Hydraulic Design Of Bosspipe Lines

Waters & Farr Bosspipe offers a hydraulically smooth bore that provides excellent flow characteristics. Other advantages of Waters & Farr Bosspipe, like inert, corrosion free pipe material, excellent toughness and abrasion resistance, ensure excellent durability of the pipes.

Waters & Farr CivilBoss pipes for municipal drainage and sewerage applications are manufactured from polypropylene to AS/NZS 5065, Type B, ID series, in Classes SN16 (typically in stock) and SN8 (typically made to order). FarmBoss, a pipe for rural drainage applications, is manufactured from polypropylene or polyethylene, typically of SN6. All pipes are designated by nominal inside diameter, DN.

Flow of fluids in a pipe is a subject to resistance due to viscous shear stresses within the fluid and friction against the pipe wall, resulting in a pressure loss. A number of formulas, both theoretical and empirical, are used for flow calculations.

The **Colebrook-White formula** for the velocity of water in a smooth bore pipe under laminar conditions takes the form:

$$V = -2 \times \sqrt{2gDJ} \times \log\left\{ \frac{k}{3.7D} + \frac{2.51\nu}{D \times \sqrt{2gDJ}} \right\} \qquad \text{(HD-1)}$$

where
V – average flow velocity, m/s,
D – mean inside diameter of pipe, m,
J – hydraulic gradient (slope), m/m,
g – gravitational acceleration; g = 9.807 m/s² may be assumed,
ν – kinematic viscosity, m²/s; ν = 1.14 × 10^{-6} m²/s for water at 15°C (AS 2200, Table 1),
k – Colebrook-White roughness coefficient, mm.

Flow (discharge), Q, l/s:

$$Q = \frac{\pi \times D^2}{4} \times V \times 10^3 . \qquad \text{(HD-2)}$$

Polypropylene and polyethylene provide excellent resistance to abrasion and growth of slime. At the same time, hydraulic design of pipelines should allow for variations in flow during their long service life due to characteristics of the carried fluids.

For polypropylene and polyethylene pipes, NZS 4404 recommends the range of Colebrook-White roughness coefficient, k, of 0.003-0.015 mm for water and stormwater applications, k = 0.6 mm for wastewater applications.

The flow charts given in Figures 1 to 4 are developed based on values of k = 0.015 mm for water applications and k = 0.3 mm for wastewater applications.

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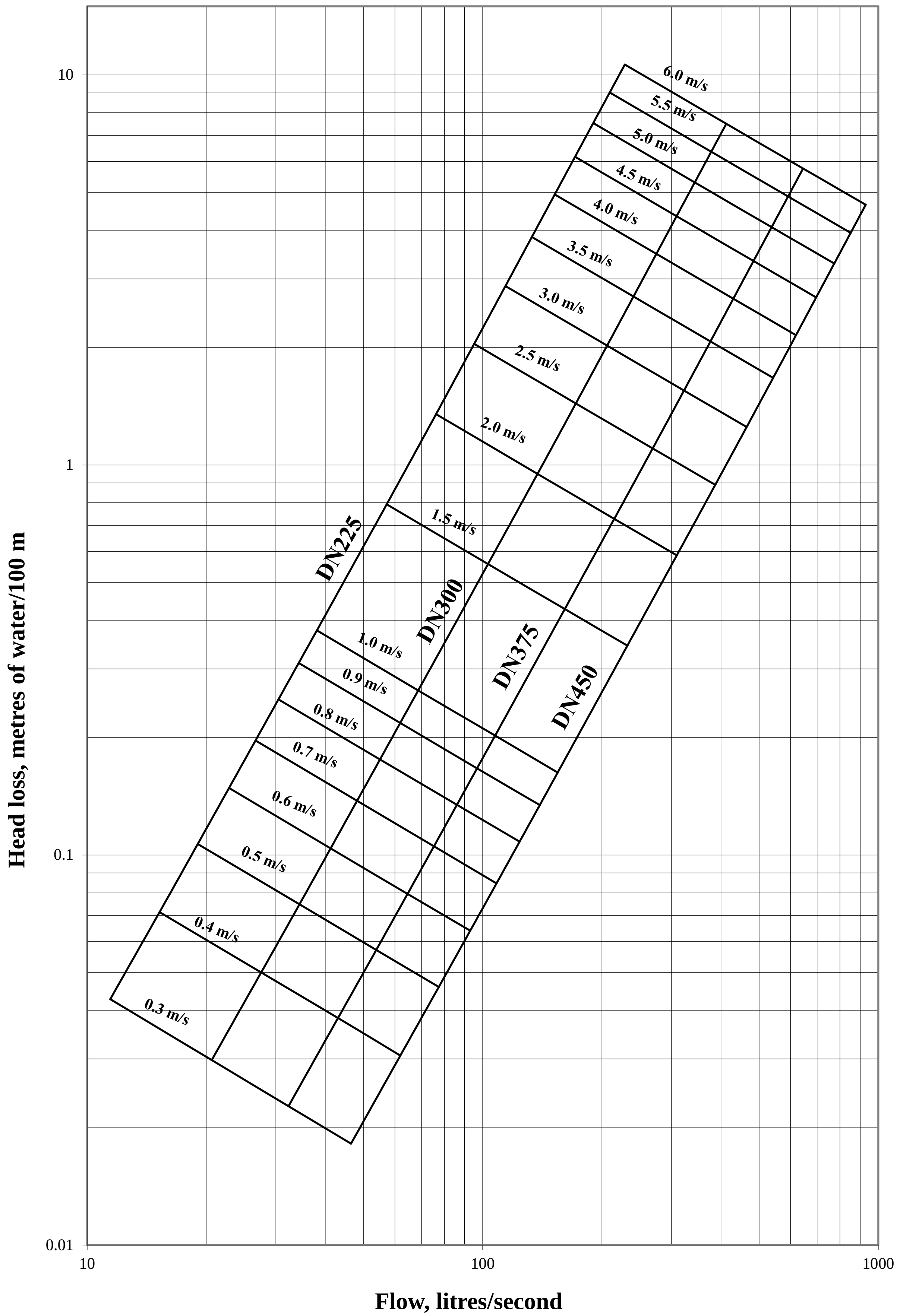


