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Basics of hydraulics & management of pressure transients

WAT-E2110 - Design and Management of Water and Wastewater Networks L, 2019

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Motivation

- Design of supply and sewage networks
- Also relates to pipe flows e.g. in industry
- Understand the physical basis of network modelling

Contents

- Basic hydraulics
- Determining resistance coefficients
- Approaches to practical problems
- Management of pressure transients

- Hints for the assignment

BASIC HYDRAULICS

Basic principles

- Continuity eq.
- Conservation of momentum (flow rate) ρQv
- Conservation of energy (Bernoulli eq.)
- Transport equations for substances

Force = rate of change of momentum

$$\sum F = \rho Q(v_2 - v_1)$$

Bernoulli eq.

$$z_1 + \alpha \frac{v_1^2}{2g} + \frac{P_1}{\rho g} = z_2 + \alpha \frac{v_2^2}{2g} + \frac{P_2}{\rho g} + \text{losses}$$

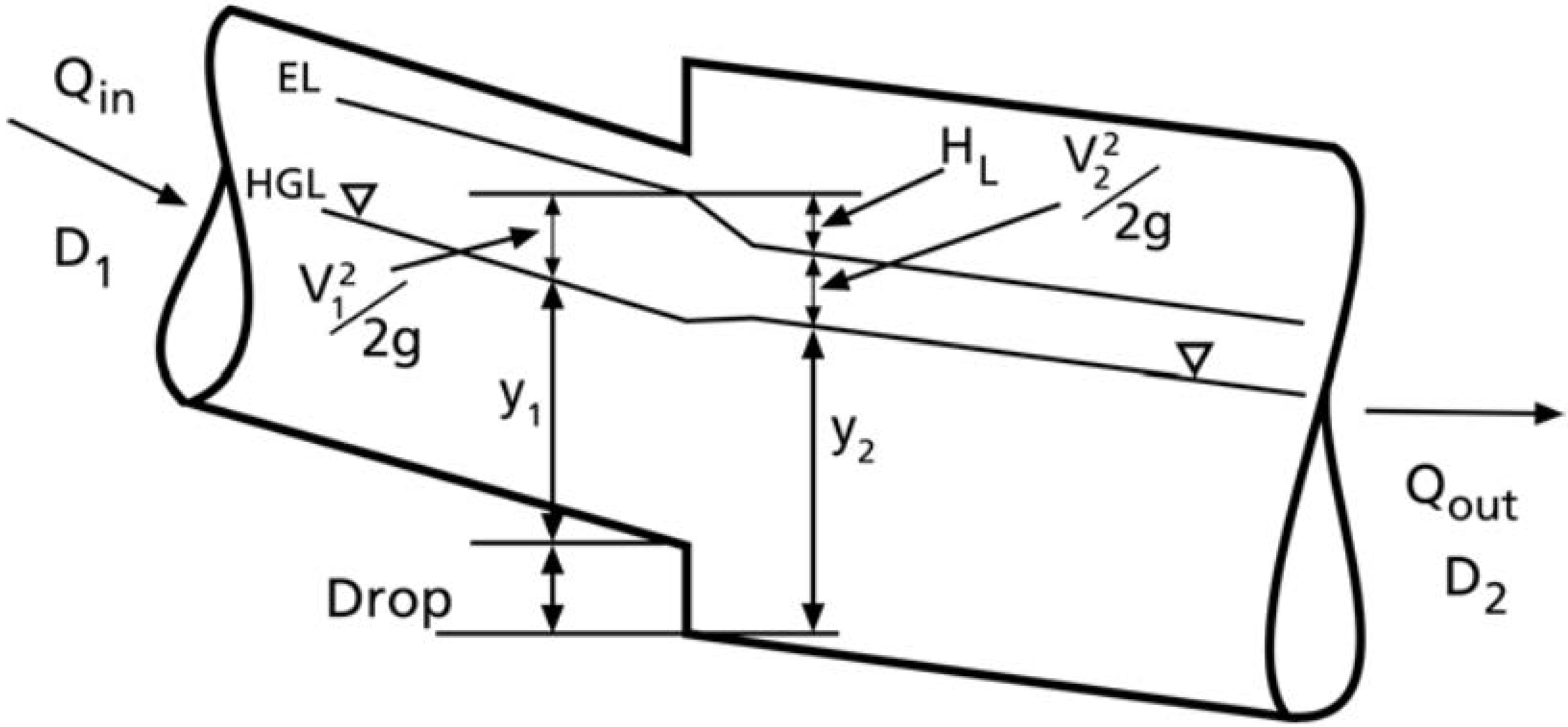
- for steady state computations
- viscous effects or turbulence not explicitly considered -> extra coefficients and approaches for determining energy losses caused by e.g. friction

Bernoulli eq:

$$z = \frac{v_2^2}{2g} + \text{losses}$$

Bernoulli computation for a manhole

$$z_1 + \alpha \frac{v_1^2}{2g} + \frac{P_1}{\rho g} = z_2 + \alpha \frac{v_2^2}{2g} + \frac{P_2}{\rho g} + \text{losses}$$



