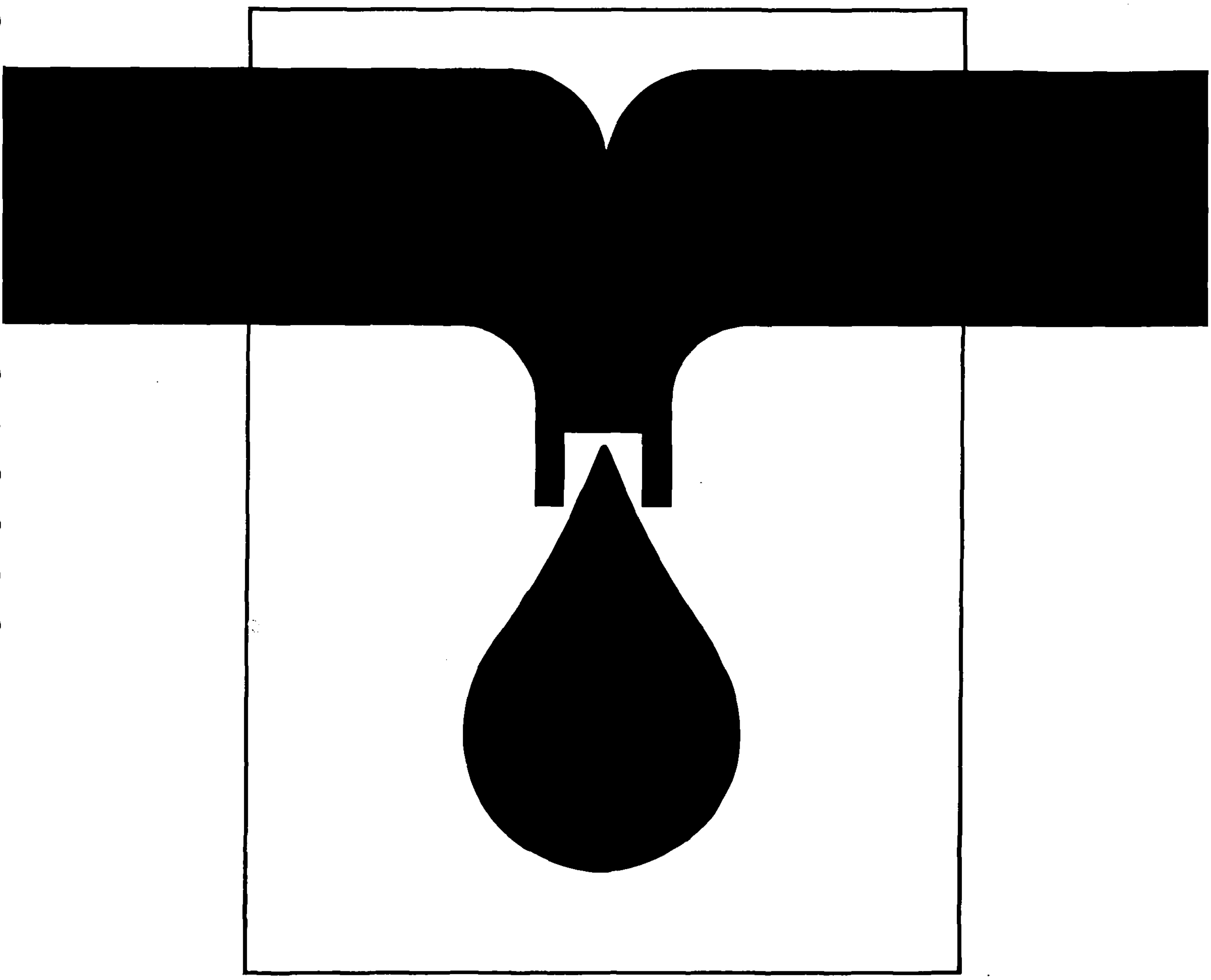
TRAINING MODULES
FOR
WATERWORKS PERSONNEL



Basic Knowledge

0.4

Basic principles of water transport

Foreword

Even the greatest optimists are no longer sure that the goals of the UN "International Drinking Water Supply and Sanitation Decade", set in 1977 in Mar del Plata, can be achieved by 1990. High population growth in the Third World combined with stagnating financial and personnel resources have led to modifications to the strategies in cooperation with developing countries. A reorientation process has commenced which can be characterized by the following catchwords:

- use of appropriate, simple and – if possible – low-cost technologies,
- lowering of excessively high water-supply and disposal standards,
- priority to optimal operation and maintenance, rather than new investments,
- emphasis on institution-building and human resources development.

Our training modules are an effort to translate the last two strategies into practice. Experience has shown that a standardized training system for waterworks personnel in developing countries does not meet our partners' varying individual needs. But to prepare specific documents for each new project or compile them anew from existing materials on hand cannot be justified from the economic viewpoint. We have therefore opted for a flexible system of training modules which can be combined to suit the situation and needs of the target group in each case, and thus put existing personnel in a position to optimally maintain and operate the plant.

The modules will primarily be used as guidelines and basic training aids by GTZ staff and GTZ consultants in institution-building and operation and maintenance projects. In the medium term, however, they could be used by local instructors, trainers, plant managers and operating personnel in their daily work, as check lists and working instructions.

45 modules are presently available, each covering subject-specific knowledge and skills required in individual areas of waterworks operations, preventive maintenance and repair. Different combinations of modules will be required for classroom work, exercises, and practical application, to suit in each case the type of project, size of plant and the previous qualifications and practical experience of potential users.

Practical day-to-day use will of course generate hints on how to supplement or modify the texts. In other words: this edition is by no means a finalized version. We hope to receive your critical comments on the modules so that they can be optimized over the course of time.

Our grateful thanks are due to

Prof. Dr.-Ing. H. P. Haug
and
Ing.-Grad. H. Hack

for their committed coordination work and also to the following co-authors for preparing the modules:

Dipl.-Ing. Beyene Wolde Gabriel
Ing.-Grad. K. H. Engel
Ing.-Grad. H. Hack
Ing.-Grad. H. Hauser
Dipl.-Ing. H. R. Jolowicz
K. Ph. Müller-Oswald
Ing.-Grad. B. Rollmann
Dipl.-Ing. K. Schnabel
Dr. W. Schneider

It is my sincere wish that these training modules will be put to successful use and will thus support world-wide efforts in improving water supply and raising living standards.

Dr. Ing. Klaus Erbel
Head of Division
Hydraulic Engineering,
Water Resources Development

Eschborn, May 1987

Basic principles of water transport

1 Foreword, concepts and symbols used

1.1 Foreword

Module 0.4 is divided into two parts. Part 1 (heading 2) discusses propositions and formulae of theoretical hydraulics (hydraulics is the study of the flow of fluids) and on which the practical hydraulic engineering of water supply systems is based.

Part 2 (heading 3) is then concerned with the application of this theoretical knowledge. The aim in this section is to enable the practically trained skilled pipe fitter or water technician to carry out simple calculations concerning the dimensioning of water mains or to check the correct sizing of existing pipes independently, without necessarily completely mastering the theoretical basis discussed under heading 2.

Appendix 1 contains tables and charts giving the head losses for the most commonly used pipe diameters.

1.2 Concepts and symbols used

l = length (m)
d = diameter (m)
s = second (s)
v = velocity (m/s)
g = acceleration of free fall (m/s^2)
p = pressure per unit area (N/cm^2)
= specific gravity
= density
A = cross-sectional area (m^2, cm^2)
Q = volume of flow (l/s)
P = force (N)
E = energy
M = mass (kg)
V = volume (l, m^3)

mH_2O = height of water column ($10\ mH_2O = 1$ bar)
H_{ST} = hydrostatic head
H_D = hydraulic head
H_{man} = manometric delivery head
h_p = pressure head
h_v = velocity head
h_F = friction head
h_z = altitude
m.s.l. = mean sea level
K = roughness factor
J = gradient (m/km)

2 Theoretical basis - hydrodynamics -

2.1 Physical properties of fluids in motion

Liquids offer only a low resistance to alterations of their shape; i.e. they take up the shape of any vessel in which they are placed virtually without the exertion of pressure - normal ambient pressure is sufficient. On the other hand, liquids resist any rapid acceleration or sudden deceleration with a force of resistance (inertia) equivalent to their mass. The energy of a moving liquid can, for instance, be transformed via turbines into electrical energy.

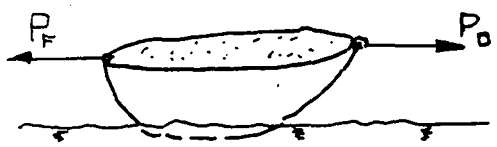
The concepts pressure, acceleration, deceleration and mass are all connected to the concept of force (P):

Force P = mass M x accerleration a (deceleration)	2.1.1
Force P = pressure p x area A	2.1.2

Mention has been made above of the relatively low resistance which has to be overcome to start a liquid moving slowly, i.e. to induce it to flow.
Every liquid possesses an inner resistance to flow which is know as viscosity. Viscosity manifests itself as a

force which opposes the movement of the liquid molecules.

This force, called fluid friction or viscous drag (P_F), continuously retards the movement of a liquid or of a body moving through a liquid. A force at least equal to P_F is necessary to drive e.g. a boat with uniform velocity through a liquid.



Driving force P_D = retarding force P_F

Fig. 1: Driving/retarding force

Equally, if a plate floats on a thin layer of liquid and is to be moved, the viscous drag must be overcome with force P_F.