Fig. 1. Colebrook-White friction loss chart for SN16 CivilBoss pipelines for water and stormwater applications (running full bore; 15°C; k = 0.015 mm)

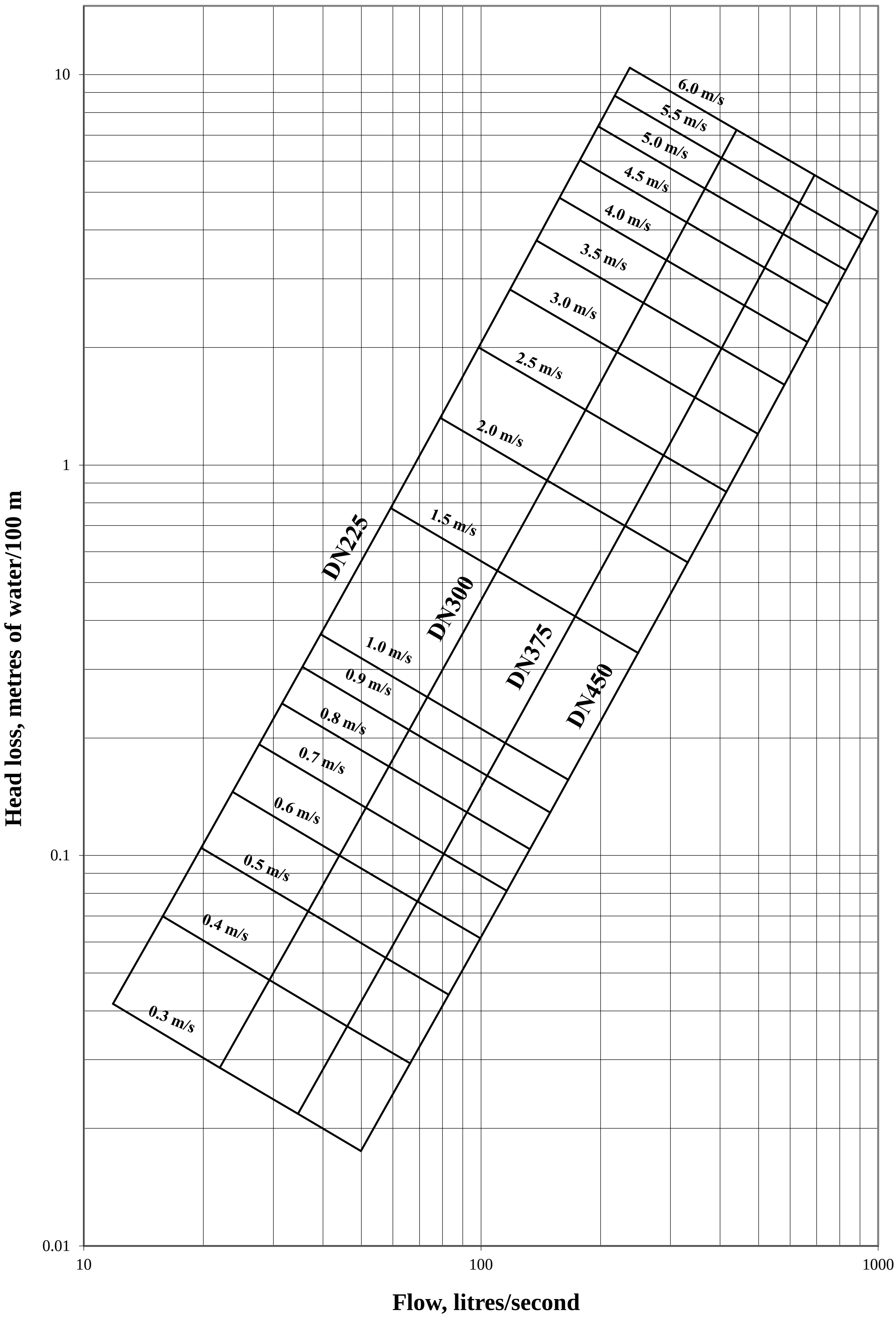


Fig. 2. Colebrook-White friction loss chart for SN8 CivilBoss and FarmBoss pipelines for water and stormwater applications (running full bore; 15°C; k = 0.015 mm)