Determination of head loss h_f for pipe flows

- Laminar flows (Poiseuille 1841)

$$h_f = 32\nu \frac{L\nu}{gD^2}$$

  - Not affected by pipe roughness
  - L = pipe length, D= pipe diameter

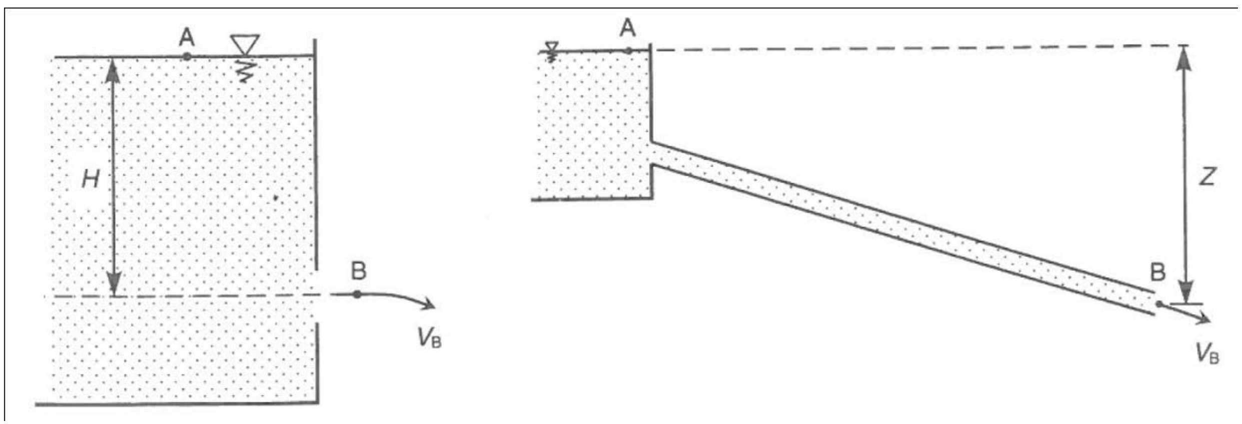
- Turbulent flows (Darcy-Weisbach ca. 1850)

$$h_f = f \frac{L}{D} \frac{v^2}{2g}$$

  - Depends on pipe roughness (friction factor f)
  - Head loss ~ velocity squared

Flow from reservoir to atmosphere and flow between reservoirs

- With Bernoulli eq.: $$Z = V_B{}^2/2g + f\frac{L}{D}\frac{v^2}{2g} + \text{local losses}$$
- For pipe flows the loss terms are important
  - vs. discharge through a small orifice, for which $H = V_B{}^2/2g$



- For flow between reservoirs,

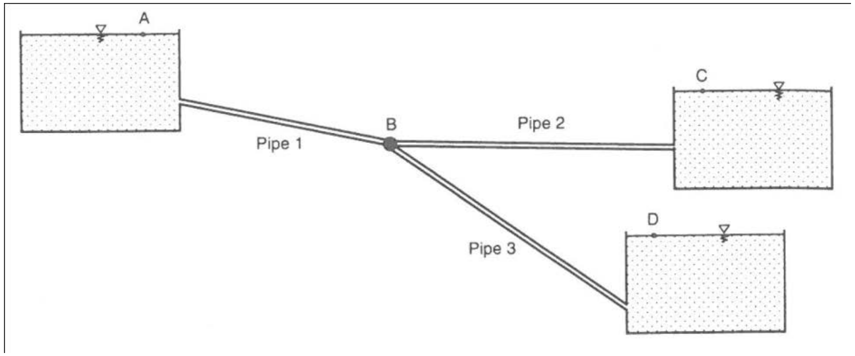
  $V_A=V_B=0$ and $Z = \text{head losses} + \text{local losses}$

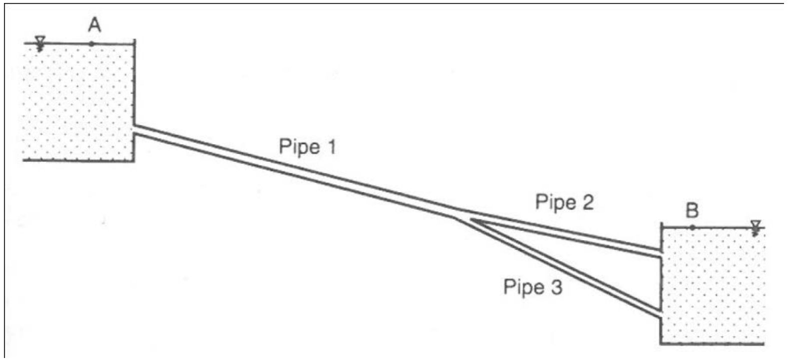
Branching and parallel pipelines

Need as many equations as there are unknowns

-> Bernoulli eq. for each flow path + continuity eq.