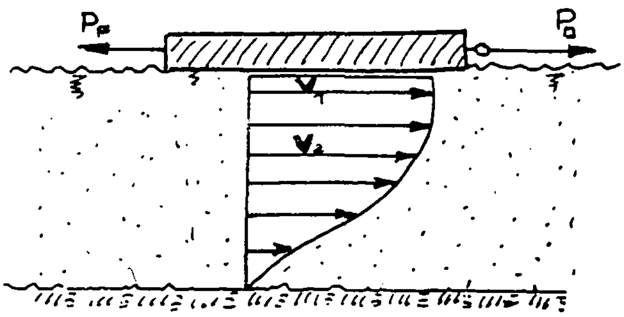


Fig. 2: Distribution of velocities

Due to adhesive force, the liquid molecules directly adjacent to the plate and to the bottom of the vessel cling to these surfaces. For this reason, the upper layer of liquid moves with the same velocity as the moving plate, whilst the layers underneath have increasingly lower velocities, down to the bottom layer, which remains stationary. Each faster layer attempts to pull the next slower layer along with it. The sum of the forces required thus gives the fluid friction which is equal to the force P_D necessary to move the plate with uniform velocity v over the layer of liquid. The decrease of the velocities towards the bottom is not linear, but roughly as shown in fig. 2. The exact shape of the curve and, equally, the magnitude of the required force P_D depend on the viscosity of the fluid in question, i.e. its property of resistance to flow. The viscosity of alcohol and of water is low, for instance, whereas that of fuel oil, lubricating oil and tar is progressively higher.

2.2 Flow of liquid through a pipe

Whenever a force (pressure) exerted on a liquid is greater than its fluid friction, the liquid begins to move. The liquid molecules move away from the pressure in all directions, unless they are contained and forced in one direction only through a pipe.

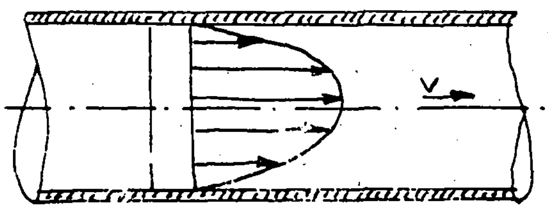


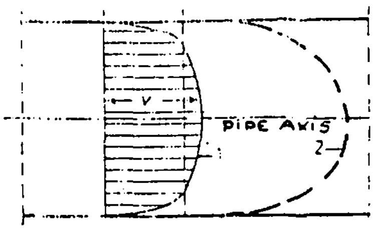
Fig. 3: Distribution of velocities in a pipe

As in the example above, the distribution of velocities over the pipe's cross-sectional area is not linear, and decreases sharply in the proximity of the pipe wall, since the forces of adhesion holding the molecules to the wall are not overcome.

Even at very high flow velocities, the velocity in the direct proximity of the pipe wall still drops to zero.

The fluid reaches its highest velocity along the axis of the pipe, the velocity decreasing with relative proximity to the pipe wall until it equals zero directly next to it. The molecules retain their position in the flowing liquid, i.e. a molecule in the centre will continue to travel along the pipe axis, whilst a molecule at the outer edge will be left far behind on its much slower path. The molecules move through the pipe in streamlines; i.e. along parallel paths in the direction of the fluid velocity at that point. Streamlines cannot intersect and have no kinks. They can be made visible in a glass tube with the aid of light particles (e.g. sawdust) which follow them. The particles follow their own streamline even round bends in the tube. This flow of a fluid in parallel layers, where the liquid particles remain within their own layer, is also called laminar flow. The aggregate of streamlines at any moment forms the flow pattern. All streamlines are bound by the wall of the pipe; where the pipe widens they fan out, where it narrows they close together. A steady flow is given where both the physical shape of the pipe remains the same (i.e. the cross-sectional area is not altered by changing the position of sluice valves, etc.) and the velocity to the fluid flow is constant (i.e. the prevailing conditions remain the same).

Above a certain point, known as the "critical velocity", the motion of the fluid changes from laminar to turbulent. Turbulent flow is characterized by the fact that the distribution of velocities is no longer parabolic, but instead the velocity of all fluid particles is virtually the same over the complete cross-sectional area.



1 = Turbulent flow (flat curve)

2 = laminar flow (parabolic curve)

Fig. 4: Laminar and turbulent distribution of velocities

Thus a steady flow can be either laminar or turbulent, depending on its velocity. In a turbulent flow, the molecules leave their streamlines (paths), however, which leads to collisions and reciprocal exchanges of momentum. The result is the virtually uniform speed of all molecules over the pipe cross-section (but still 0 at the wall!), since the faster particles lose velocity due to the collisions, whereas the slower particles are accelerated by them.

2.3 Equation of continuity

Liquids are only very slightly compressible, even at very high pressures. Gases, in contrast, are highly compressible. Thus, in the following, liquids are considered to be incompressible.

This still applies in any pipe of which the diameter widens and narrows, provided no water enters or leaves the pipe along its complete run.

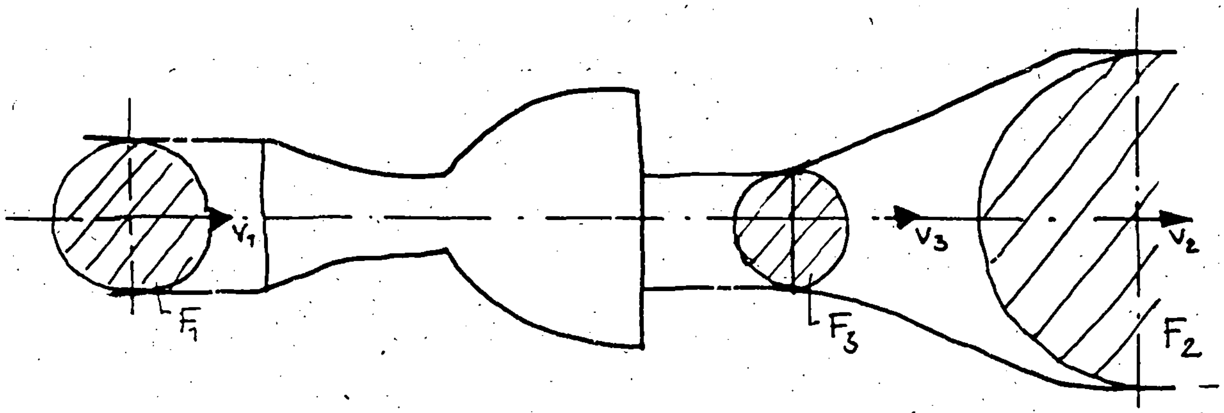


Fig. 5: Equation of continuity

The inflowing volume of water 1 is equal to the outflowing volume of water 2, whereby the same volume of water (e.g. 3) must flow through any cross-sectional area of the pipe within the same period of time. Clearly, where the cross-sectional area is large, the velocity of flow (v_2) must be low, and conversely, where the cross-sectional area is smaller, the flow velocity (v_3) must be higher, since the volume of water flowing per second through a stream tube is constant, provided the flow is steady.

$$\underbrace{A_1 x h_1}_{\text{volume 1}} \div t = \underbrace{A_2 x h_2}_{\text{volume 2}} : t = \underbrace{A_3 x h_3}_{\text{volume 3}} \div t = \text{constant} \qquad 2.3.1$$

The flow volumes (Q in m^3 per unit of time) are equal.

Where the areas are greater, the cylinder heights h are smaller in order to contain the same volume. The distances s covered by the molecules can be substituted for the heights h, and the equation re-formulated as follows:

$$A_1 \times \frac{s_1}{t} = A_2 \times \frac{s_2}{t} = A_3 \times \frac{s_3}{t} = \text{constant} \qquad 2.3.2$$

The distance s per time unit is the velocity prevailing in each cross-sectional area (e.g. $v_1 = \frac{s_1}{t}$)

$$A_1 \times v_1 = A_2 \times v_2 = A_3 \times v_3 = \text{constant} \qquad 2.3.3$$

The equation of continuity states that v is inversely proportional to A and that their product is therefore constant.

2.4 Bernoulli's equation

If it assumed that a fluid is non-viscous, the relationships given below can be determined. (A "perfect fluid" has zero viscosity: water has a low viscosity and can therefore be considered as approaching this state.)

Flowing water contains energy. Thus, assuming there is no fluid friction, it is true to say that the inflowing water possesses the same energy as the outflowing water, or as the water flowing through any arbitrary cross-sectional area of the pipe. In a water mains system, there is a difference in level. This causes the water to begin to flow.

Force P = mass M x acceleration of free fall g 2.4.1

Energy E = force P x distance s = constant 2.4.2

Energy E = volume V x density ρ x acceleration a x distance s = constant

$E = V \times \rho \times a \times s = \text{constant}$ 2.4.3

Further, the law of conservation of energy always applies:

The entire energy of the loss-free system is conserved; i.e. the sum of the kinetic energy E_{kin} (flowing water) plus potential energy E_{pot} plus pressure energy E_{press} is constant.

