and King, 1976; Orsan and Hansen, 1962; Mays, 1999; Munson *et al.,* 1998 and Streeter *et al.,* 1998). This equation has a long history of development. It is named after two of the great hydraulic engineers of the middle 19^{th} century, but others have also played a major role. Juleis Weisbach (1806-1871) a native of Saxony, proposed eq. 4 in 1845 (Glenn Brown, 2002). Consider a uniform horizontal pipe with cross- sectional area (A) through which water is flowing at velocity (V). Let p_1 and p_2 be the pressures at two points at distance L. If f indicates the frictional resistance per unit area at unit velocity, the total frictional resistance over the L is given as

$$\mathrm{F} = \mathrm{f}'(Surface\ area)\ \mathrm{v}^2$$

$$F = f'(PL)V^2 \qquad \text{... (2)}$$

where,

P is the wetted perimeter.

For circular pipe running full, the wetted perimeter is equal to the circumference πD, where D is the diameter of the pipe.

The pressure force acting at the ends of the pipe is given by:

$$F = \left(P_1 - P_2\right)\frac{\pi D^2}{4} \qquad \text{... (3)}$$

where,

P_1 = pressure at first point

P_2 = pressure at second point

Since the fluid is moving at constant velocity, the acceleration is zero. According to Newton's second law of motion, the net force on the fluid must be zero i.e.

$$\left(P_1 - P_2\right)\frac{\pi D^2}{4} = f'(PL)V^2$$

$$\frac{\left(P_1 - P_2\right)}{\gamma} = \frac{4f'}{\gamma}\left(\frac{L}{D}\right)V^2$$

Or the head loss due to friction h_f is given by

$$h_f = f\left(\frac{L}{D}\right)\frac{V^2}{2g} \qquad \dots (4)$$

where,

$$f = \frac{4f'}{\gamma}(2g)$$

Eq. 4 is known as a Darcy Weisbach equation. In this equation f is a dimensionless coefficient known as the friction factor, γ is density of water. The pipe friction factor (f) depends on the Reynolds Number (R_e) of the flow and the roughness of the pipe. Reynolds Number, R_e which is dimensionless. The friction coefficient f is given by, Eq. 5 for laminar, (eq. 6) for transitional and (eq. 7 and 8) for turbulent flow condition, for smooth plastic pipes:

$$\text{For } R_e < 2000 \quad f = \frac{64}{R_e} \qquad \dots (5)$$

$$\text{For } 2000 < R_e < 4000 \quad f = 3.42 \times 10^{-5} R_e^{0.85} \qquad \dots (6)$$

$$\text{For } 4000 < R_e < 10^5 \quad f = \frac{0.316}{R_e^{0.25}} \qquad \dots (7)$$

$$\text{For } 10^5 < R_e < 10^7 \quad f = \frac{0.13}{R_e^{0.172}} \qquad \dots (8)$$

Keller and Bliesner (1990) and Boswell (1984) recommend eqs. 5, 7, and 8 in micro irrigation design, with eq. (7), the Blasius equation having a Reynolds Number lower limit of 2000. The Reynolds Number lower limit for the Blasius equation is typically 3000 to 4000, however, for desk top calculation eq. 6 was defined by Wu and Fangmeier (1974) ignored by setting the lower limit for the Blasius equation at 2000. By combining the Darcy Weisbach (eq. 4) and the Blasius (eq. 7) an equation for smooth pipes, similar in form to the Hazen-Williams equation is obtained for $2000 < R_e < 10^{-5}$

$$h_f = K_1 \frac{Q_1^{1.75}}{D^{4.75}} L \qquad \dots (9)$$

where,

D is the internal diameter (mm), for pipes less than 128 mm diameter; K is 7.89 x 10^5 for SI units (0.00133, English units), and for water temperature at 20^0C (68^0F), Q_1 is the flow within pipeline (l/s), L is the length of pipeline (m).