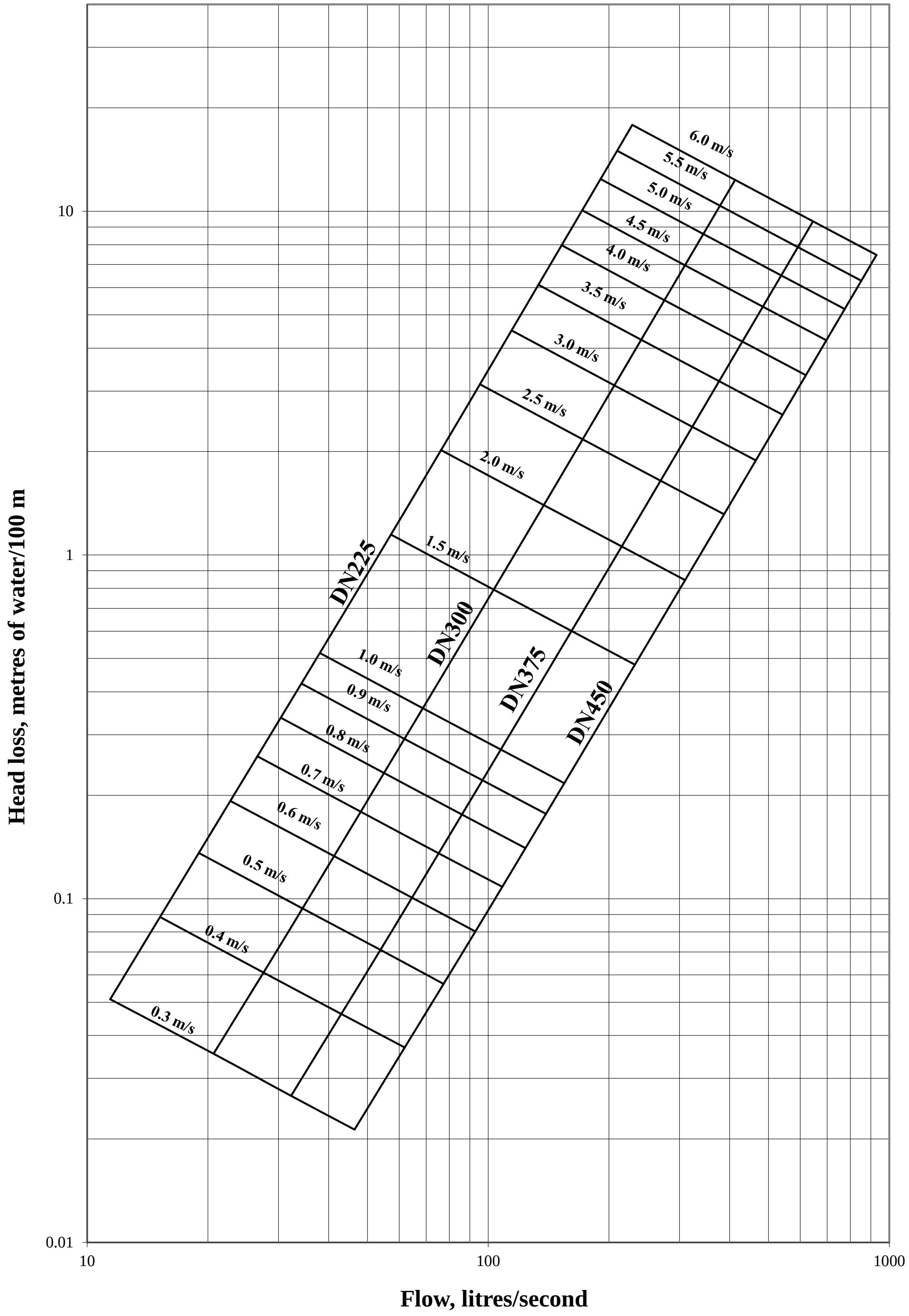


Fig. 3. Colebrook-White friction loss chart for SN16 CivilBoss pipelines for wastewater applications (running full bore; 15°C; *k* = 0.3 mm)