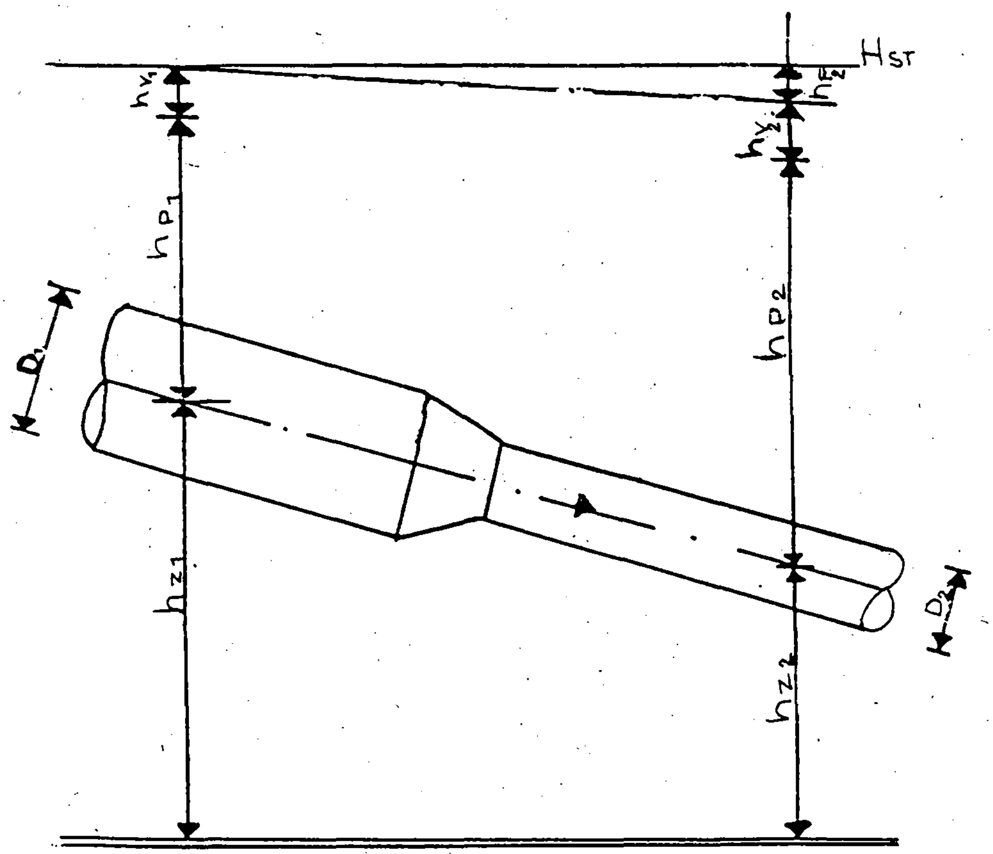


Fig. 6: Conservation of energy (Bernoulli's equation)

Kinetic energy $\boxed{E_{kin} = \frac{M}{2} v^2}$ 2.4.4

Potential energy E_{pot} = weight x height Z 2.4.5

Weight = mass M x g 2.4.6

(g - acceleration of free fall in $\frac{m}{s^2}$)

$\boxed{E_{pot} = M \times g \times Z}$ 2.4.7

Pressure energy performs displacement work, since a force (weight of the water) moves the water along through the distance s.

E_{press} = force x distance 2.4.8

Force = pressure p x area A 2.4.9

E_{press} = pressure p x $\underbrace{\text{area A x distance s}}_{\text{volume transported}}$ 2.4.10

E_{press} = pressure p x volume V 2.4.11

Force = weight = volume V x specific gravity γ

Volume V = $\frac{\text{weight}}{\text{specific gravity } \gamma}$ 2.4.12

E_{press} = pressure p x $\frac{\text{weight}}{\text{specific gravity } \gamma}$ 2.4.13

Weight = mass M x acceleration of free fall 2.4.14

E_{press} = pressure p x $\frac{\text{mass M x acc. of free fall g}}{\text{specific gravity } \gamma}$

$\boxed{E_{press} = p \times \frac{M \times g}{\gamma}}$ 2.4.15

$\boxed{E_{kin} + E_{presas} + E_{pot} \quad \text{constant}}$ 2.4.16

Law of the conservation of energy

$$\frac{M}{2} v^2 + p \frac{M \times g}{\gamma} + m \times g \times Z = \text{constant} \qquad 2.4.16$$

$$\frac{v^2}{2} \times \frac{p \times g}{\gamma} + g \times Z = \text{constant} \qquad 2.4.16$$

$$\boxed{\frac{v^2}{2g} \times \frac{p}{\gamma} + Z = \text{constant}} \qquad 2.4.16$$

<u>Bernoulli's equation</u>

Height Z in m

Velocity v in m/s

Pressure p in kp/m^2

Acceleration of free fall g in m/s^2

Specific gravity γ in kp/m^3

$$\left[\frac{m^2 \times s^2}{s^2 \times m}\right] \left[\frac{kp \times m^3}{m^2 \times kp}\right] \left[m\right]$$

m	m	m
velocity head h_v	pressure head h_p	altitude h_z

Velocity head + pressure head + altitude = constant   2.4.17

$$h_v + h_p + h_z = \text{constant}$$

Since the energy of the raised water is proportional to the height H, the heads as given above can be inserted into Bernoulli's equation and the formula expressed as follows:

$$h_{v_1} + h_{p_1} + h_{z_1} = h_{v_1} + h_{p_2} + h_{z_2} = \text{constant} \qquad 2.4.17$$

In reality, in natural flowing liquids energy losses are caused by friction of the molecules against each other and against the interior pipe wall. These losses lead to an alteration of the energy head h along the flow path of the liquid. The energy losses due to friction at position 1 are negligible. Thus the head loss h_F does not appear in the first expression. For this reason, a friction head h_F is added to Bernoulli's equation, and the extended equation then appears as:

$$h_{v_1} + h_{p_1} + h_{z_1} = h_{v_2} + h_{p_2} + h_{z_2} + h_{F_2}$$

2.5 Velocity of flow in a pressure pipe

$$\boxed{\frac{v^2{}_1}{2g} + \frac{p_1}{\rho \times g} + z_1 = \frac{v^2{}_2}{2g} + \frac{p_2}{\rho \times g} + z_2}$$

Bernoulli's equation

$$\frac{v^2{}_2}{2g} = \frac{v^2{}_1}{2\ g} + \frac{p_1}{\rho \times g} - \frac{p_2}{\rho \times g} + \underbrace{z_1 - z_2}_{H} \qquad 2.5.1$$

$$v_2 = \sqrt{v^2{}_1 + \frac{2}{\rho \times g}\ (p_1 - p_2) + 2g\ H} \qquad 2.5.2$$

If the pressure pipe is fed from a large tank, the velocity of the water is extremely low, due to the large cross-sectional area A_1 (i.e. $v_1 \approx 0$; equation of continuity).
The formula of velocity v_2 in cross-sectional area A_2 is thus simplified:

$$v_2 = \sqrt{\frac{2}{\rho}\ (p_1 - p_2) + 2\ g\ H} \qquad 2.5.3$$

If the tank is not under pressure and position 2 is an outlet, virtually the same ambient pressure obtains at both positions, and $p_1 = p_2 = 0$. Formula 2.5.3 is thus further simplified, and the outflow velocity (or through the narrow pipe with A_2) is:

$$\boxed{v_2 = \sqrt{2gH}} \qquad 2.5.4$$

2.6 Suction pressure

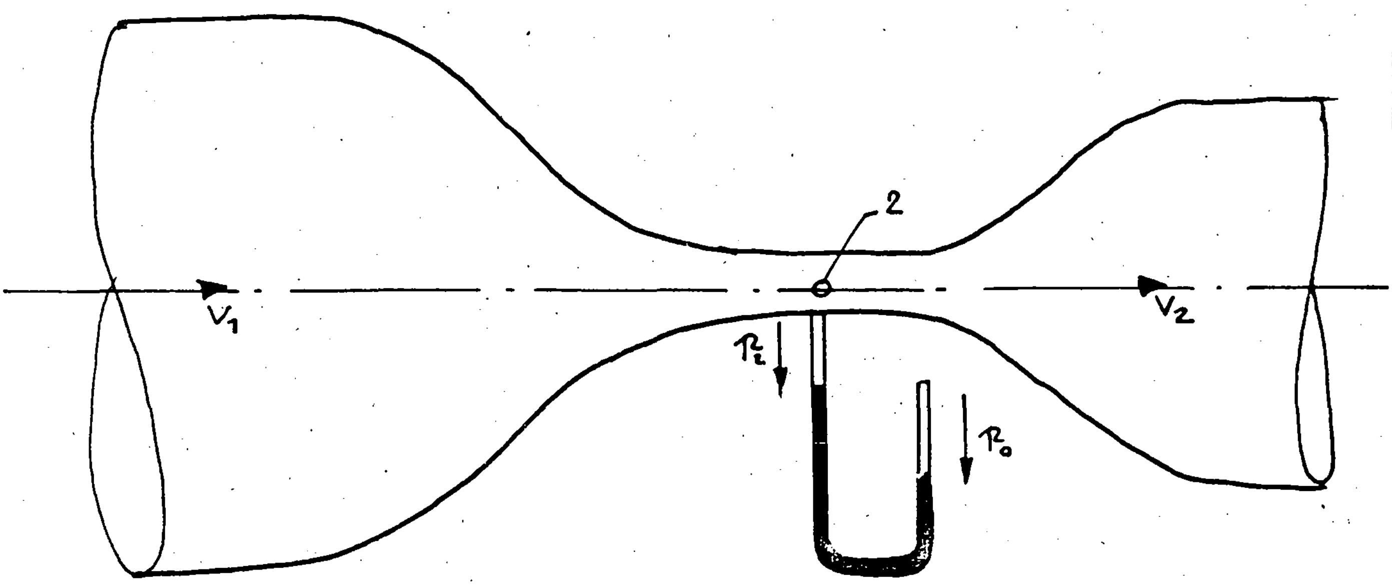


Fig. 7: Suction pressure

$$\boxed{\frac{v_1^2}{2g} + \frac{p_1}{\gamma} + Z_1 = \frac{v_2^2}{2g} + \frac{p_2}{\gamma} + \frac{Z_2}{2g}}$$ Bernoulli's equation

In a pipe wich has no differences in altitude - $Z_1 = Z_2$ - the equation is simpler:

$$\frac{v_1^2}{2g} + \frac{p_1}{\gamma} = \frac{v_2^2}{2g} + \frac{p_2}{\gamma} \qquad 2.6.1$$

(Acceleration of free fall g and specific gravity are constants.)