$Q_1+Q_2=Q_3$





- For each flow path

$$Z = \text{head losses} + \text{local losses}$$

Local losses

- Local/minor losses caused by
  - expansions and contractions (changes in pipe cross-section)
  - manholes
  - branches
  - valves
  - bends
- Can be computed through velocity head and loss coefficient

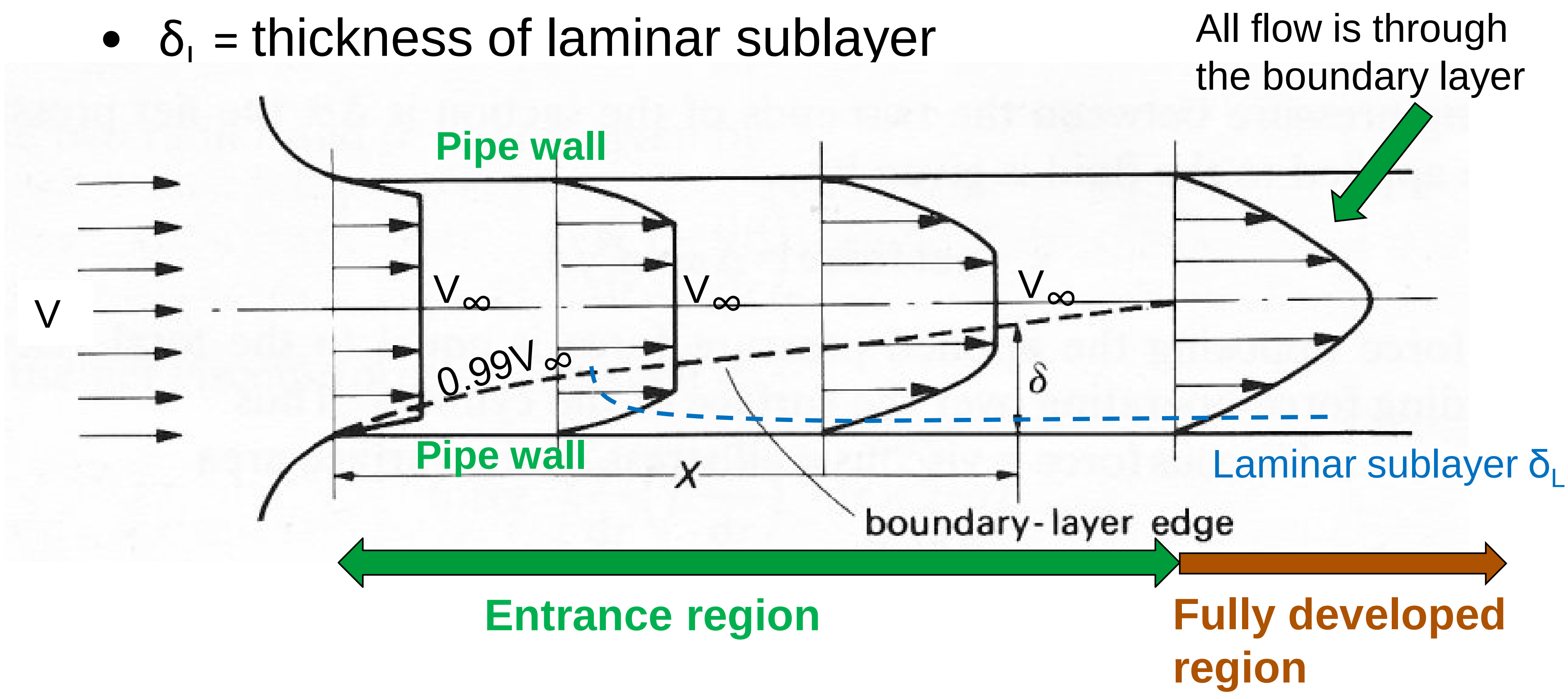
$$h_p = \xi \frac{v^2}{2g}$$

  - ξ-coefficients for different situations can be found in reference tables

DETERMINING RESISTANCE COEFFICIENTS

Boundary layer

- Pipe flows are affected by pipe walls
- In turbulent flows, the flow is laminar at a small distance from pipe wall
- δ = thickness of boundary layer
- δ_l = thickness of laminar sublayer



Categories of pipe flow for the determination of the resistance coefficient

- Laminar: Re<2000
- Transitional: 2000<Re<4000
- Turbulent: Re>4000

  - smooth turbulent

  - transitional turbulent

  - rough turbulent

Flow type	Re for pipe flow	Re for open channel flow
Laminar	< ~2000	< ~500
Transitional	2000 < Re < 4000	500 < Re < 2000
Turbulent	>~ 4000	> ~2000