For smooth pipes larger than 128 mm, Keller and Bliesner (1990) noted that the Reynolds number will typically be larger than 100000. Therefore, by incorporating eq. 8 into eq. 4, a friction loss equation for large smooth pipes was obtained for $10^5 < R_e < 10^7$

$$h_f = K_1 \frac{Q_1^{1.828}}{D^{4.828}} \qquad \dots (10)$$

where,

D is the internal diameter (mm) for pipes greater than 128 mm diameter, K_1 is 9.58x10^5 for SI units, (0.001 for English units), and for water temperatures at 20^0C. Hence, eq. 9 and 10 are for small smooth and large smooth pipes respectively. The final turbulent flow formulas recommended for calculating friction losses in pipe systems when using the Darcy-Weisbach equation. Simplify calculations for these equations (Fig. 1) is based on friction loss eqs. 9 and 10. The Christiansen (1942) reduction coefficient, F, which is commonly used to calculate head losses in multiple outlet pipes and to initially size the diameters of mainline, manifolds, and laterals. Applying the Christiansen reduction coefficient to head loss eqs. 9 and 10. Simplifies calculations for multiple outlet pipes because it estimates the friction loss along the entire length of multiple outlet pipes such as manifolds and laterals. By using the Christiansen coefficient, the total friction loss for a multiple outlet pipe is expressed as:

$$h_f = F_2 \times h_{f'} \qquad \dots (11)$$

where,

h_f is the friction head loss between the upstream and downstream ends of a multiple outlet pipe (m), F_2 is the Christiansen reduction coefficient that depends on the number of outlets along the multiple- outlet pipe (Table 1). $h_{f'}$ is the friction loss in a length of pipe assuming no outlets along the pipe (m).

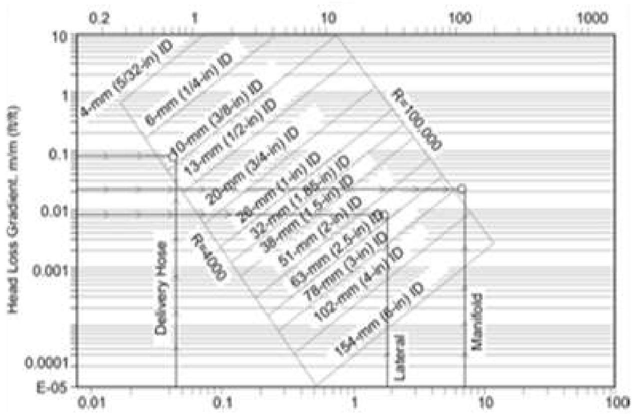

Fig. 1. Head loss based on Darcy weicbach Equation

Table 1. Christiansen reduction coefficient, F_2 for equally spaced outlets along manifolds and laterals[a]

Number of	F_2		Number of	F_2	
Outlets	End[b]	Mid[c]	outlets	End	Mid
1	1	1	8	0.42	0.38
2	0.64	0.52	9	0.41	0.37
3	0.54	0.44	11-Oct	0.40	0.37
4	0.49	0.41	15-Dec	0.39	0.37
5	0.46	0.40	16-20	0.38	0.36
6	0.44	0.39	21-30	0.37	0.36
7	0.43	0.38	>31	0.36	0.36

[a] After Keller and Bliesner (1990).
[b] First outlet is a full space from pipe inlet
[c] First outlet is one –half space from pipe inlet.

Common Christiansen factors are shown in Table 1. They are based on the assumption that all water is carried to the end of the line and that all the multiple outlets are evenly spaced with equal discharge. From Table 1, it is noted that Christiansen factors with half spacing are smaller than for full spacing, there by reducing the total friction loss in the lateral. For the design of pipe systems, any savings in friction head loss is critical, and therefore, designing the first outlet with half–spacing is recommended to minimize head loss and to utilize the field area more efficiently for orchard crops. The slope of the field is another crucial factor in design. The design for systems located level tied and those on gradual slopes differ slightly because the location of the maximum and minimum delivery heights will occur at different points along the lateral. Also systems on fields with gradual slopes will gain energy down slope, thus increasing the allowable head loss gradient, which is used for sizing pipe diameters. This extra energy allows laterals to be longer on fields with gradual slopes than on level fields and permits greater diversity in design for a given available head.

The allowable head loss gradient for sizing pipeline diameters is determined by the following equation for both level and gradual field slope designs:

$$\frac{h_f}{L} = \frac{\left(H_u - H_d\right) - \Delta z}{F_2 L} = \frac{\left(h_f\right)_a}{F_2 L} \qquad \ldots(12)$$

Where.