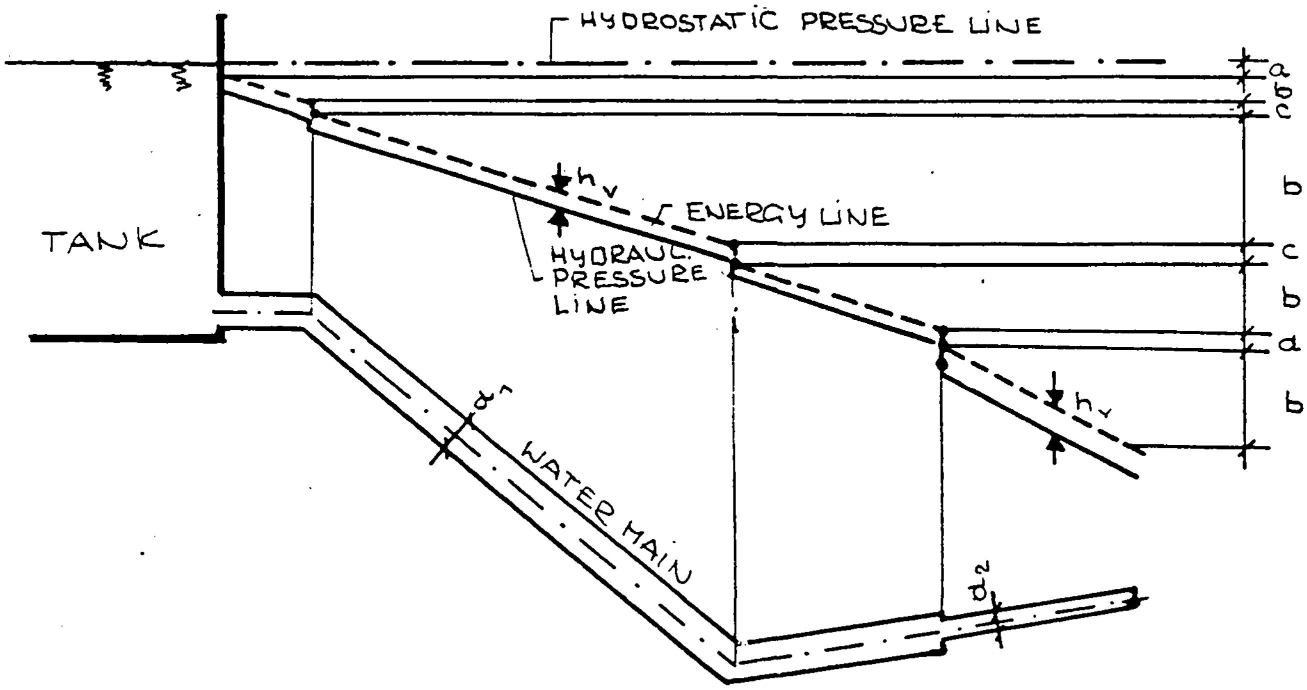
If the cross-sectional area of the pressure pipe is considerably reduced at position 2, the velocity v_2 will increase considerably as v_2 is raised to the second power, a high figure will be obtained for thc addend $\frac{v_2^2}{2g}$) with only a very small share remaining for the augend

$\frac{p_2}{\gamma}$ in order to satisfy the equation. The pressure p_2 in the constricted zone will then drop below the ambient pressure, producing negative (i.e. suction) pressure. If there were a small opening in this zone, no water would flow out but instead air would flow in. A mixed fluid could also be sucked in, however, through a thin feed pipe. If the opening where too big, ambient pressure would immediately re-establish itself in the constricted zone and the velocity v_2 drop proportionately in order to satisfy the equation.

3 Water transport in pressure pipes

3.1 Hydraulics of water mains

The drawing below and the following notes indicate the losses of head sustained in a pressure pipe connected to a high-level storage tank.



Losses of head

h_v = velocity head

a = head loss at pipe entry

b = friction losses in the pipe

c = losses at elbows

d = loss due to alteration of the cross-sectional area

Fig. 8 : Heads and head losses

Fig. 8 shows a pressure pipe connected to a high-level storage tank. When the pipe is full and there is no consumption, i.e. no movement of water through the main, hydrostatic pressure prevails along the complete pipe length. The level of the hydrostatic pressure depends on the depth of water in the tank.

In the drawing, the hydrostatic pressure is indicated by the hydrostatic pressure line.

If water is drawn off the main or an outlet sluice valve opened, water begins to flow and a transformation of energy occurs which leads to losses of pressure or pressure head.

The reduction of the hydrostatic pressure head and the pressure losses give the hydraulic pressure or pressure head. This is the pressure exerted by the water against the inner pipe wall.

Hydraulic pressure alters with the volumetric flow through the main. Large volumes of flow result in high losses of head and thus a reduction of the hydrostatic pressure.

For the water to flow through the pipe, the hydraulic pressure must always be higher than the apex of the pipe run.

The following separately identifiable losses of head occur:

1. To produce and maintain the velocity of the fluid in the pipe, a velocity head $h_{v_1} = \frac{v^2}{2g}$ is necessary.

In closed pipes with an invariable diameter, the velocity of flow is the same for every constant volume of water and thus the difference between velocity heads equals zero. The loss due to velocity head can be neglected for rough dimensioning of pressure pipes.

2. As the water enters the pipe, a slight loss of pressure head occurs, its magnitude depending on the shape of the pipe entrance. This is also generally ignored in initial rough calculations.

3. Loss of pressure head due to friction ("friction head", "hydraulic friction") h_{F_1}, caused by friction between the molecules and between the fluid and the pipe wall. Friction heads always have a considerable effect on the hydraulic pressure line and must therefore be given adequate consideration in dimensioning water mains.

4. Pipe elbows and components such as sluice valves, fittings, water meters also cause losses of pressure head, h_{F_2}, which vary according to the size and type of the part.

3.2 Pumping of water

Where there is no natural gradient, or the available gradient is inadequate, the pressure required to pipe water to the consumers must be produced by pumping.

The example below shows a hydraulic calculation for pumping water from a tank containing treated water to a high-level storage tank.

The pump is positioned here at a higher level than the surface of the water in the treated-water tank: this means that a suction pump has to be used. In practice, it is always preferrable to install the pump, wherever possible, at a point lower than the water surface, to avoid use of the normally less robust suction pumps.

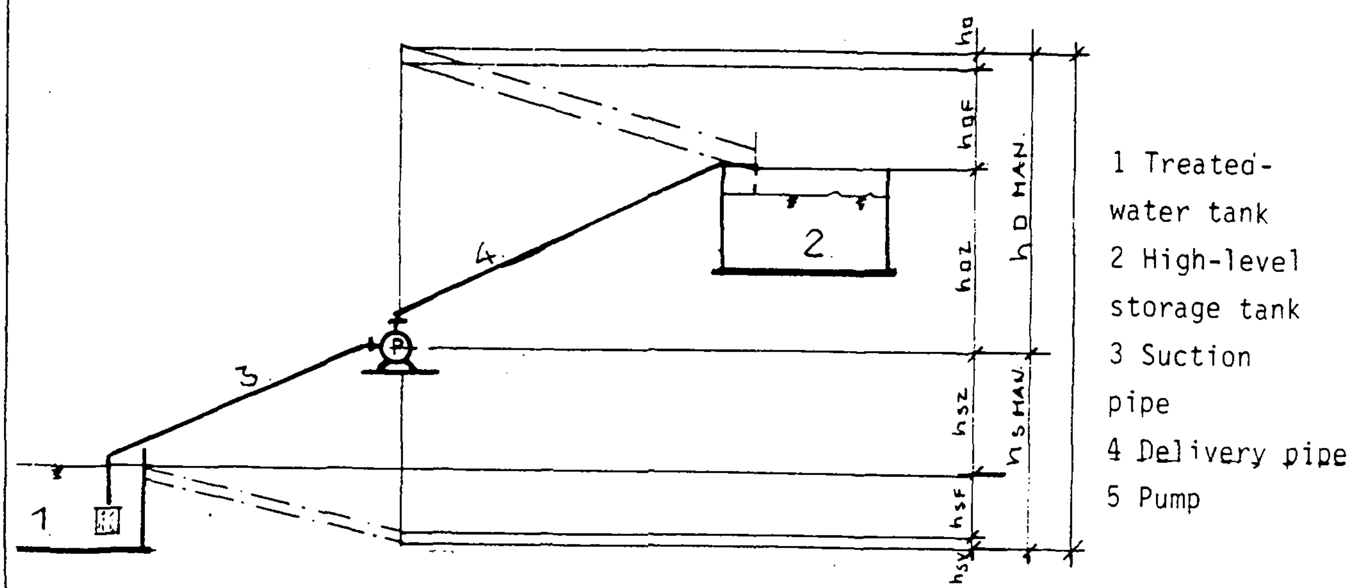


Fig. 9: Suction and pressure heads in pumping

Key to the notation used:

h_{sz} = geodetic suction head (difference in height between the free water level in the treated-water tank and the pump)

h_{dz} = geododetic delivery head (difference in height between the pump and the water surface in the high-level tank)

h_{sF} = friction head in the suction pipe

h_{dF} = friction head in the delivery pipe

h_{sv} = velocity head in the suction pipe

The manometric suction head $h_{s\ man}$ which the pump has to achieve is calculated by adding the difference in height between the lowest water level in the treated-water tank and the suction branch of the pump to the head losses due to friction and velocity

$$h_{s\ man} = h_{sz} + h_{sF} + h_{sv}$$

The suction head (also called "static suction lift") of a pump is limited. The suction operation depends largely on the extent to which air can be evacuated from the suction pipe so that atmospheric pressure can force water up it.

Theoretically, the maximum suction head, which depends on the temperature of the water and on atmospheric pressure, is 10.33 m.

In practice, this theoretical suction head is not achievable. Pumps used in water-supply systems lift between 5.00 and 7.00 m.

The manometric delivery head $h_{d\ man}$ (a determinative quantity for dimensioning the pumps) is calculated by adding the geodetic delivery head (measured between pump delivery branch and the highest water level in the high-level storage tank) to the friction head in the delivery pipe:

$$h_{d\ man} = h_{dz} + h_{dF}$$

Under normal circumstances, the head loss due to velocity can be disregarded in this context.