$$Re = vD/\nu$$

Where ν is kinematic viscosity

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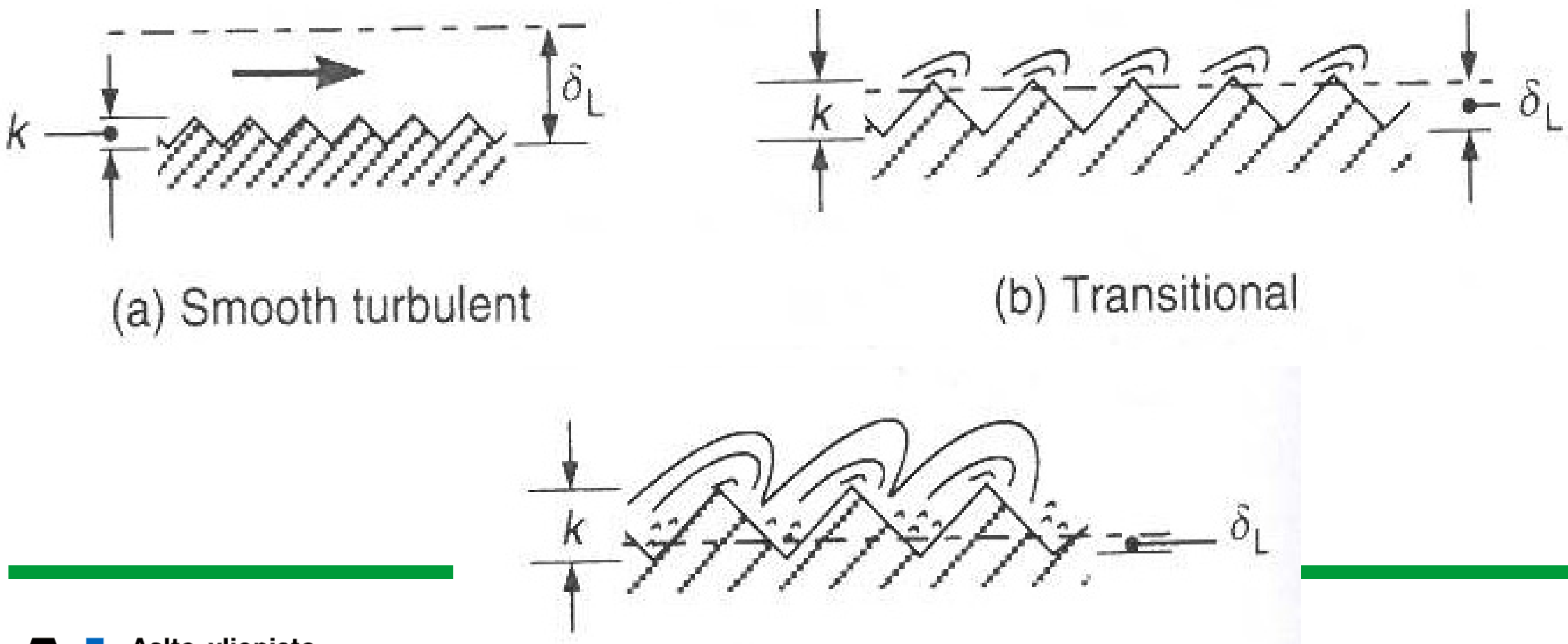
Categories of turbulent flows according to pipe roughness

- Roughness values (k) compiled for different materials

- Flow is categorized according to the relationship between k and thickness of the laminar sublayer



(a) Smooth turbulent

(b) Transitional

(c) Rough turbulent

Reynolds roughness number for categorizing turbulent flows

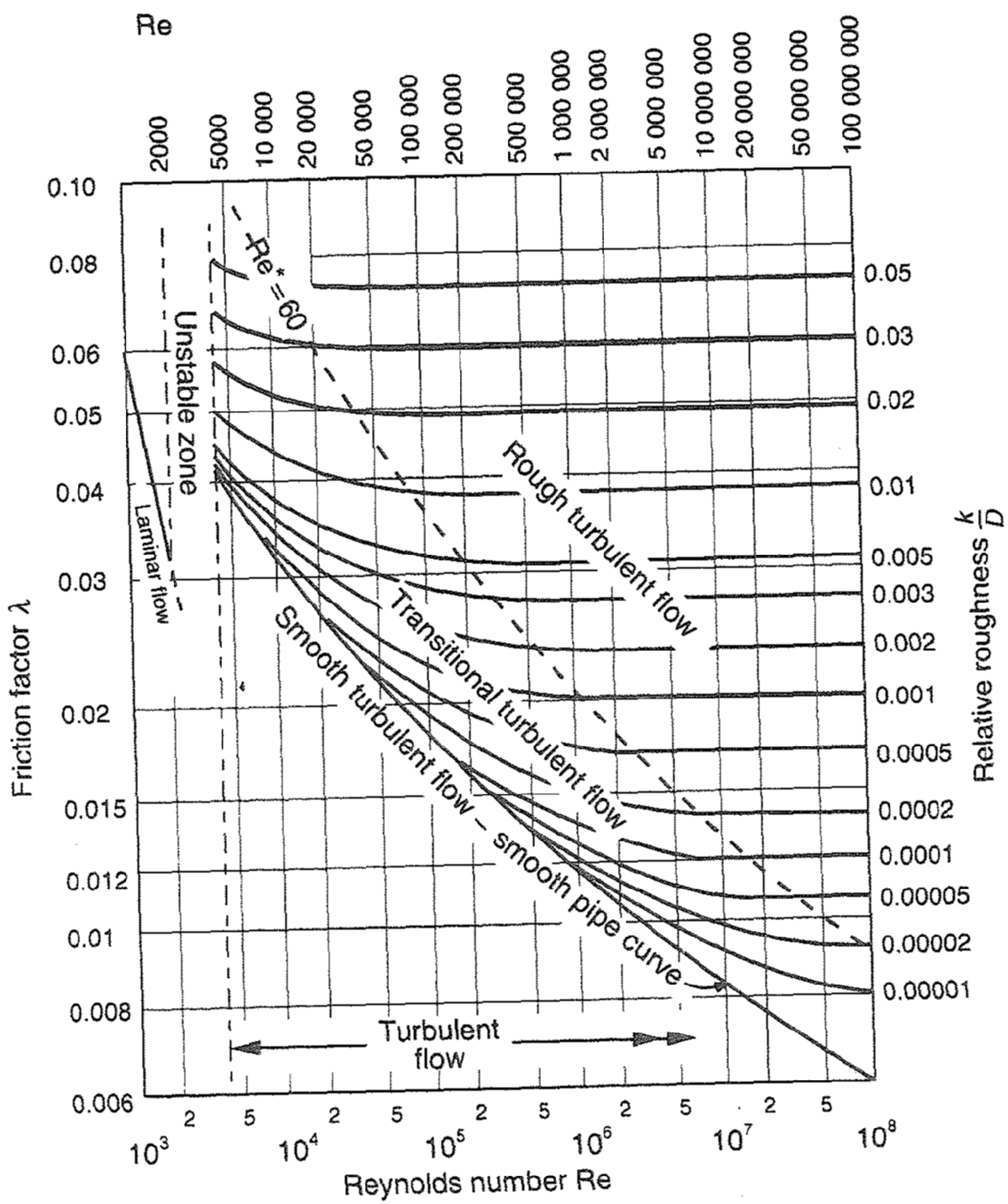
- Flows are categorized according to the k/D value or Reynolds roughness number (Re*)

