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THE EFFECT OF PRESSURE ON LEAKAGE IN WATER DISTRIBUTION SYSTEMS

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Abstract

The results of pressure management field studies have shown that the leakage exponent is often considerably higher than the theoretical orifice value of 0.5. The purpose of this paper is to identify and analyse factors that may be responsible for the higher leakage exponents. Four factors are considered: leak hydraulics, pipe material behaviour, soil hydraulics and water demand. It is concluded that a significant proportion of background leakage can consist of transitional flow, and thus have a leakage coefficient value above 0.5 (although not above 1). An important factor is pipe material behaviour: laboratory test results are presented to show that pipe material behaviour can explain the range of leakage exponents observed in the field. The complexity of the interaction between a leaking pipe and its surrounding soil is discussed and it is concluded that the relationship between pressure and leakage is unlikely to be linear. Finally, it is noted that if water demands are present in minimum night flows, the resulting leakage exponent is probably underestimating the true value.

NOTATION

A	orifice or hole area
c	leakage coefficient
c'	stress factor
C	constant
C_d	discharge coefficient
d	hole diameter
d_0	original hole diameter
Δd	change in hole diameter

D pipe diameter

E elasticity modulus

F shape factor for soil flow region

g acceleration due to gravity

h pressure head

k soil coefficient of permeability

n aspect ratio of a rectangle

P wetted perimeter

q flow rate

Q_{dem} demand flow rate

R hydraulic radius

Re Reynolds number

t pipe wall thickness

v velocity

α leakage exponent

β water demand elasticity

ε material strain

ρ fluid density

σ material stress

ψ kinematic viscosity

Note to the Editor: The accepted symbol for kinematic viscosity is not ψ as used in this manuscript, but the Greek letter nu (ν). The change was made to avoid confusion with the symbol v, which looks very similar to the Greek nu in the font used. We will appreciate it if the correct symbol ν (Greek nu) can be used in the printed paper, provided that it can be distinguished clearly from the letter v.

1. INTRODUCTION

Water distribution systems world-wide are aging and deteriorating, while the demands on these systems, and thus on natural water resources, are ever increasing. Losses from water distribution systems are reaching alarming levels in many towns and cities throughout the world. Water losses are made up of various components including physical losses (leaks), illegitimate use, unmetered use and under-registration of water meters. Leakage makes up a large part, sometimes more than 70 % of the total water losses[1].

One of the major factors influencing leakage is the pressure in the distribution system. In the past the conventional view was that leakage from water distribution systems is relatively insensitive to pressure, as described by the orifice equation:

$$q = AC_d\sqrt{2gh} \qquad \text{... (1)}$$

Where q the flow rate, C_d a discharge coefficient, A the orifice area, g acceleration due to gravity and h the pressure head differential over the orifice. To apply this equation to leaks in pipes it can be written in more general form as:

$$q = ch^{\alpha} \qquad \text{... (2)}$$

Where c is defined as the leakage coefficient and α as the leakage exponent (α is sometimes referred to as *N1*). A number of field studies have shown that α can be considerably larger than 0.5, and typically varies between 0.5 and 2.79 with a median of 1.15 [2]. This means that leakage in water distribution systems is much more sensitive to pressure than conventionally believed. The range of exponents observed reflects substantial differences in the impact of pressure on rate of leakage. For example, halving the pressure in a pipe will result in reductions in flow rate of 29 %, 50 % and 82 % respectively for exponents of 0.5, 1.0 and 2.5. The reasons for the high leakage exponents are not well understood, but an important cause is believed to be the expansion of the hole opening with increasing pressure [2].

The large influence of the leakage exponent when estimating the potential impact of pressure management on leakage rate means that it is essential to develop an understanding of the mechanisms responsible for the observed behaviour. The purpose of this paper is to identify possible causative factors and, where possible, quantify the effect of these factors on the leakage exponent. The possible causative factors are discussed under four headings: leak hydraulics, pipe material behaviour, soil hydraulics and water demand.