The friction heads for the most commonly used pipe diameters are given in the tables and charts in Appendix 1.

3.3 Dimensioning of pressure pipes; use of tables and charts

3.3.1 General points

The importance of the part played by friction head, h_F, in hydraulic engineering has already been pointed out (heading 3.1).

In pratice, when dimensioning pressure pipes, the friction heads that have to be entered into the calculations are taken from tables or charts. Tables and charts covering the most commonly used pipe diameters are appended to this module.

The resistance to flow through the pipe which is the cause of friction head is determined by a roughness factor, "K", the theoretical provenance of which will not be discussed here.

Experience has shown that the roughness factor in new water mains is determined much less by surface texture of the pipe wall than by other resistances, caused by e.g. elbows, valves, other components, air bubbles etc. The part played by surface roughness of the pipe wall, which is dependent on material, is only a small element in the overall effect.

For this reason, the roughness factor is given in the appended tables and charts as "integral roughness factor k_i".

The influence of the pipe material in this context is so negligible that detailed figures are omitted.

The roughness factor k_i as given in millimetres in the head loss tables must not be understood as an indication of the measurable height of any obstructions or unevenness on the pipe wall, but as a measure of the hydraulic friction for the pipe run as a whole.

3.3.2 Choice of appropriate roughness factor

Measurements of head losses have shown that use of the roughnesses as suggested below results in a good correlation beween calculation and measurement:

k_i = 0.1 (mm):	Long-distance transmission and feeder mains laid in direct runs and made either of cast steel or iron lined with cement mortar or bitumen, or of prestressed concrete or asbestos cement.
k_i = 0.4 (mm):	Trunk mains laid mainly in direct runs and made of the same materials as above, but with the addition of unlined cast steel or iron pipes, providing water properties and method of operation do not lead to formation of deposits (Charts II and IIa and Table II).
k_i = 1.0 (mm)	New water mains; raising k_i from 0.4 (mm) to 1.0 (mm) approximately takes the effect of interconnection into account (Chart II and Table III).

3.3.3 Use of the charts and tables

The following units were used in drawing up the tables and plotting the charts:

Volume of flow Q	l/s
Inside diameter d	mm

Velocity of flow v		m/s
Relative head loss (gradient) $J = \frac{h}{l}$		m/km
Friction head h_F		mH_2O

The charts were calculated for water with a temperature of 10°C, a density of 99.6 kg/cm^3 and a kinematic viscosity of 1.31 x 10^6 m^2/s.
The tables are based on round figures for the flow volume. When carrying out rough calculations, intermediate values can be interpolated on a linear basis; exact figures can be taken from the charts. It should be noted that the examples use the inside diameter Ø (mm) and not the nominal dia. DN.

The vertical axis in the charts represents the volume of flow Q (l/s) and the horizontal axis the relative head loss J (m/km), from top left to bottom right the velocity v (m/s) and from bottom left to top right the inside diameter Ø (mm).

3.3.4 Selection of the most appropriate flow velocity

The velocity of flow not only influences the overall economy of a water-supply system but also plays a major part in operational reliability and safety. High velocities result in high head losses; sudden and extreme changes of velocity can cause water hammer. Flow velocities which are too low result in over-long dwell times. An examination of the technically and economically most satisfactory range of flow velocities is always necessary, but is especially important for long-distance transmission and feeder mains.

Initial considerations should be based on the following figures:

Long-distance transmission and feeder mains:	2.0 m/s
Trunk mains inside service systems	1.0 - 2.0 m/s
Service mains	0.5 - 0.8 m/s

3.4 Applied examples

Example 1

At a gradient of J = 2.0 m/km, i.e. h_F = 2 m at a pipe length of l = 1000 m, a flow volume of Q = 100 l/s ist to be achieved. What inside diameter Ø (mm) must be chosen?

a) Calculated for a trunk main:
On Chart I (k_i = 0.1 mm), J = 2.0 m/km and Q = 100 l/s intersect between Ø 300 mm and 400 mm. The choice should fall on Ø 400 mm at v = 0.80 m/s. Here, Q = 122 l/s can flow at v = 0.97 m/s, whilst the pipe with Ø 300 can transport only Q = 57 l/s at v = 0.81 m/s.

b) Calculated for a service main:
Chart II (k_1 = 0.4 mm) gives Q = 109.5 l/s flowing through the pipe with Ø 400 mm at v = 0.87 m/s, whereas Ø 300 mm could transport only Q = 51 l/s at v = 0.72 m/s.

Example 2 (trunk main)

From a high-level storage tank A, the lowest water level of which is m.s.l. + 100 m, Q = 15 l/s are to be piped by gravity to a point B, altitude m.s.l. + 92 m, at a distance of l = 7.28 km.

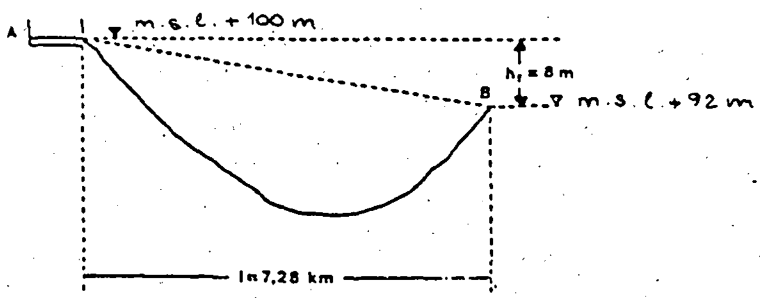


Fig. 10

What inside diameter Ø (mm) is necessary and how high is the velocity in the pipe?

The available head h_F results from the difference in height between A und B, i.e.

$$h_F = (\text{m.s.l.} + 100 \text{ m}) - (\text{m.s.l.} + 92) = 8 \text{ m}$$

Thus the gradient over l = 7.28 is $J = \frac{h}{l} = \frac{8}{7.28} = 1.1$ m/km

Since this is a trunk main, Chart or Table I should be used. Table I (k_i = 0.1 mm) gives for Q = 15 l/s and Ø 200 mm

J = 1.23 m/km at v = 0.48 m/s. Thus $h_F = l \times J = 7.28 \times 1.23 = 8.95$ m

Chart I shows that with the available gradient J = 1.1 m/km, a pipe with Ø 200 mm can transport only 14.2 l/s. It should therefore be considered whether this smaller volume of flow is adequate, or whether the head at B can be lowered from m.s.l. + 92 m to m.s.l. + 100 m - 8.95 m = m.s.l. + 91.5 m. If in doubt, the next largest pipe Ø 250 mm should be chosen, which according to Chart I can transport a volume of flow Q = 25.5 l/s at J = 1.1 m/km and v = 0.51 m/s.

Example 3 (service main)

A pump at P, centre axis at m.s.l. + 27 m, is to lift Q = 50 l/s through a cast-iron pipe to a high-level storage tank B, having its highest water level at m.s.l. + 76 m, at a distance of l = 6.0 km. The water main passes through a district A and serves at the same time as a service main for the supply of this area (fig. 11). How high is the total delivery head?

The calculation must start with the permissible economic velocity. This can be assumed at approx. v = 0.8 m/s.

Table II (k_i = 0.4 mm) gives for Q = 50 l/s and v = 0.71 m/s an inside diameter Ø 300 mm with J = 1.91 m/km.
Head loss $h_F = l \times J$ is $h_F = 6.0 \text{ m} \times 1.91 \text{ m/km} = 11.5$ m

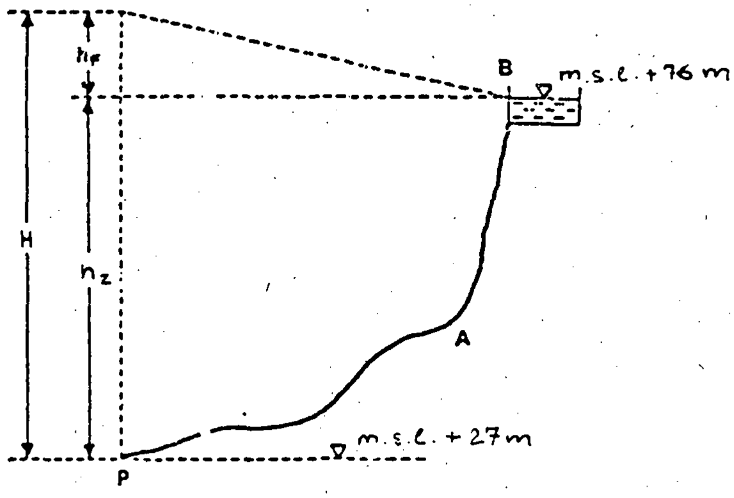


Fig. 11

Since the static pressure head (difference in level) is

$$h_z = (\text{m.s.l.} + 76\text{ m}) - (\text{m.s.l.} + 27\text{ m}) = 49\text{ m},$$

a total delivery head of

$$H = h_F + h_z = 11.5\text{ m} + 49\text{ m} = 60.5\text{ m}$$

results.

Example 4 (pump rising main and feeder main)

In a water impounding area at an altitude of m.s.l. + 50 m, the water table stands at a depth of 80 m, i.e. at m.s.l. - 30 m. Q = 30 l/s are to be pumped out of a deep well and transported to a point at a distance of l = 2 km having an altitude of m.s.l. + 95 m. Due to the extraction of water, the water table drops by 15 m to m.s.l. - 45 m. The pump is to be installed at a level 5 m below this, i.e. at m.s.l. - 50 m (fig. 12). For what delivery head must the pump be dimensioned?