$$\mathrm{Re}^* = \mathrm{Re}\left(\frac{k}{D}\right)\sqrt{\frac{f}{8}}$$

- Re* < 4: smooth turbulent flow
- 4 < Re* < 60: transitional turbulent flow
- Re* > 60: rough turbulent flow

Moody chart

Experiments of Nikuradse
ca 1930



Friction factor *f* for pipe flows

Darcy:

$$h_f = f \frac{L}{D}\frac{v^2}{2g}$$

- Laminar flow
  (from Poiseuille & Darcy)

$$f = \frac{64}{\mathrm{Re}}$$

- Turbulent flow
  - smooth (Prandl), Re*<4

$$\frac{1}{\sqrt{f}} = 2\log\left(\mathrm{Re}\sqrt{f}\right) - 0.8$$

  - rough (Prandl)

$$\frac{1}{\sqrt{f}} = 2\log\left(\frac{D}{k}\right) + 1.14$$

  - generally (Colebrook-White)

$$\frac{1}{\sqrt{f}} = -2\log\left(\frac{k/D}{3{,}7} + \frac{2{,}51}{\mathrm{Re}\sqrt{f}}\right)$$

  - generally, ~ 5% accuracy (Moody)

$$f = 0.0055\left[1 + \left(20000\frac{k}{D} + \frac{10^6}{\mathrm{Re}}\right)^{1/3}\right]$$

+ a number of newer approximations

APPROACHES TO PRACTICAL PROBLEMS

Simplified/empirical formulae for computing flow velocity

- Blasius for Re*<4 $v = 75D^{5/7}{S_F}^{4/7}$

- Hazen-Williams for 4<Re*<60

- Manning eq. for Re*>60 and for gravity flows

Hazen-Williams eq. for transitional turbulent flows

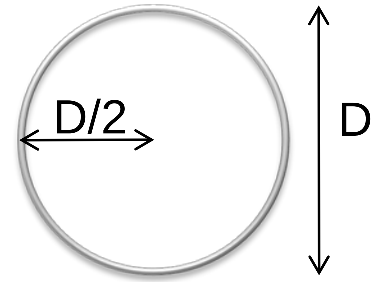
- for 4<Re*<60

$$h_f = \frac{Lv^{1.852}}{(0.355C_H)^{1.852} D^{1.167}} \Leftrightarrow v = 0.355 C_H S_F^{\;0.54} D^{0.63}$$

c_H =Hazen-Williams coefficient

- reasonably accurate for pipes with D>0.15 m, v<3 m/s and c_H>100
- mainly used for determining head losses in supply networks
- c_H depends on flow velocity, pipe diameter and material

Manning eq. for rough turbulent flows



area: $A = \pi(D/2)^2$
wetted perimeter:
$P = 2\pi(D/2)$

- Manning (1889)

$$v = \frac{1}{n} R^{2/3} S_f^{\;1/2} \qquad S_f = \frac{h_f}{L}$$

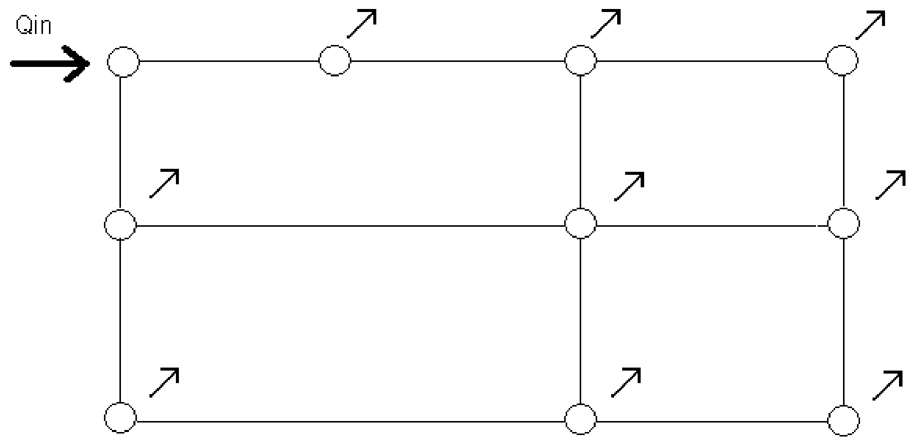
  – where R = A/P is hydraulic radius → R = D/4 for circular pipes