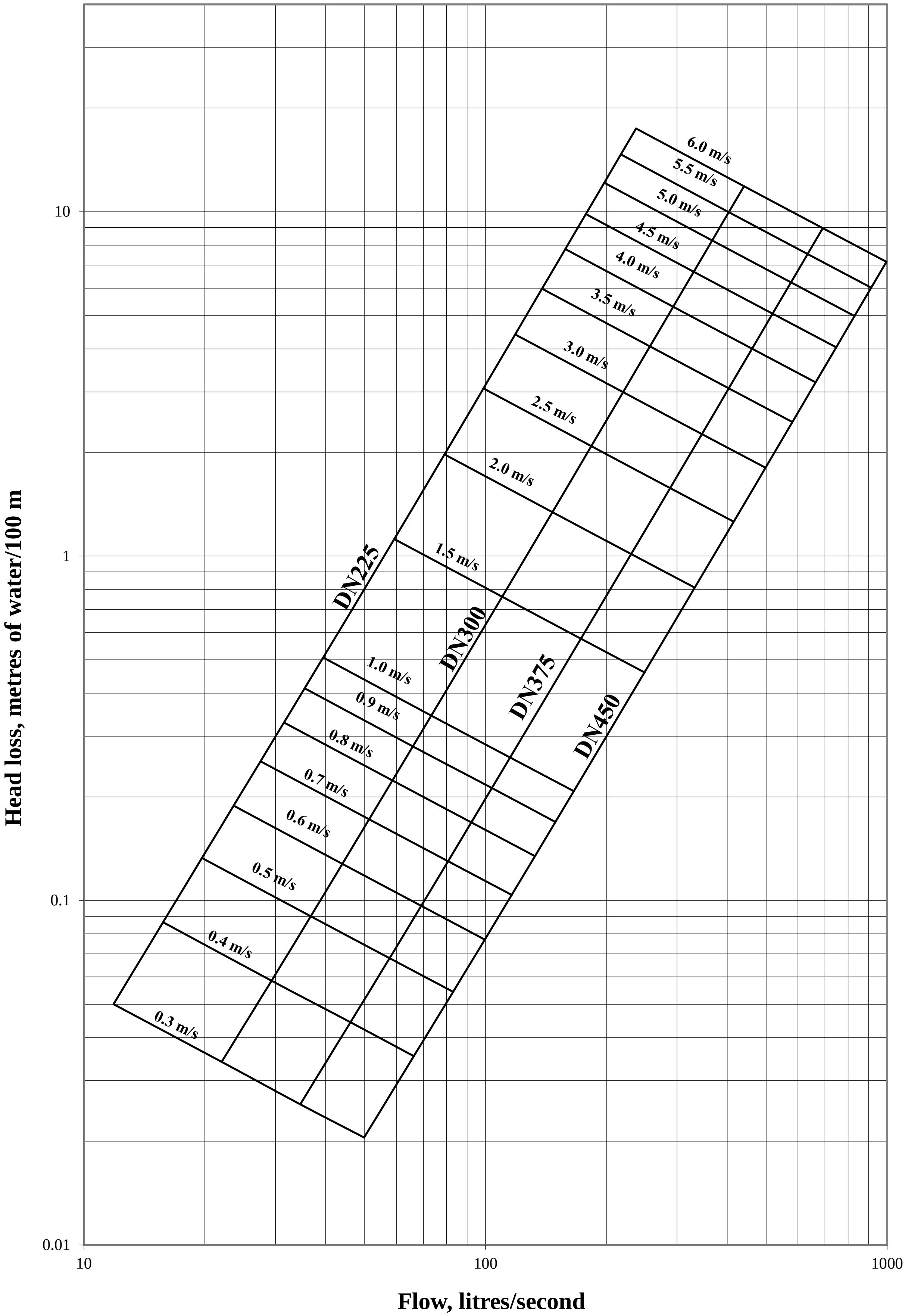


Fig. 4. Colebrook-White friction loss chart for SN8 CivilBoss pipelines for wastewater applications (running full bore; 15°C; *k* = 0.3 mm)

Changes in elevation provide for losses or gains in line pressure and have to be taken into account in hydraulic calculations. For liquids, the pressure loss (or gain) for a given elevation change may be calculated as follows:

$$H_E = h_1 - h_2 \,, \qquad \text{(HD-3)}$$

where H_E – elevation head loss, m,
h_1 – pipeline elevation at point 1, m,
h_2 – pipeline elevation at point 2, m.

The elevation points may be taken at the ends of the pipeline in case of uniform elevation change, or at the ends of each section of the pipeline that is a subject to uniform elevation change in case of several elevation changes (overall elevation head loss is then calculated by summing the sectional head losses).

If a pipeline has a high point significantly elevated in relation to both ends, accumulation of air or vacuum at the high point will affect the flow in the line (or lead to vacuum collapse of the pipe). Where such events are possible, appropriate vents should be incorporated in the pipeline design to prevent their occurrence.

The **Manning formula** for smooth bore pipe running full of water may be written in form:

$$V = \frac{1}{n} \times R^{2/3} \times J^{1/2}, \text{ or} \qquad \text{(HD-4)}$$

$$V = \frac{0.3950}{n} \times D^{0.67} \times J^{0.5} \text{ for circular pipes (AS 2200)} \qquad \text{(HD-5)}$$

where n – Manning roughness coefficient, dimensionless; for polypropylene and polyethylene pipes, the range of n is 0.008-0.009 for water and stormwater applications, 0.009-0.011 for wastewater applications (NZS 4404),
V – average flow velocity, m/s,
R – hydraulic radius, m; R may be calculated as follows:

$$R = \frac{A}{P}, \text{ or } R = \frac{D}{4} \text{ for circular pipes,} \qquad \text{(HD-6)}$$