The delivery head is composed of the difference in level between the point of supply at m.s.l. + 95 m and the lowest level of the lowered water table at m.s.l. - 45 m (h_z), the friction head of the pumping main Ø 150 mm from the

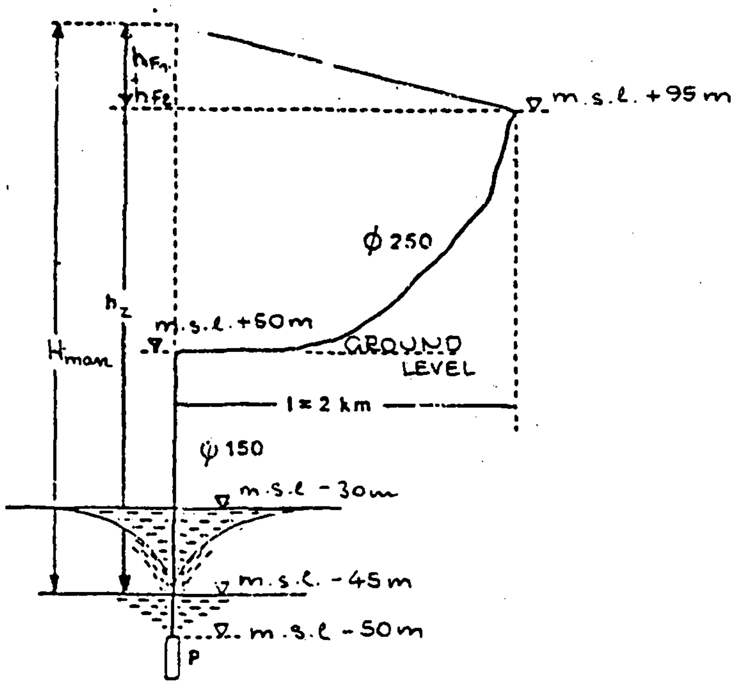


Fig. 12

pump to the top edge of the well or ground surface (h_{F_1})

and the friction head in the delivery pipe Ø 250 mm to the supply point (h_{F_2}).

Thus $\quad H_{man} = h_z + h_{F_1} + h_{F_2}$

where $\quad h_z = (\text{m.s.l.} + 95\text{ m}) - (\text{m.s.l.} - 45\text{ m}) = 140\text{ m}.$

The rising main from the pump to the top edge of the well has a length of

$$l_1 = (\text{m.s.l.} + 50\text{ m}) - (\text{m.s.l.} - 50\text{ m}) = 100\text{ m} = 0.1\text{ km}$$

Table II (k_i = 0.4 mm) gives, at Q = 30 l/s and Ø 150 mm, the gradient J_1 = 25.5 m/km with v = 1.7 m/s, thus

$$h_{F_1} = l_1 \times J_1 = 0.1\text{ km} \times 25.5\text{ m/km} = 2.6\text{ m}$$

For the delivery pipe with Ø 250 mm, Table II (k_i = 0.4 mm) gives:

$$J_2 = 1.81 \text{ m/km and } v = 0.61 \text{ m/s},$$

thus $h_{F_2} = l_2 \times J_2 = 2.0 \text{ km} \times 1.81 \text{ m/km} = 3.6 \text{ m}$

The total delivery head is thus

$$H_{man} = h_z + h_{F_1} + h_{F_2} = 140 + 2.6 + 3.6$$

$$H_{man} = 146.2 \text{ m}$$

Appendix 1

Basic principles of water transport

TABLES AND CHARTS

Basic prinicples of water transport

Table I: Relative head losses for $k_i = 0.1$ (mm)
Ø: Inside diameter (mm)

	Ø 80		Ø 100		Ø 125	
Q	v	J	v	J	v	J
l/s	m/s	m/km	m/s	m/km	m/s	m/km
1	0.20	0.790	0.13	0.269	0.08	0.092
1.5	0.30	1.638	0.19	0.553	0.12	0.188
2	0.40	2.766	0.25	0.927	0.16	0.314
3	0.60	5.838	0.38	1.938	0.24	0.650
4	0.80	9.982	0.51	3.289	0.33	1.096
5	0.99	15.189	0.64	4.974	0.41	1.649
6	1.19	21.452	0.76	6.992	0.49	2.307
7	1.39	28.769	0.89	9.340	0.57	3.070
8	1.59	37.137	1.02	12.016	0.65	3.936
9	1.79	46.554	1.15	15.020	0.73	4.905
10	1.99	57.021	1.27	18.350	0.81	5.977
15	2.98	125.066	1.91	39.893	1.22	12.865
20			2.55	69.566	1.63	22.291
30					2.44	48.723
40					3.26	85.244

Basic principles of water transport

Table 1: Relative head losses for k_i = 0.1 mm
Ø: Inside diameter (mm)

	Ø 150		Ø 200		Ø 250	
Q	v	J	v	J	v	J
l/s	m/s	m/km	m/s	m/km	m/s	m/km
1	0.06	0.039				
1.5	0.08	0.079				
2	0.11	0.130				
3	0.17	0.269	0.10	0.067		
4	0.23	0.450	0.13	0.112	0.08	0.038
5	0.28	0.675	0.16	0.167	0.10	0.057
6	0.34	0.941	0.19	0.231	0.12	0.079
7	0.40	1.248	0.22	0.306	0.14	0.104
8	0.45	1.595	0.25	0.389	0.16	0.132
9	0.51	1.983	0.29	0.482	0.18	0.163
10	0.57	2.411	0.32	0.585	0.20	0.197
15	0.85	5.148	0.48	1.233	0.31	0.412
20	1.13	8.869	0.64	2.105	0.41	0.698
30	1.70	19.242	0.95	4.509	0.61	1.482
40	2.26	33.509	1.27	7.787	0.81	2.543
50	2.83	51.663	1.59	11.933	1.02	3.876
60	3.40	73.699	1.91	16.945	1.22	5.481
70			2.23	22.821	1.43	7.358
80			2.55	29.561	1.63	9.504
90			2.86	37.164	1.83	11.921
100			3.18	45.630	2.04	14.607
150					3.06	32.080

Basic principles of water transport

Table 1: Relative head losses for k_i = 0.1 mm
Ø: Inside diameter (mm)

	Ø 300		Ø 400		Ø 500	
Q l/s	v m/s	J m/km	v m/s	J m/km	v m/s	J m/km
6	0.08	0.033				
7	0.10	0.043				
8	0.11	0.055				
9	0.13	0.068				
10	0.14	0.082	0.08	0.021		
15	0.21	0.169	0.12	0.043		
20	0.28	0.286	0.16	0.072	0.10	0.025
30	0.42	0.602	0.24	0.149	0.15	0.051
40	0.57	1.027	0.32	0.250	0.20	0.085
50	0.71	1.559	0.40	0.377	0.25	0.127
60	0.85	2.198	0.48	0.529	0.31	0.177
70	0.99	2.941	0.56	0.703	0.36	0.235
80	1.13	3.790	0.64	0.902	0.41	0.301
90	1.27	4.744	0.72	1.126	0.46	0.374
100	1.41	5.802	0.80	1.372	0.51	0.454
150	2.12	12.658	1.19	2.958	0.76	0.969
200	2.83	22.117	1.59	5.130	1.02	1.671
300			2.39	11.219	1.53	3.622
400			3.18	19.633	2.04	6.304
500					2.55	9.714
600					3.06	13.850
700					3.56	18.714

Basic principles of water transport

Table I1: Relative head losses for k_i = 0.40 mm
Ø: Inside diameter (mm)

	Ø 80		Ø 100		Ø 125	
Q	v	J	v	J	v	J
l/s	m/s	m/km	m/s	m/km	m/s	m/km
1	0.20	0.921	0.13	0.302	0.08	0.101
1.5	0.30	1.972	0.19	0.638	0.12	0.210
2	0.40	3.408	0.25	1.094	0.16	0.356
3	0.60	7.431	0.38	2.359	0.24	0.759
4	0.80	12.986	0.51	4.095	0.33	1.307
5	0.99	20.072	0.64	6.301	0.41	2.001
6	1.19	28.688	0.76	8.977	0.49	2.839
7	1.39	38.835	0.89	12.123	0.57	3.821
8	1.59	50.513	1.02	15.738	0.65	4.948
9	1.79	63.721	1.15	19.822	0.73	6.219
10	1.99	78.459	1.27	24.375	0.81	7.635
15	2.98	175.102	1.91	54.182	1.22	16.876
20			2.55	95.719	1.63	29.723
30					2.44	66.229
40					3.26	117.154

Basic principles of water transport

Table II: Relative head losses for k_i = 0.40 mm

Ø: Inside diameter (mm)

	Ø 150		Ø 200		Ø 250	
Q	v	J	v	J	v	J
l/s	m/s	m/km	m/s	m/km	m/s	m/km
1	0.06	0.041				
1.5	0.08	0.085				
2	0.11	0.144				
3	0.17	0.304	0.10	0.073		
4	0.23	0.520	0.13	0.124	0.08	0.041
5	0.28	0.791	0.16	0.187	0.10	0.062
6	0.34	1.118	0.19	0.262	0.12	0.086
7	0.40	1.501	0.22	0.350	0.14	0.115
8	0.45	1.938	0.25	0.449	0.16	0.147
9	0.51	2.431	0.29	0.562	0.18	0.183
10	0.57	2.979	0.32	0.686	0.20	0.223
15	0.85	6.546	0.48	1.489	0.31	0.479
20	1.13	11.490	0.64	2.595	0.41	0.828
30	1.70	25.509	0.95	5.715	0.61	1.809
40	2.26	45.034	1.27	10.044	0.81	3.164
50	2.83	70.064	1.59	15.582	1.02	4.892
60	3.40	100.601	1.91	22.328	1.22	6.994
70			2.23	30.283	1.43	9.470
80			2.55	39.447	1.63	12.320
90			2.86	49.819	1.83	15.543
100			3.18	61.400	2.04	19.139
150					3.06	42.726

Basic principles of water transport

Table II: Relative head losses for k_i = 0.40 mm

Ø: Inside diameter (mm)