- Darcy-Weisbach (f) and Manning coefficients (n) are related

$$n = \sqrt{\frac{f}{8g}} R^{1/3}$$

- Used mainly for gravity flows

Pipe networks



- Equations needed for the solution
  - At each node, the continuity eq. must hold

$$\sum Q = 0$$

  - the energy losses between two nodes must be identical for all ”routes”
- Modeling software
- Manually solvable through Hardy-Cross method (assignment)

Hardy-Cross-method: determining discharges iteratively (1/2)

1. Guess discharges and directions in different pipes so that

   $\sum Q = 0$ at nodes

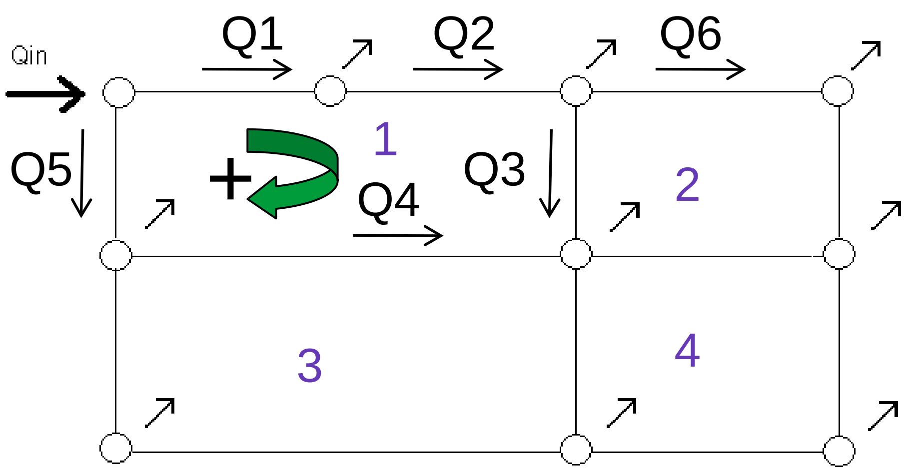
2. Compute h_f in each pipe using the guessed discharge and e.g. Hazen-Williams eq.

$$h_f = \frac{Lv^{1.852}}{(0.355c_H)^{1.852} d^{1.167}}$$

3. Compute total head loss $h = \sum h_f$ in each loop by taking into account the flow direction

   -e.g. h for loop 1:

$$h = h_{f1} + h_{f2} + h_{f3} - h_{f4} - h_{f5}$$



Hardy-Cross-method: determining discharges iteratively (2/2)

$$h = \sum h_f$$

4. If guessed discharges are correct, h~0 and iteration ends

   If h≠0, start iteration.

   Correct each discharge by

$$\Delta Q = -\frac{h}{2\sum \frac{h_f}{Q}}$$

Note the flow direction in all computations !!

   - in the example

$$\Delta Q = -\frac{h}{2*(\frac{h_{f1}}{Q_1} + \frac{h_{f2}}{Q_2} + \frac{h_{f3}}{Q_3} - \frac{h_{f4}}{Q_4} - \frac{h_{f5}}{Q_5})}$$

5. Repeat steps 2-4 with new discharges Q+ΔQ for each loop in turn until h~0

(Derivation of Hardy-Cross method)

- We denote $h_f = KQ^2$.
- ΔQ is solved from the approximation:

$$\sum_c K(Q+\Delta Q)^2 - \sum_a K(Q-\Delta Q)^2 \approx$$

$$\sum_c K(Q^2 + 2\Delta Q * Q) - \sum_a K(Q^2 - 2\Delta Q * Q) \approx 0$$

where the losses for the corrected discharges in the clockwise direction (c) (Q+ΔQ) and anti-clockwise direction (a) (Q-ΔQ) are summed

- The solution is $$\Delta Q = -\frac{h}{2\sum KQ} = -\frac{h}{2\sum \frac{h_f}{Q}}$$

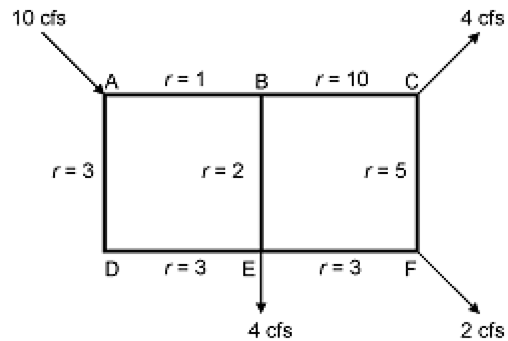
Example: Determining disharcges when the K coefficient is known 1/2