$\frac{h_f}{L}$ is the head loss gradient (m/m), H_u is the pressure head upstream (m), H_d is the pressure head downstream (m), Δz is the change in elevation between upstream and downstream (negative for downstream; m), $\left(h_f\right)_a$ is the allowable head loss in the pipe (m).

Darcy (1857) introduced the concept of the pipe roughness scale by the diameter as the relative roughness when applying the diagram. Therefore, it is traditional to call f, the "Darcy factor", even though Darcy never proposed it in that form. Fanning (1877) apparently was the first effectively put together two concepts. He published a large compilation of f values as a function of pipe material, diameter and velocity. However, it should be noted that Fanning used hydraulic radius, instead of D in the friction equation, thus "Fanning f" values are only 1/4th of Darcy f values. Parallel to the development in the hydraulics, viscosity and laminar flow were defined by Jean Poisseuille (1799-1869) and Gotthilf Hagen (1797-1884), while Osborne Reynolds (1842-1912) described the transition from laminar to turbulent flow in 1883. During the early 20th century, Ludwig Prandtl (1875-1953) and Th. Von Karman (1881-1963) Paul Blasius (1883) and Johnann Nikuradse (1894-1979) attempted and provided an analytical prediction of the friction factor using both theoretical considerations data from smooth and uniform sand lined pipes. Their work was complimented by Colebrook and White (1939). The Darcy Weisbach's equation was not made universally useful until the development of the Moody diagram (Moody, 1944). Fig. 2, which built on the work of Hunter Rouse. Rouse (1946) gives a good feel for the development of the f factor, but he doesn't reference Moody. Rouse felt that Moody was given too much credit for what Rouse himself and others did (Rouse, 1976). Rouse, 1946 appears to be the first to call it Darcy-Weisbach equation (Glenn Brown, 2002).

The Darcy Weisbach equation with the Moody's diagram are considered to be the most accurate model for estimating frictional head loss in steady pipe flow.

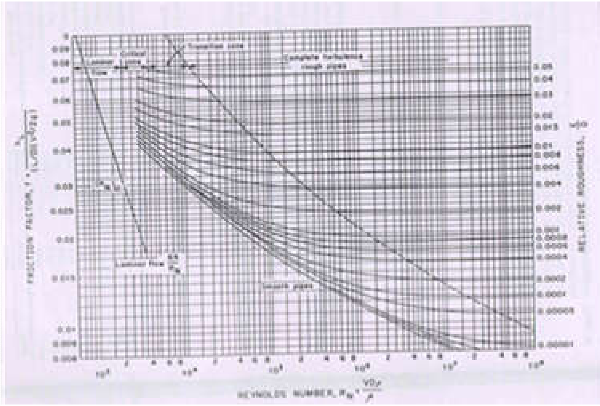

Fig. 2. Moody diagram for friction loss in pipes (adopted from Moody 1944)

Since, the approach does not require a efficient trial and error solution, or an alternative empirical head loss calculation that do not require the trial and error solutions, as the Hazen-Williams, (1960) equation, may be preferred (www.EngineeringtoolBox.com).

Finkel (1985) expressed Hazen-Williams equation, as

$$V = 354Q\,D^{-2} = 1.096\times10^{-4}\,C\,J^{0.54}\,D^{0.63} \qquad \ldots(13)$$

$$= 3.97\times10^{-3}\,C^{0.761}Q^{0.239}J^{0.411}$$

$$J = 1.131\times10^{12}\left(\frac{Q}{C}\right)^{1.852}D^{-4.87}$$

$$= 2.16\times10^{7}\left(\frac{V}{C}\right)^{1.852}D^{-1.167}$$

$$= 7.02\times10^{5}\,C^{-1.852}V^{2.436}Q^{-0.584}$$

$$Q = 2.83\times10^{-3}VD^{2}$$

$$= 3.1\times10^{-7}CJ^{0.54}D^{2.63}$$

$$= 1.057\times10^{10}\left(\frac{V}{C}\right)^{4.175}J^{-1.714}$$

$$D = 18.8Q^{0.5}V^{-0.5}$$

$$= 298\left(\frac{Q}{C}\right)^{0.38}J^{-0.205}$$

$$= 1.93\times10^{6}\left(\frac{V}{C}\right)^{1.587}J^{-0.857}$$

where,

V = velocity (m/sec), C = coefficient (co-efficient for PVC pipe from 140 to 180), J = hydraulic gradient (ppm), Q = discharge (m^3/hr), D = pipe diameter (mm). Hazen Williams equation (Mays, 1999); Streeter *et al.,* 1998; Viessman and Hammer, 1993) where K_3=0.85 for meter and seconds units.