	Ø 300		Ø 400		Ø 500	
Q	v	J	v	J	v	J
l/s	m/s	m/km	m/s	m/km	m/s	m/km
6	0.08	0.035				
7	0.10	0.047				
8	0.11	0.060				
9	0.13	0.074				
10	0.14	0.090	0.08	0.023		
15	0.21	0.191	0.12	0.046		
20	0.28	0.329	0.16	0.079	0.10	0.026
30	0.42	0.712	0.24	0.167	0.15	0.056
40	0.57	1.240	0.32	0.287	0.20	0.095
50	0.71	1.910	0.40	0.441	0.25	0.144
60	0.85	2.725	0.48	0.624	0.31	0.203
70	0.99	3.682	0.56	0.841	0.36	0.271
80	1.13	4.783	0.64	1.088	0.41	0.349
90	1.27	6.028	0.72	1.367	0.46	0.438
100	1.41	7.416	0.80	1.679	0.51	0.535
150	2.12	16.504	1.19	3.709	0.76	1.175
200	2.83	19.175	1.59	6.531	1.02	2.061
300			2.39	14.552	1.53	4.570
400			3.18	25.740	2.04	8.063
500					2.55	12.539
600					3.06	18.000
700					3.56	24.442

Revised:

Basic principles of water transport

Table III: Relative head losses for k_i = 1.0 mm

Ø: Inside diameter (mm)

	Ø 80		Ø 100		Ø 125		Ø 150	
Q	v	J	v	J	v	J	v	J
l/s	m/s	m/km	m/s	m/km	m/s	m/km	m/s	m/km
1	0.20	1.129	0.13	0.357	0.08	0.115	0.06	0.046
1.5	0.30	2.472	0.19	0.773	0.12	0.245	0.08	0.097
2	0.40	4.331	0.25	1.346	0.16	0.424	0.11	0.167
3	0.60	9.595	0.38	2.962	0.24	0.924	0.17	0.360
4	0.80	16.922	0.51	5.203	0.33	1.615	0.23	0.626
5	0.99	26.311	0.64	8.071	0.41	2.496	0.28	0.964
6	1.19	37.763	0.76	11.564	0.49	3.568	0.34	1.374
7	1.39	51.277	0.89	15.683	0.57	4.830	0.40	1.856
8	1.59	66.854	1.02	20.428	0.65	6.282	0.45	2.411
9	1.79	84.492	1.15	25.799	0.73	7.926	0.51	3.038
10	1.99	104.193	1.27	31.795	0.81	9.795	0.57	3.737
15	2.98	233.634	1.91	71.164	1.22	21.783	0.85	8.315
20			2.55	126.177	1.63	38.567	1.13	14.697
30					2.44	86.417	1.70	32.877
40					3.26	153.308	2.26	58.269
50							2.83	90.881
60							3.40	130.711

Basic principles of water transport

Table III: Relative head losses for k_i = 1.0 mm

Ø: Inside diametaer (mm)

	Ø 200		Ø 250		Ø 300	
Q	v	J	v	J	v	J
l/s	m/s	m/km	m/s	m/km	m/s	m/km
3	0.10	0.083				
4	0.13	0.143	0.08	0.046		
5	0.16	0.218	0.10	0.070		
6	0.19	0.309	0.12	0.099	0.08	0.039
7	0.22	0.417	0.14	0.133	0.10	0.052
8	0.25	0.539	0.16	0.171	0.11	0.068
9	0.29	0.677	0.18	0.214	0.13	0.084
10	0.32	0.831	0.20	0.262	0.14	0.103
15	0.48	1.836	0.31	0.575	0.21	0.224
20	0.64	3.233	0.41	1.007	0.28	0.391
30	0.95	7.202	0.61	2.233	0.42	0.863
40	1.27	12.738	0.81	3.939	0.57	1.517
50	1.59	19.840	1.02	6.126	0.71	2.355
60	1.91	28.510	1.22	8.793	0.85	3.376
70	2.23	38.746	1.43	11.941	0.99	4.580
80	2.55	50.549	1.63	15.569	1.13	5.968
90	2.86	63.919	1.83	19.678	1.27	7.538
100	3.18	78.856	2.04	24.267	1.41	9.292
150			3.06	54.418	2.12	20.808
200					2.83	36.904

Basic principles of water tranpsort

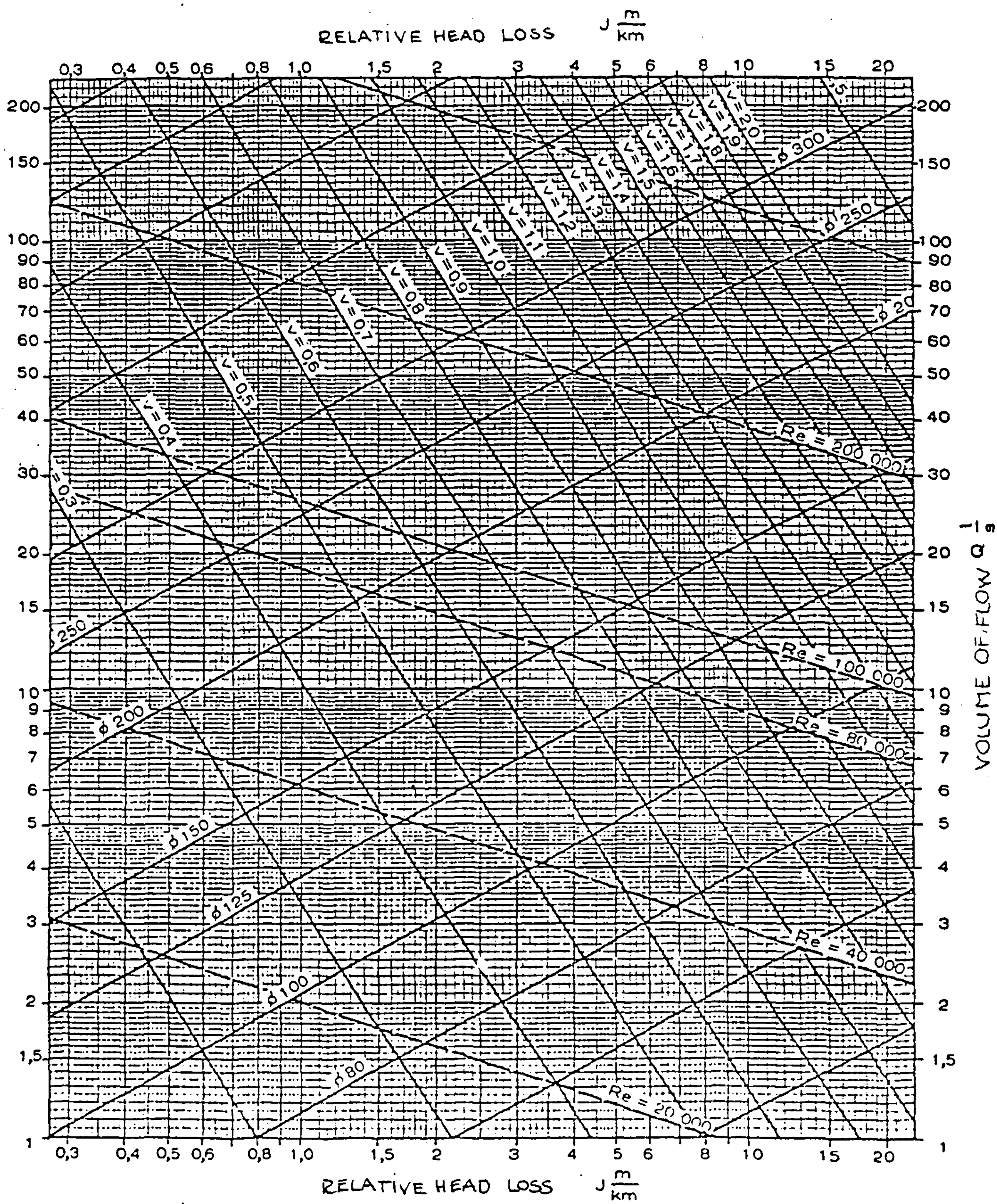
Table III: Relative head losses for k_i = 1.0 (mm)

Ø: Inside diameter (mm)

	Ø 400		Ø 500		Ø 600		Ø 700	
Q	v	J	v	J	v	J	v	J
l/s	m/s	m/km	m/s	m/km	m/s	m/km	m/s	m/km
10	0.08	0.025						
15	0.12	0.052						
20	0.16	0.090	0.10	0.030				
30	0.24	0.196	0.15	0.064	0.11	0.026		
40	0.32	0.341	0.20	0.110	0.14	0.044	0.10	0.020
50	0.40	0.527	0.25	0.167	0.18	0.065	0.13	0.031
60	0.48	0.752	0.31	0.237	0.21	0.093	0.16	0.043
70	0.56	1.018	0.36	0.320	0.25	0.126	0.18	0.057
80	0.64	1.325	0.41	0.416	0.28	0.163	0.21	0.074
90	0.72	1.617	0.46	0.524	0.32	0.204	0.23	0.093
100	0.80	2.058	0.51	0.643	0.35	0.251	0.26	0.115
150	1.19	4.591	0.76	1.429	0.53	0.555	0.39	0.250
200	1.59	8.127	1.02	2.526	0.71	0.976	0.52	0.439
300	2.39	18.207	1.53	5.645	1.06	2.175	0.78	0.974
400	3.18	32.298	2.04	10.002	1.41	3.849	1.04	1.720
500			2.55	15.596	1.77	5.996	1.30	2.677
600			3.06	22.429	2.12	8.618	1.56	3.846
700			3.56	30.499	2.48	11.713	1.82	5.224
800					2.83	15.283	2.08	6.815
900					3.18	19.327	2.34	8.615
1000					3.54	23.845	2.60	10.626
1500							3.90	23.845