2. LEAK HYDRAULICS

The Orifice equation (equation 1) is derived for an orifice in the side of a tank and describes the conversion of all the potential energy, in the form of pressure, to kinetic energy. The discharge coefficient is added to incorporate energy losses and the reduction of jet diameter downstream of the orifice. The pressure in the jet downstream of the orifice is assumed to equal that of the surrounding fluid.

The hydraulic behaviour of orifices has been researched extensively and can be predicted with some degree of certainty. The exponent of 0.5 is generally only true for large Reynolds numbers (Re). For smaller Reynolds numbers, equation 1 is typically modified by writing the coefficient c as a function of the Reynolds number. This variable coefficient can also be expressed as a fixed coefficient with an exponent that is not 0.5. For example, substituting the expression for laminar flow through an orifice (from [3]) into equation 1 results in an equation with constant coefficient and an exponent of 1. For transitional flow, the equivalent exponent will vary between 0.5 at the transitional-turbulent flow boundary to 1.0 at the laminar-transitional flow boundary.

As noted above, the flow regime is determined by the Reynolds number (Re). Flow through orifices is typically laminar at Re below 10 and turbulent at Re above 4000 to 5000 [3]. The Reynolds number for a general leak opening or orifice can be written as:

$$\begin{aligned} \mathrm{Re} &= \frac{4vR}{\psi} \\ &= \frac{4q}{\psi P} \end{aligned} \qquad \text{... (3)}$$

Where v velocity and ψ kinematic viscosity of the fluid, and R the hydraulic radius of the orifice (defined as flow area A divided by the wetted perimeter P).

Since the kinematic viscosity of a fluid is a function of temperature, it follows from the equation that the leakage flow rate for a fixed Reynolds number (e.g. for maximum laminar or transitional flow) and fluid is only affected by two variables: the temperature of the fluid and the wetted perimeter of the orifice. The viscosity of water approximately halves when its temperature increases from 0 to 30 ˚C, meaning that the maximum laminar or turbulent flow will approximately double. Leak openings with large wetted perimeters (such as cracks) will be able to sustain much larger laminar or transitional flow rates than circular openings with the same areas.

It is possible to find an expression for the maximum laminar and transitional flow rates through different types of leak openings for the typical pressure range in a water distribution system. First, the flow rate is written as the product of the velocity and area of an opening. For a circular opening, this is given by:

$$Q = 0.25\pi D^2 v \qquad \ldots(4)$$

Where D the diameter of the leak opening. Writing equation 3 in terms of the hole diameter and replacing it and equation 1 into equation 4 results in the expression:

$$q = \frac{\pi\psi^2 \operatorname{Re}^2}{4C_d\sqrt{2gh}} \qquad \ldots (5)$$

For a rectangular leak opening with an aspect ratio of *n*, the expression is given by:

$$q = \frac{(n+1)^2 \psi^2 \operatorname{Re}^2}{4nC_d\sqrt{2gh}} \qquad \ldots (6)$$

If a constant discharge coefficient (say C_d = 0.6) and kinematic viscosity (say ψ = 1.14 x 10^{-6} for water at 15 ˚C) are assumed, the equations can be used to estimate the maximum laminar and transitional flow rates that are possible in water distribution networks. Cracks can be viewed as rectangular leak openings with high aspect ratios. The maximum laminar and transitional flow rates for different types of leak openings are shown in Figure 1 for the 10 to 100 m pressure range, which covers the pressures found in most water distribution systems.

The figure shows that cracks can have much higher laminar or transitional flow rates than round or square holes. This is due to the role of their much larger wetted perimeters. Theory predicts that the maximum possible flow rates that are fully laminar are typically very small (e.g. less than 3 l/day even for a crack with an aspect ratio of 10 000) and it is thus unlikely that substantial losses from water distribution systems will occur in the fully laminar zone.