$$H = L\left[\frac{V}{K_3 C(4/D)^{0.63}}\right]^{\frac{1}{0.54}} \quad \ldots (14)$$

where,

$V=Q/A,\ A = \pi D^2/4$

Brater and King, 1976 and Jaico 2003 expressed Hazen-Williams equation as

$$V = CR_2^{0.63}\, S_2^{0.54} \quad \ldots (15)$$

where,
R_2 = the hydraulic radius (m)
S_2 = the friction slope (m/m)
$C_2 = 0.0109C_3$

where,
C_3 = the Hazen-Williams resistance coefficient.

The Hazen- Williams method is very popular especially among civil engineering, since its friction coefficient (c) is not a function of velocity or duct diameter. Hazen-Williams is simple than Darcy-Weisbach equation for calculation of flow rate, velocity or diameter (LMNO Engineering Research and Software, 2001). There are numerous methods for computing head loss due to friction in pipelines. One of the most common and convenient methods applicable to pumping water through irrigation systems is the Hazen-Williams equation (Cuena, 1985).

$$h_f = K_4 L \frac{(Q_3/C_4)^{1.852}}{D^{4.87}} \quad \ldots (16)$$

where,
K_4 = Conversion constant, L = Length of pipe, L,Q_3 = Volumetric flow rate, L^3/T, C_4 = Hazen William's coefficient, D = Pipe diameter, L

Table 2 indicates common units associated with flow in pipes in the SI and English systems and the required conversion constant K_1 for Eq. (16). Hazen-Williams C coefficient for polyethylene (PE) and polyvinyl chloride (PVC) are 140 for design, for new pipe 150. The Hazen-Williams equation is only applicable to water at standard operating temperature (i.e., 20°C), or more specifically to fluids with a specific gravity of 1.0. Such an assumption is almost always valid for analysis of flow in irrigation systems.

Table 2. Conversion constants for the Hazen-Williams equation with different combinations of units

h_l	L	Q_3	D	K_4
m	m	L/s	mm	1.22×10^{10}
m	m	L/h	mm	3163
m	m	M^3/d	mm	3.162×10^6
ft	ft	ft^3/s	ft	4.73
ft	ft	gpm	in	10.46

Based on the Hazen-Williams flow formula several graphs has been presented in several books. Considering mean internal dimensions and tolerances to IS 4985 the graph has been developed (Jain 2004) and presented in the Fig. 3. Based on this graph the soft ware is used for designing the underground pipe system in this study.

Minor losses

Minor losses such as pipe elbows, bends, and valves may be included by using the equivalent length of pipe method (Mays, 1999). Local head losses occur in a pipe network due to entry, bends, valves, and changes in diameter. These may amount to from 2 to 20 % of the total head losses, and consequently not always negligible. The local losses are expressed in head, h (m) as a function of the velocity head, as follows:

$$h_{ml} = K_{ml}\frac{V^2}{2g} \quad \ldots (17)$$

where,

h_{ml} is expressed in m, V in m/sec, Values for K_{ml} are given in Table 2.6.

Equivalent pipes

Two pipes are to be equivalent if they produce equal losses of head at the same discharge. Similarly a given section of pipe may be equivalent to a fitting or other local head loss produce if the pipe section causes the same loss of head as the fitting at corresponding values of discharge.

The concept of an equivalent pipe is used in the analysis of networks to simplify the computations and avoid going into details at early stages of the analysis. Considering the head losses due to fittings such as valves, pipe bends, reducers, etc. which are located at various points in a pipe line, they are usually represented by an equation of the.

$$h_{eq} = K_{ml}\frac{V^2}{2g} \quad \ldots. (18)$$

where,
K is the local head loss coefficient, h_{eq} is the head loss caused by the fitting, and V is the mean velocity in the pipeline.