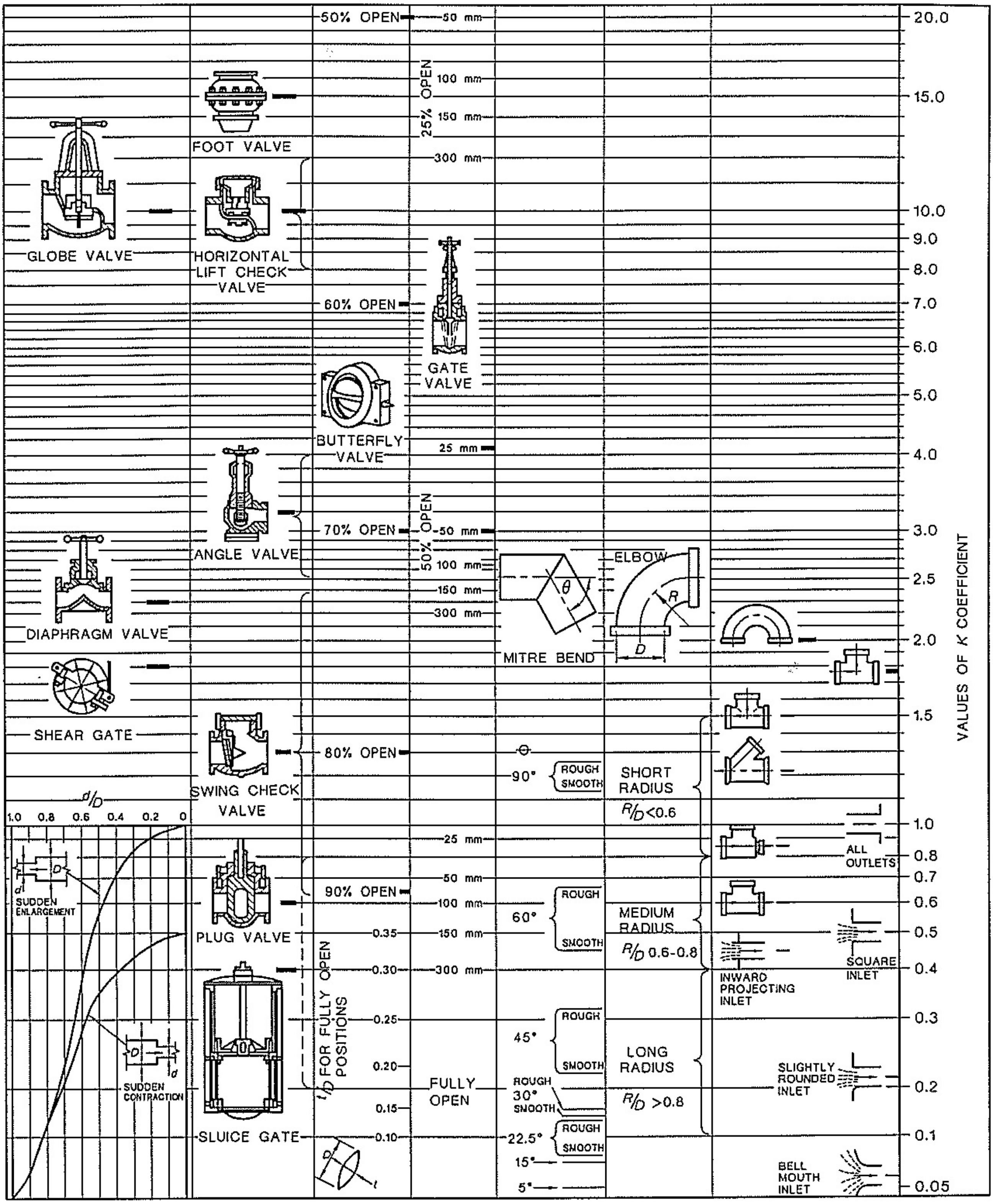
J – hydraulic gradient (slope), m/m,
A – flow cross-sectional area, m²,
P – wetted perimeter, m.

Design charts for hydraulic design of pipes using Manning formula (and examples) are given in AS 2200 and other literature.

Fittings and valves are causing additional friction losses to the flow of fluids in a pipeline. The Darcy-Weisbach equation modified for head losses in fittings and valves is:

$$H = k \times \frac{V^2}{2g}, \qquad \text{(HD-7)}$$

where H – head loss, m,
k – friction coefficient, dimensionless, dependent on type of fitting: see Fig. 5 displaying Chart 14 of AS 2200,
V – average flow velocity, m/s,
g – gravitational acceleration, m/s².



NOTES:

1 To obtain approximate head loss in metres multiply k by $V^2/2g$ (V = velocity in m/s, g = acceleration due to gravity in m/s^2).

2 All valves fully open unless otherwise indicated.

3 Brackets signify a range of values.

Fig. 5. Chart 14 of AS 2200: Resistance coefficients of valves and fittings

Note: For partially closed gate valve, k may be taken as: 1.0 for ¼ closed, 6.0 for ½ closed, 24.0 for ¾ closed.

Head losses at manholes are dependent upon relative manhole/pipe size, changes in flow direction and pipe sizes, flow depth, differences in inflow and outflow, relative position of inflow pipes and manhole benching. As a result, the value for K can vary between 0.2 to 2.5.

Due to change in flow velocity, **head loss at the pipe exit** should be taken into account:

$$H = k \times \frac{V^2 - V_d^2}{2g} \quad \text{(HD-8)}$$

where V_d – average flow velocity downstream of the outlet, m/s.

In usual circumstances, flow in drainage and sewer pipelines is only **partial** (Fig. 6). Proportional (in relation to full pipe flow) average velocity as calculated by Manning formula (HD-4), proportional discharge as well as proportional hydraulic radius and utilised area at various filling levels (expressed as a ratio of height and inside diameter), are given as a chart on Fig. 7.