A distinction is often made between bursts and background leakage. Bursts are large individual leaks that come to the surface or are found through active leakage control initiatives. Background leakage comprises numerous small leaks that are very difficult or impossible to detect without excavating the pipe. In a well-run system, much of a network's water loss that we seek to reduce through pressure control may thus result from background leakage. This view is supported by water leakage figures for England and Wales (about 25 million connected properties) that has been estimated by OFWAT [4] in 2004 to be on average of the order of 10 m^3/km of main/day, or 360 l/h/km of main. Comparing this figure with the maximum transitional flow rates above indicate that it is possible that much of the background leakage can occur in this range, especially in systems that are likely to have pipes that develop crack failures. Transitional flow can thus be an important cause of a leakage exponent above 0.5 (although not above 1.0) when background leakage is a large contributor to leakage from a system.

3. PIPE MATERIAL BEHAVIOUR

Pipe material plays an important role in the leakage behaviour of pipes. Water pressure in a pipe is taken up by stresses in the pipe wall, and thus may be a factor in failure and leakage behaviour. The following effects can be linked to an increase in the internal pressure of a pipe:

- Small cracks or fractures that do not leak at low pressures open up to create new leaks.
- The area of existing leak openings in a pipe increase due to increased stresses in the pipe wall.
- The frequency of pipe bursts increases [2, 5] with a corresponding increase in maintenance costs.

Greyvenstein and Van Zyl [6] used an experimental setup to measure the leakage exponents of failed pipes taken from the field and pipes with artificially induced leaks. The study included round holes, and longitudinal and circumferential cracks in uPVC, steel and asbestos cement pipes. All flows were turbulent and leaks were exposed to the atmosphere. The resulting leakage exponents varied between 0.42 and 2.4 as detailed in Table 1. The main findings of the study were:

- The results confirm that the leakage exponents found in field studies are not unrealistic.
- The highest leakage exponents occurred in corroded steel pipes, probably due to corrosion reducing the support material around the hole. This is contrary to the

perception that plastic pipes will have higher leakage exponents due to their lower modulus of elasticity.

- Round holes had leakage exponents close to the theoretical value of 0.5 and no significant difference was observed between steel and uPVC pipes.
- Besides corrosion holes, the largest exponents were found in longitudinal cracks. This is due to the fact that circumferential stresses in pipes are normally significantly higher than longitudinal stresses.
- The leakage exponents for circumferential cracks in uPVC pipes were sometimes less than 0.5, suggesting that the leak opening might be contracting with increasing pressure. This is explained by the fact that the experimental setup did not allow substantial longitudinal stresses to develop in the pipe. It is thought that the circumferential stresses caused the cracks to elongate, and at the same time reduce in area. These results have subsequently been verified through finite element analysis [7].

Theoretical expressions for the longitudinal and circumferential stresses in a pipe under pressure are given in many textbooks (for example, see [8]). These expressions show that the stresses in the circumferential direction are double those in the longitudinal direction. When a discontinuity such as a hole is present, the pipe wall stresses are increased in the vicinity of the hole. The circumferential stress in the vicinity of the hole is now written as:

$$\sigma = \frac{c'\rho g D h}{2t} \quad \text{... (7)}$$

Where σ the pipe wall stress, c' a stress factor, D the pipe diameter, h the pressure head and t the pipe wall thickness. The stress factor incorporates both the variation in stress around the circumference of the hole and the stress concentration factor. Assuming linear elastic behaviour, the wall stress can also be written in terms of the strain ε and elasticity modulus E:

$$\sigma = \varepsilon E = \frac{\Delta d}{d_0} E \qquad \text{... (8)}$$

Where d_0 the original hole diameter and Δd the change in diameter due to the pressure in the pipe. Using equations 7 and 8, the hole diameter $d = d_0 + \Delta d$ can now be expressed as:

$$\begin{aligned} d &= d_0\left(1 + \frac{c'\rho g D h}{2tE}\right) \\ &= d_0(1 + Ch) \end{aligned} \qquad \text{... (9)}$$

Where C is a constant. Substituting the equation for the area of a hole (based on equation 9) into equation 1 results in the following expression for the leakage flow rate from a circular hole in a pipe:

$$q = \sqrt{0.125g}\ \pi C_d d_0^2 \left(h^{\frac{1}{2}} + 2Ch^{\frac{3}{2}} + C^2 h^{\frac{5}{2}}\right) \qquad \text{... (10)}$$

The relationship shows that the processes involved in the expanding leak opening are more complex than the simple power relationship normally used to describe leakage. The equation contains the sum of three terms with leakage exponents of 0.5, 1.5 and 2.5 respectively, which seem to tie in well with field and experimental observations. However, when calculating the leakage from typical pipes using equation 10 it is found that the terms with exponents 1.5 and 2.5 contribute little to the leak under normal pressure conditions.