In this example we denote r=K so that

$h_f = rQ^2$, $\Delta Q = -\frac{h}{2\sum rQ} = -\frac{h}{2\sum \frac{h_f}{Q}}$

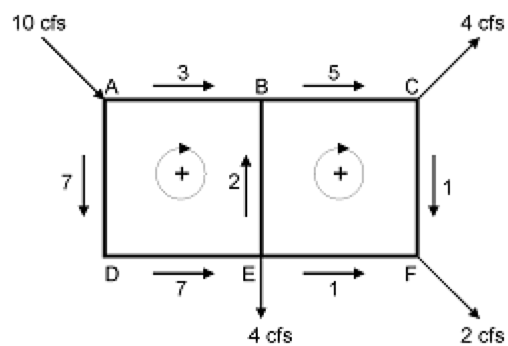
1) Starting point

Pipe Network



2) Guessed Q and directions

First Guess



3) Computing the losses and iteration: loop 1 > correct the value for the shared pipe BE to loop 2 -> second iteration round using the corrected discharges from last round

guessed Q | h_f | $2h_f/Q$ | ΔQ | $Q+\Delta Q$

$\sum$

Iteration	Loop	Pipe	r	Guess Q_i	$rQ_i\|Q_i\|$	$2r\|Q_i\|$	ΔQ	Corrected Q_i
1	1	AB	1	3.00	9.00	6.00		5.99
		BE	2	**-2.00**	-8.00	8.00		**0.99**
		ED	3	-7.00	-147.00	42.00		-4.01
		DA	3	-7.00	-147.00	42.00		-4.01
				Σ	-293.00	98.00	2.99	

Iteration	Loop	Pipe	r	Q_i	$rQ_i\|Q_i\|$	$2r\|Q_i\|$	ΔQ	Q_i
1	2	BC	10	5.00	250.00	100.00		2.92
		CF	5	1.00	5.00	10.00		-1.08
		FE	3	-1.00	-3.00	6.00		-3.08
		EB	2	**-0.99**	-1.96	3.96		**-3.07**
				Σ	250.04	119.96	-2.08	

Iteration	Loop	Pipe	r	Guess Q_i	$rQ_i\|Q_i\|$	$2r\|Q_i\|$	ΔQ	Corrected Q_i
2	1	AB	1	5.99	35.88	11.98		6.57
		BE	2	**3.07**	18.90	12.30		**3.65**
		ED	3	-4.01	-48.25	24.06		-3.43
		DA	3	-4.01	-48.25	24.06		-3.43
				Σ	-41.71	72.40	0.58	

Iteration	Loop	Pipe	r	Q_i	$rQ_i\|Q_i\|$	$2r\|Q_i\|$	ΔQ	Q_i
2	2	BC	10	2.92	85.01	58.31		2.68
		CF	5	-1.08	-5.88	10.84		-1.32
		FE	3	-3.08	-28.54	18.51		-3.32
		EB	2	**-3.65**	-26.65	14.60		**-3.88**
				Σ	23.94	102.26	-0.23	

Example: Determining discharges when the r coefficient is known 2/2

Iteration	Loop	Pipe	r	Guess Q_i	$rQ_i\|Q_i\|$	$2r\|Q_i\|$	ΔQ	Corrected Q_i
3	1	AB	1	6.57	43.11	13.13		6.53
		BE	2	**3.88**	30.18	15.54		**3.85**
		ED	3	-3.43	-35.38	20.60		-3.47
		DA	3	-3.43	-35.38	20.60		-3.47
				Σ	2.53	69.88	-0.04	

Iteration	Loop	Pipe	r	Q_i	$rQ_i\|Q_i\|$	$2r\|Q_i\|$	ΔQ	Q_i
3	2	BC	10	2.68	71.91	53.63		2.68
		CF	5	-1.32	-8.69	13.18		-1.32
		FE	3	-3.32	-33.04	19.91		-3.32
		EB	2	**-3.85**	-29.62	15.39		**-3.85**
				Σ	0.56	102.12	-0.01	

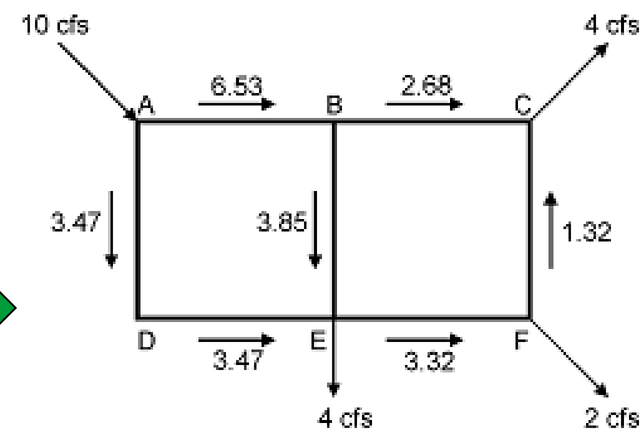
Iteration	Loop	Pipe	r	Guess Q_i	$rQ_i\|Q_i\|$	$2r\|Q_i\|$	ΔQ	Corrected Q_i
4	1	AB	1	6.53	42.64	13.06		6.53
		BE	2	**3.85**	29.70	15.41		**3.85**
		ED	3	-3.47	-36.13	20.82		-3.47
		DA	3	-3.47	-36.13	20.82		-3.47
				Σ	0.08	70.12	0.00	

Iteration	Loop	Pipe	r	Q_i	$rQ_i\|Q_i\|$	$2r\|Q_i\|$	ΔQ	Q_i
4	2	BC	10	2.68	71.61	53.52		2.68
		CF	5	-1.32	-8.76	13.24		-1.32
		FE	3	-3.32	-33.15	19.94		-3.32
		EB	2	**-3.85**	-29.68	15.41		**-3.85**
				Σ	0.02	102.11	0.00	