Flow of water in partially filled pipes may also be calculated using Colebrook-White formula (HD-2), where mean internal diameter, D, is replaced by a value ($4R$), R being hydraulic radius, m (formula HD-9):

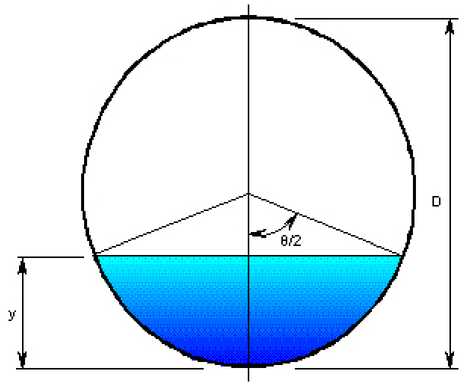


Fig. 6. Partially filled pipe

$$V = -2 \times \sqrt{2g \times 4R \times J} \times \log\left\{\frac{k}{3.7D} + \frac{2.51\nu}{D \times \sqrt{2g \times 4R \times J}}\right\}. \quad \text{(HD-9)}$$

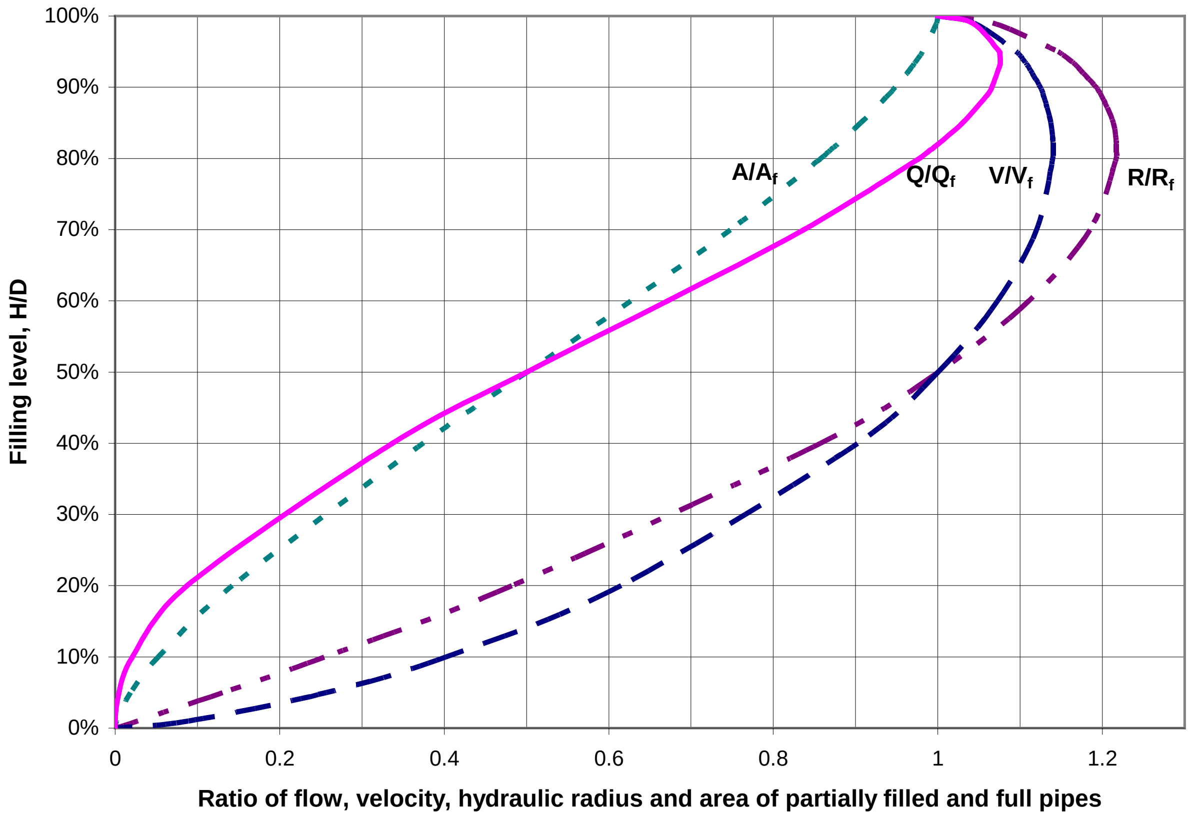


Fig. 7. Proportional flow in partially filled pipes.

Sewer pipeline design is often based on average daily dry weather flow (ADWF) of 225 litres per capita per day and peak wet weather flow/ADWF ratio of 4 (which depends on system design in respect of storm water inflow).