To derive an equation for the equivalent pipe this expression is compared to the Darcy-Weisbach equation for longitudinal head losses.

$$h_{eq} = f\left(\frac{L}{D}\right)\frac{V^2}{2g} \quad \ldots (19)$$

where,

f is the coefficient of friction, L is the length of pipe, D is the diameter of pipe. Table 3 shows the resistance co-efficient for fittings.

Comparing the two equations for head losses at the same discharge or mean velocity (V), leads to

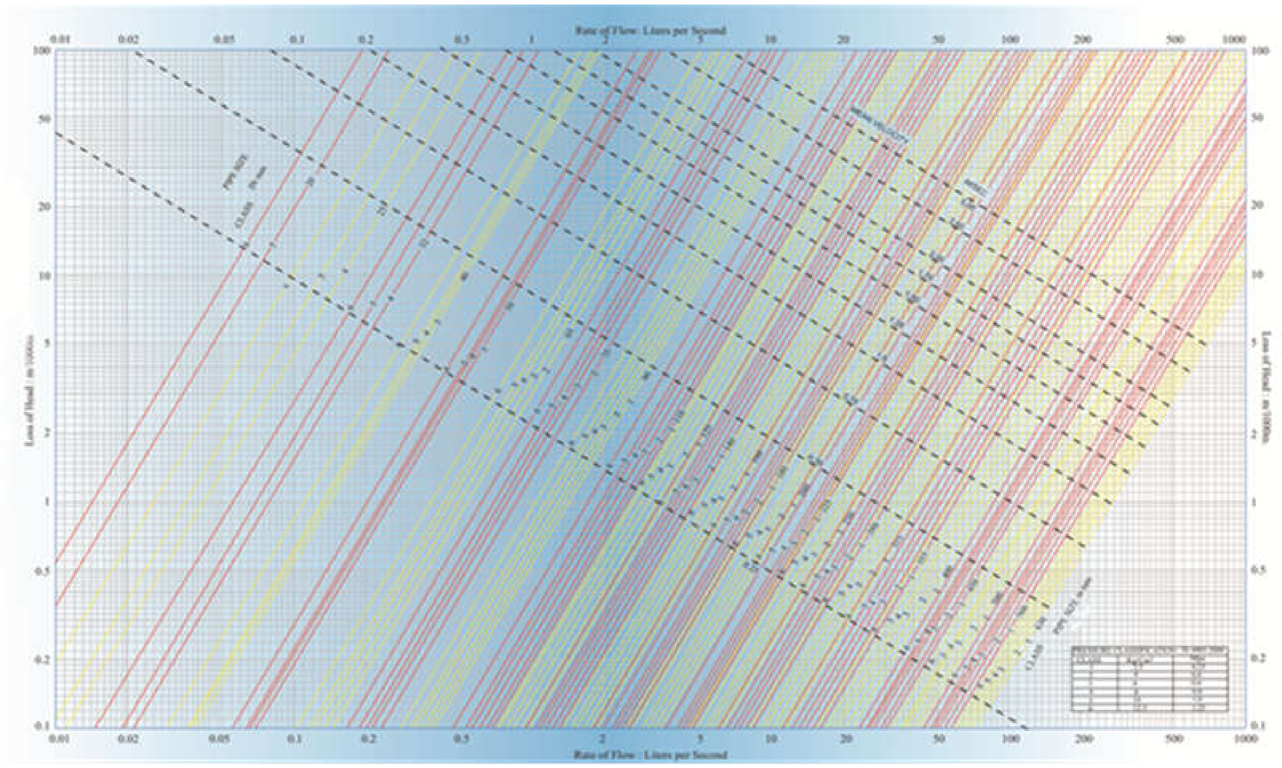

Fig. 3. Head loss based on Hazen Williams flow formula (Based on ID and tolerance to IS4985)

Table 3. Resistance coefficient K_{ml} for use in formula for fittings and values