Due to the material properties, pipes of different materials will fail in certain characteristic ways. For instance, longitudinal cracks are common in asbestos cement pipes, while steel and cast iron pipes often leak through corrosion holes. Small-diameter cast iron pipes typically fail in bending leading to circumferential cracks which, because of the relatively high coefficient of thermal expansion of the pipe material, may open and close as the temperature of the water in the system changes. Understanding the failure behaviour of pipes and the associated leakage exponents can assist with modelling the response of a given distribution system to a change in pressure and in better managing leakage reduction programmes.

4. SOIL HYDRAULICS

A simplistic application of geotechnical seepage theory would suggest, in contrast to equation (1), that if head losses through the pipe orifice are neglected, the flow rate should be linearly proportional to the head of the water in the pipe, *h*, since following Darcy's Law, the flow rate (*q*) in the soil for a given head on the orifice water/soil boundary will be [13]

$$q = F \,.\, k \,.\, h \qquad \text{... (11)}$$

Where *F* is the form factor for the soil flow region, and *k* is the coefficient of permeability of the soil. However this equation is underpinned by a number of assumptions, and these are not generally valid for seepage around a water pipe.

Firstly, in soil seepage analysis it is generally safe to assume that the velocity component of total head is very small, and can be ignored. The velocity of flow through soil under a hydraulic gradient of unity varies from 10^{-2} m/s for a clean coarse sand to 10^{-8} m/s and smaller for clays. Yet in contrast the hydraulics orifice equation (equation 1) predicts very high velocities at the soil/water interface. There is clearly an incompatibility here, and an equation for the combined system cannot be properly defined by the straightforward coupling the orifice equation with the soil seepage equation, as has been previously done [9]. Soil outside the pipe will modify downstream jet behaviour, whilst perhaps also obscuring part of the orifice itself. The simple Darcy soil seepage equation assumes that there is a fixed upstream boundary geometry with constant head applied to it but, because of the high orifice outlet velocity, both the boundary geometry and the head applied to it are likely to be modified by scour of the soil boundary and fluidisation of the soil. A number of studies have shown the complexity of these processes [10, 11, 12]

Secondly, the downstream boundary conditions in the ground surrounding the pipe are not generally constant regardless of flow rate. In many geotechnical seepage problems both upstream and downstream boundaries can reasonably be assumed to have fixed geometries and head conditions, and the position of any phreatic surface can be assumed fixed. Since water pipes are generally laid above the ground water level the seepage flow net (and thus F in equation 10 above) varies as a function of the rate of outflow from the pipe and the coefficient of permeability of the soil. For any given soil permeability, increasing flow leads to progressive build-up of pore pressure in the soil around the pipe, and eventual “mounding” of water above it. For low flow rates relative to the permeability of the soil, seepage will not reach the ground surface, and leakage will go unnoticed. At higher rates of leakage water will emerge at ground surface and a burst will be detected.

Thirdly there are limits to validity of Darcy's law. A linear relationship between head and flow in soil is, as observed by Osborne Reynolds in 1883, only valid for laminar flow. The critical value of Re (expressed in soil mechanics as $R = vD\rho / \mu$, where v is the discharge velocity (flow per unit cross section of soil), and D is the average soil particle diameter) at which flow in soil changes from laminar to turbulent has been found to range between about 1 and 10 (for example, see [13]) . Discharge velocity depends upon both hydraulic gradient and permeability (which is itself a function of particle size). Under the low hydraulic gradients ($\Delta h/\Delta l \ll 1$) typical of many soil seepage situations laminar flow can be expected in sands and finer materials, but not in gravels. However the hydraulic gradient around a leaking pipe will be much larger - water distribution pipes are generally buried at a depth of less than 1m, and have supply heads of the order of 30 m. Non-laminar flow can therefore be expected in most coarse granular soils and loose backfills.