Sewers and storm water drains are also designed to ensure minimum self-cleansing flow at least once daily. Traditionally it is regarded that self-cleansing velocity is achieved at 0.6-0.75 m/s. More accurate design for part full pipes is based on theory of fluid flow in open channel. Bottom (boundary) shear stress equation may be expressed as:

$$\tau = \rho g R J \tag{HD-10}$$

where
- τ – boundary shear stress, N/m²,
- ρ – fluid density, kg/m³; for water, ρ = 1000 kg/m³ may be assumed,
- g – gravitational acceleration, m/s²,
- R – hydraulic radius, m,
- J – hydraulic gradient (slope), m/m.

For circular sewer pipe, the required gradient may be calculated by the following equation:

$$J = \frac{4\tau}{\rho g D (R/R_f)} \tag{HD-11}$$

where
- D – mean internal diameter of pipe, m,
- R/R_f – ratio of hydraulic radius for part full and full pipe. For known daily peak flow in dry weather and full pipe flow (see Figure 7).

A value of $\tau \geq 1.5$ N/m² is generally regarded as a criterion of self-cleansing flow of water ensuring separation of organic solids. Then the minimum slope for self-cleansing flow is:

$$J_{min} \approx \frac{6 \times 10^{-4}}{D(R/R_f)} \tag{HD-12}$$

The resulting minimum slopes as a function of Bosspipe filling level during daily dry weather peak flow are given on Figure 8.

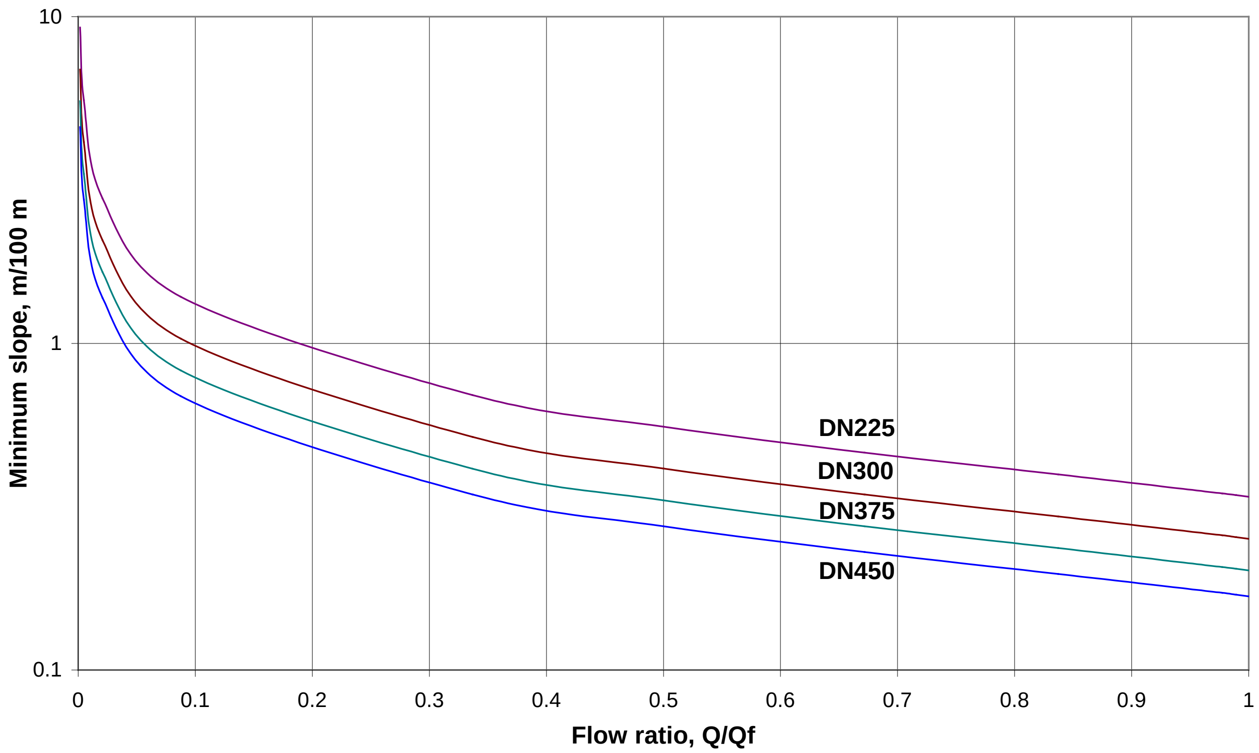


Fig. 8. Minimum self-cleansing slope for SN16 Bosspipe as a function of ratio of daily dry weather peak flow (partially filled pipe) and full pipe flow.