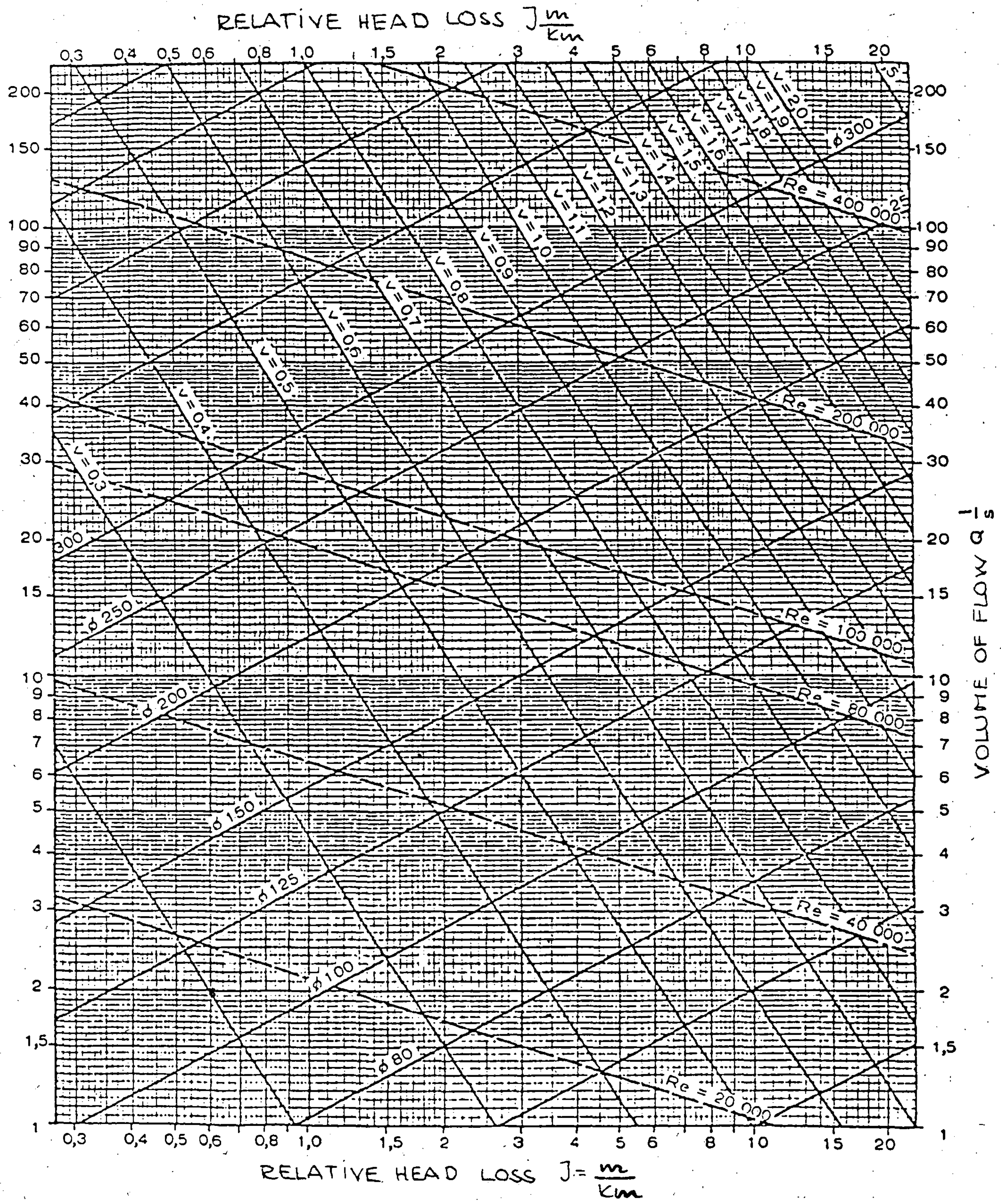
Basic principles of water transport

Chart Ia: Roughness k_i = 0.1 mm (enlarged extract from Chart I), v (m/s), Ø (mm)



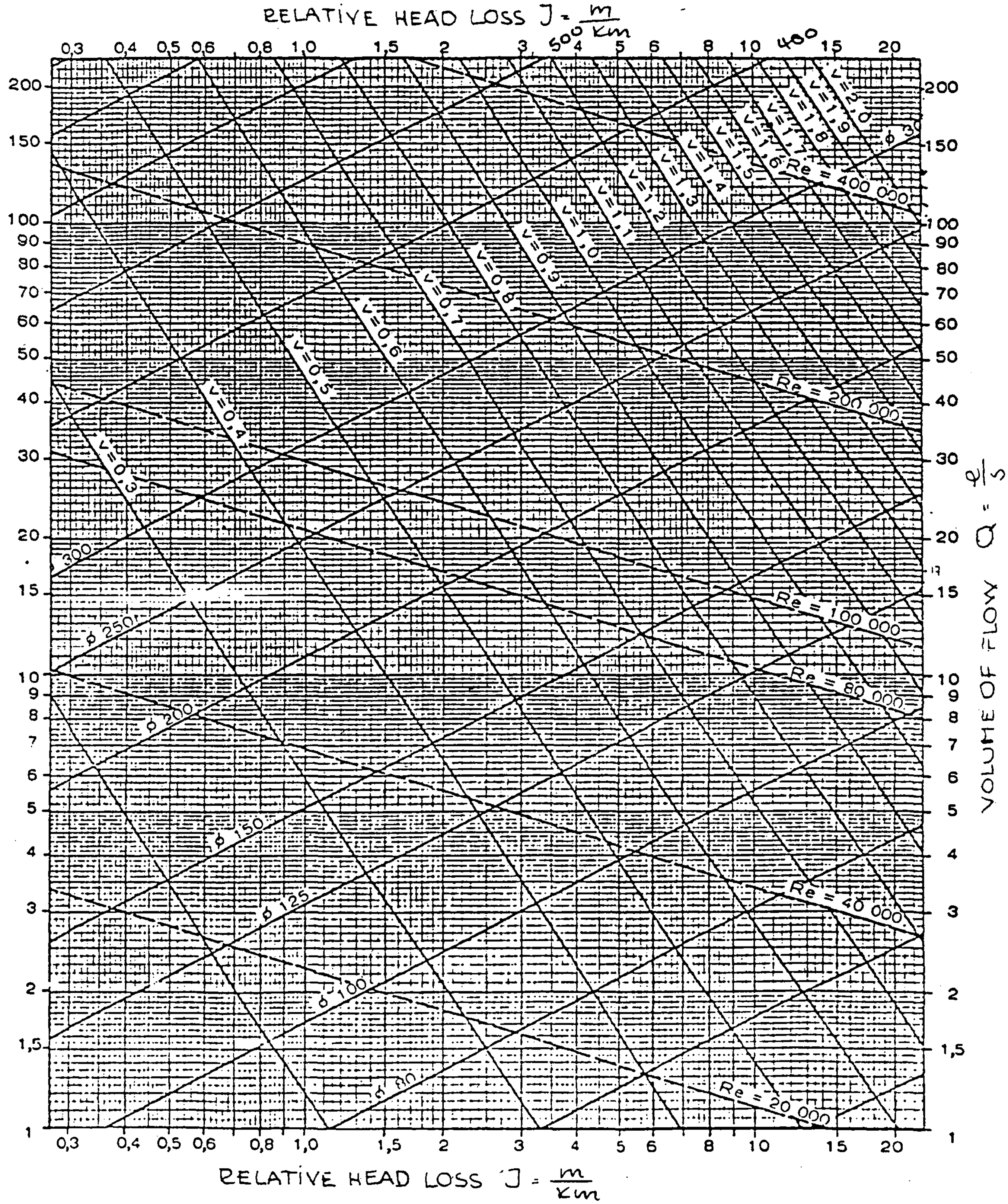
Basic principles of water transport

Chart IIa: Roughness k_i = 0.4 mm (enlarged extract from Chart II), v (m/s), Ø (mm)



Basic principles of water transport

Chart IIIa: Roughness k_i = 1.0 mm (enlarged extract from Chart III), v (m/s), Ø (mm)



Deutsche Gesellschaft für Technische Zusammenarbeit (GTZ) GmbH
Dag-Hammarskjöld-Weg 1 + 2 · D 6236 Eschborn 1 · Telefon (0 61 96) 79-0 · Telex 4 07 501-0 gtz d

The government-owned GTZ operates in the field of Technical Cooperation. Some 4,500 German experts are working together with partners from some 100 countries in Africa, Asia and Latin America in projects covering practically every sector of agriculture, forestry, economic development, social services and institutional and physical infrastructure. - The GTZ is commissioned to do this work by the Government of the Federal Republic of Germany and by other national and international organizations.

GTZ activities encompass:

- appraisal, technical planning, control and supervision of technical cooperation projects commissioned by the Government of the Federal Republic of Germany or by other authorities
- advisory services to other agencies implementing development projects
- the recruitment, selection, briefing and assignment of expert personnel and assuring their welfare and technical backstopping during their period of assignment
- provision of materials and equipment for projects, planning work, selection, purchasing and shipment to the developing countries
- management of all financial obligations to the partnercountry.

The series **"Sonderpublikationen der GTZ"** includes more than 190 publications. A list detailing the subjects covered can be obtained from the GTZ-Unit 02: Press and Public Relations, or from the TZ-Verlagsgesellschaft mbH, Postfach 36, D 6101 Roßdorf 1, Federal Republic of Germany.

TRAINING MODULES FOR WATERWORKS PERSONNEL

List of training modules:

gtz

Deutsche Gesellschaft für Technische Zusammenarbeit (GTZ) GmbH

P. O. Box 5180
Dag-Hammarskjöld-Weg 1+2
D 6236 Eschborn/Ts. 1
Telephone (0 6196) 79-0
Telex 4 07 501-0 gtz d
Fax No. (0 6196) 79-1115