Finally, the stress conditions in the ground contribute to the way in which flow takes place. Calculations of Darcy flow generally assume permeability to be constant, with flow distributed across the entire region of permeable soil. Considerations of force equilibrium make it clear, however, that for a particulate material such as soil the maximum water pressure in the pores between the particles, on any given plane, cannot exceed the (total[i]) stress on that plane. Once the water pressure at any point in the ground rises above the minor total principal stress (which may be in the horizontal or vertical direction, but is unlikely to

[i] The *total stress* on a plane in a soil mass is the stress on that plane that arises as a result of external loading and of the self weight of the soil. This is distinct from the *effective stress*, which governs the strength, compressibility and to some extent permeability of soil, and is the numerical difference between total stress and pore pressure on any given plane.

exceed 20-30kPa for typical pipe burial depths) then hydraulic fracture takes place. The soil cracks along planes of weakness, flow occurs preferentially along these cracks, flow rates rise through orders of magnitude, and conventional seepage analysis is no longer applicable (for an example see [14]). Because of their size, and for the reasons discussed above, flow along these cracks is unlikely to be laminar. As heads increase, the move from Darcy flow to hydraulic fracturing can be expected to produce flow increases that contribute to leakage exponents greater than unity.

Even if the water pressure is not sufficiently high to cause hydraulic fracture, if upward flow takes place in unbonded granular soil and its velocity become sufficiently great then fluidisation may occur. “Piping”, as this is known, results when the upward force on the soil particles resulting from seepage exceeds its buoyant self-weight, and occurs at a hydraulic gradient approximately equal to unity. Since the particles in the fluidised zone move as an integral part of the fluid, the overall permeability of the flow region is greatly reduced.

On average, leakage figures for a well-maintained system (such as the England and Wales estimate of about 0.1 l/s/km of main [4]) probably represent a few larger-volume infrequent bursts combined with a much greater number of continuous but undetected losses from smaller defects in the network. For example, vertical downward flow (gravitational flow, i.e. without any development of excess pore water pressure in the soil) from a single 0.1 litre/s leak would occupy a plan area of only about 10cm x 10cm in gravel, and 1m x 1m in sand, suggesting that in coarse granular soil leaks will be absorbed without trace by the ground around the pipe.

In summary, it can be concluded that the interaction between a leaking pipe and its surrounding soil is complex, and requires further investigation. The relationship between head loss and flow is unlikely to be linear, as a result of interaction of soil particles with the orifice, turbulent flow in the soil, the changing geometry of the unconfined flow regime, hydraulic fracturing and piping. Theoretical considerations suggest that small continuous leaks from pipes will drain away without trace into underlying granular soil. This cannot be expected to occur in lower permeability clays and silts, where hydraulic fracture is more likely, with leaks rapidly becoming visible as wet patches and bursts at the ground surface.

5. WATER DEMAND

While water demand is not classified as leakage, it is often impossible to separate legitimate water consumption from leakage measurements in the field. It is thus important to understand the behaviour of water demand as a function of pressure. The effect of pressure on demand Q_{dem} can be expressed as [15]:

$$Q_{dem} = Ch^{\beta} \qquad \text{... (12)}$$

With C a constant coefficient and β the elasticity of demand with respect to pressure. There is a clear resemblance between equations for leakage (equation 2) and demand elasticity (equation 12). The elasticity includes the effects of human behaviour, such as reacting to an increased pressure by opening taps less to obtain the same flow rate. In a study of water consumption patterns at a student village on the campus of the University of Johannesburg, Bartlett [16] found the indoor demand elasticity for pressure to be approximately 0.2. Outdoor water consumption such as garden irrigation is typically time-based rather than volume-based, meaning that a higher exponent can be expected for outdoor use. The typical exponent for outdoor irrigation equipment is around 0.5 [5, 16] although soaker hoses were found to have values as high as 0.75 [17].