Fitting or Valve	a. Nominal diameter						
	75mm	100mm	125mm	150mm	175mm	200mm	250mm
Standard pipe							
Elbows							
Regular flanged 90º	0.34	0.31	0.3	0.28	0.27	0.26	0.25
Long radius flanged 90º	0.25	0.22	0.2	0.18	0.17	0.15	0.14
Regular screwed 90º	0.8	0.7					
Tees							
Flanged line flow	0.16	0.14	0.13	0.12	0.11	0.1	0.09
Flanged branch flow	0.73	0.68	0.65	0.6	0.58	0.56	0.52
Screwed line flow	0.9	0.9					
Screwed branch flow	1.2	1.1					
Valves							
Globe flanged	7	6.3	6	5.8	5.7	5.6	5.5
Gate flanged	0.21	0.16	0.13	0.11	0.09	0.075	0.06
Swing check flanged	2	2	2	2	2	2	2
Foot	0.8	0.8	0.8	0.8	0.8	0.8	0.8
Strainers-basket type	1.25	1.05	0.95	0.85	0.8	0.75	0.67
Other Inlets or entrances							
In ward projecting	0.778			All diameters			
Sharp cornered	0.5			All diameters			
Slightly rounded	0.23			All diameters			
Bell-mouth	0.04			All diameters			
Sudden enlargement	$K_{ml} = \left(1 - \frac{d_1^{\,2}}{d_2^{\,2}}\right)^2$ where d_1 = diameter of smaller pipe, d_2 = diameter of larger pipe						
Sudden contraction	$K_{ml} = 0.7\left(1 - \frac{d_1^{\,2}}{d_2^{\,2}}\right)^2$ where d_1 = diameter of smaller pipe, d_2 = diameter of larger pipe						

Source: USDA, SCS, 1968

$$V = \frac{K_{ml} D}{f} \quad \ldots (20)$$

which is an expression for the length of a pipe equivalent to the given fitting. Adopting a mean value of f= 0.025, the last equation may be replaced by the following equation;

$$L = 40kD$$

indicating that the length expressed in pipe diameters of the equivalent pipe for a given fitting is approximately 40 times the local head loss coefficient.

If two pipes carrying the same discharge are compared, Darcy-Weisbach's equation should be written in the form as

$$h_{eq} = \frac{8 f L Q^2}{\pi^2 g D^5} \quad \ldots (21)$$

Comparing two such equations with f_1, L_1, D_1 representing one pipe and f_2, L_2, D_2 for the second pipe, and assuming equal discharge Q and equal head losses h_{eq} in the two pipes, the following expression is obtained for the equivalent length.

$$L_2 = \left(\frac{f_1}{f_2}\right) \times \left(\frac{D_2}{D_1}\right)^5 L_1 \quad \ldots (22)$$

This equation gives the length of a pipe of diameter D_2 and friction coefficient f_2 which is equivalent to a pipe of diameter D_1, length L_1, and friction coefficient f_1. If the two friction coefficients are equal, the above expression becomes;

$$L_2 = \left(\frac{D_2}{D_1}\right)^5 L_1 \quad \ldots (23)$$

By a similar procedure based on the Hazen-Williams equation it can be shown that if this equation is adopted the expression for the equivalent length of one pipe which represents another pipe;

$$L_2 = \left(\frac{C_{H_2}}{C_{H_1}}\right)^{1.852} \left(\frac{D_2}{D_1}\right)^{4.87} L_1 \quad \ldots (24)$$

where,
C_{H_1} and C_{H_2} and are the Hazen-Williams coefficients for the two pipes.

If the two coefficients are equal then the eq. (2.35) becomes

$$L_2 = \left(\frac{D_2}{D_1}\right)^{4.87} L_1 \quad \ldots (25)$$

Except for cases where short lengths of pipes of one diameter are included in pipe lines of another diameter, the concept of an equivalent pipe replacing a given pipe is not much used. The other concept of an equivalent pipe to replace a fitting is more common. In the design of pipe lines for the conveyance and distribution of water an allowance is usually made for the effect of fittings in the pipe lines by adding to the actual length of the pipe an equivalent length to represent the fittings. For preliminary designs the added length is taken to be between 5 and 20 % of the original length depending on the number of fittings.

Conclusion

There are numerous methods for computing head loss due to friction in pipelines. One of the most common and convenient methods applicable to pumping water through irrigation systems is the Hazen-Williams equation. The Hazen- Williams method is very popular especially in design of pipe system, since its friction coefficient (c) is not a function of velocity or duct diameter. Hazen-Williams is simple than Darcy-Weisbach equation for calculation of flow rate, velocity or diameter of pipe system.

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