In large systems it becomes likely that even minimum measured night flows will include some legitimate consumption. Since the combined ‘leakage exponent’ for outdoor and indoor consumption is likely to be less than 0.5, it may be concluded that measured leakage exponents in systems with demand are likely to underestimate the true leakage exponent of the system, provided that the level of demand in the measured night flows do not differ significantly.

6. CONCLUSIONS

The leakage exponent determined from field studies differ significantly from the theoretical orifice exponent of 0.5. The purpose of this paper has been to identify and analyse factors that may be responsible for the range of leakage exponents observed in the field. Leak hydraulics, pipe material behaviour, soil hydraulics and water demand were considered as possible causative factors. It is concluded that a significant proportion of background leakage can consist of transitional flow, and thus have a leakage coefficient value above 0.5 (although not above 1). Both experimental and theoretical investigations indicate that pipe material behaviour can provide one explanation for the observed range of leakage exponents.

The interaction between a leaking pipe and its surrounding soil is complex, and flow rates are unlikely to be a linear function of pressure, as a result of interaction of soil particles with the jet and the orifice, turbulent flow in the soil, the changing geometry of the unconfined flow regime, hydraulic fracturing and piping. Finally, if water demands are present in minimum night flows, the resulting leakage exponent is probably an underestimate of the true value.

References

1. WHO - WORLD HEALTH ORGANISATION. *Leakage management and control - a best practice manual*, 2001,WHO, Geneva.
2. FARLEY M. AND TROW S. *Losses in water distribution networks*, 2003, IWA Publishing, London.
3. IDELCHIK I. E. *Handbook of hydraulic resistance (3rd edition)*, 1994, Begell House, New York.
4. OFWAT - OFFICE OF WATER SERVICES. *Security of supply, leakage and the efficient use of water 2003-2004 Report,* 2004, OFWAT, Birmingham.
5. THORNTON J. and LAMBERT A. Progress in practical prediction of pressure: leakage, pressure: burst frequency and pressure: consumption relationships, *Proceedings of Leakage 2005*, 2005, IWA, London.
6. Greyvenstein, B., Van Zyl, J.E. (2007) An Experimental Investigation into the Pressure-Leakage Relationship of Some Failed Water Pipes, *Journal of Water Supply: Research and Technology – AQUA*, Accepted for publication.
7. CASSA A.M. *A numerical investigation into the behaviour of leak openings in pipes under pressure,* 2006, Master's degree thesis, Department of Civil Engineering Science, University of Johannesburg.
8. POPOV E.P. *Mechanics of materials (second edition)*, 1978, Prentice Hall, London.

9. WALSKI, T., BEZIS, W., POSLUSZNY, E.T., WEIR, M. AND WHITMAN, B. *Understanding the hydraulics of water distribution system leaks.* Proc. 6th ASCE/EWRI Annual Symp. on Water Distribution System Analysis, 2004, 27 June – 1 July, Salt Lake City.
10. NIVEN, R.K, KHALILI, N. In situ fluidisation by a single internal vertical jet, *Journal of Hydraulic Research*, 1998, **36**, No2, 199-229.
11. RAJARATNAM, N. (1982), Erosion by submerged circular jets, *Journal of Hydraulic Div. Am. Soc. Civ. Eng.*, 1982, **108**, No. 2, 262-267.
12. HANSON, G.J. Development of a jet index to characterize erosion resistance of soils in earthen spillways, *Trans ASAE*, 1991, **34**, No. 5, 2015-2020.
13. HARR M.E. *Groundwater and seepage*, 1962, McGraw Hill, New York.
14. BJERRUM L.J., NASH J.K.T.L., KENNARD R.M. and GIBSON R.E. Hydraulic fracturing in field permeability testing. *Geotechnique*, 1972, **22**, No.2, 319-332.
15. VAN ZYL J.E., HAARHOFF J. and HUSSELMANN M.L. Potential application of end-use demand modelling in South Africa, *Journal of the South African Institute of Civil Engineering*, 2003, **45** No. 2, 9 - 19.
16. BARTLETT L. *Pressure dependant demands in Student Town Phase 3*, 2004, Final Year Civil Engineering Project Report, Department of Civil and Urban Engineering, Rand Afrikaans University (now University of Johannesburg), South Africa**.**
17. CULLEN R. *Pressure vs consumption relationships in domestic irrigation systems*, 2004, Research thesis, Department of Civil Engineering, University of Queensland, Australia.

Table 1. Leakage exponents found in an experimental study by Greyvenstein [6]

Failure type	**Leakage exponent for pipe material**		
	uPVC	**Asbestos cement**	**Mild steel**
Round hole	0.52	-	0.52
Longitudinal crack	1.38 – 1.85	0.79 – 1.04	-
Circumferential crack	0.41 – 0.53	-	-
Corrosion cluster	-	-	0.67 – 2.30

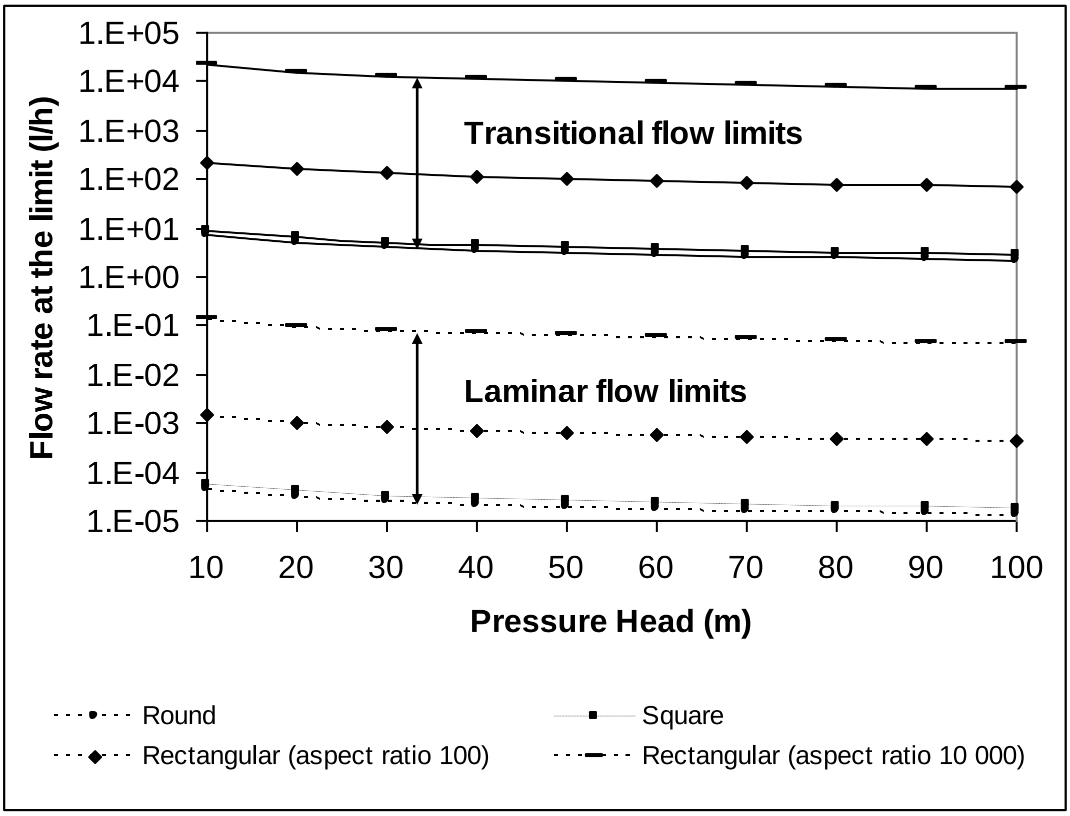

Fig 1. Maximum laminar and transitional flow rates for different types of leak openings