In this example we denote $h_f = rQ^2$,

$$\Delta Q = -\frac{h}{2\sum rQ} = -\frac{h}{2\sum \frac{h_f}{Q}}$$

5) Final solution, with discharges and directions changed compared to the original guess



4) Iteration continues until h~0 in all loops

(http://www.engineering.uiowa.edu/~water/handouts/Hardy-Cross-1-Soln.pdf)

Design computations for gravity sewers

- Manning eq. for uniform flow (i.e., bed slope = friction slope)
- Bernoulli eq. for gradually-varied flow
  - atmospheric pressure
  - loss terms e.g. with Manning eq. using friction slope

- Minimum slopes/velocities ->self-cleansing
- To reduce energy losses
- Manholes should be designed so that flow passes them smoothly

MANAGEMENT OF PRESSURE TRANSIENTS

Pressure transients

- Pressure transient= hydraulic transient= surge
- Result from any change in the steady-state flow conditions in a pipeline

  -> pressure waves propagating with the velocity of sound

  -> dissipated through damping/friction

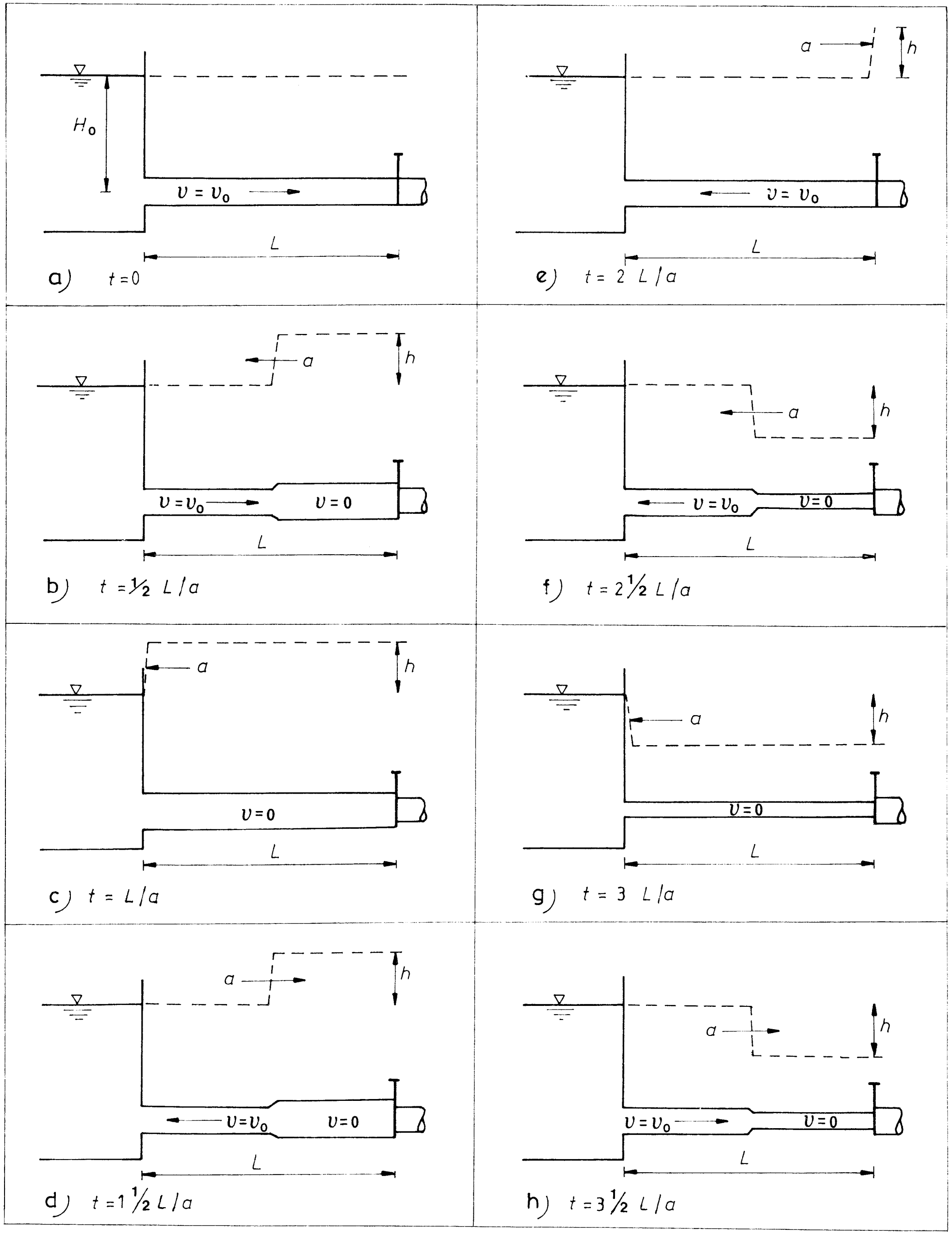
  -> a new steady-state
- Caused by e.g.:
  - Pump startup/shutdown/trip
  - Valve opening/closing
  - Main break

Effects

- Damage to pumps, devices and pipes
- Unwanted mixing of waters
- Intrusion of contaminated water
- Cavitation

-> understanding and controlling important

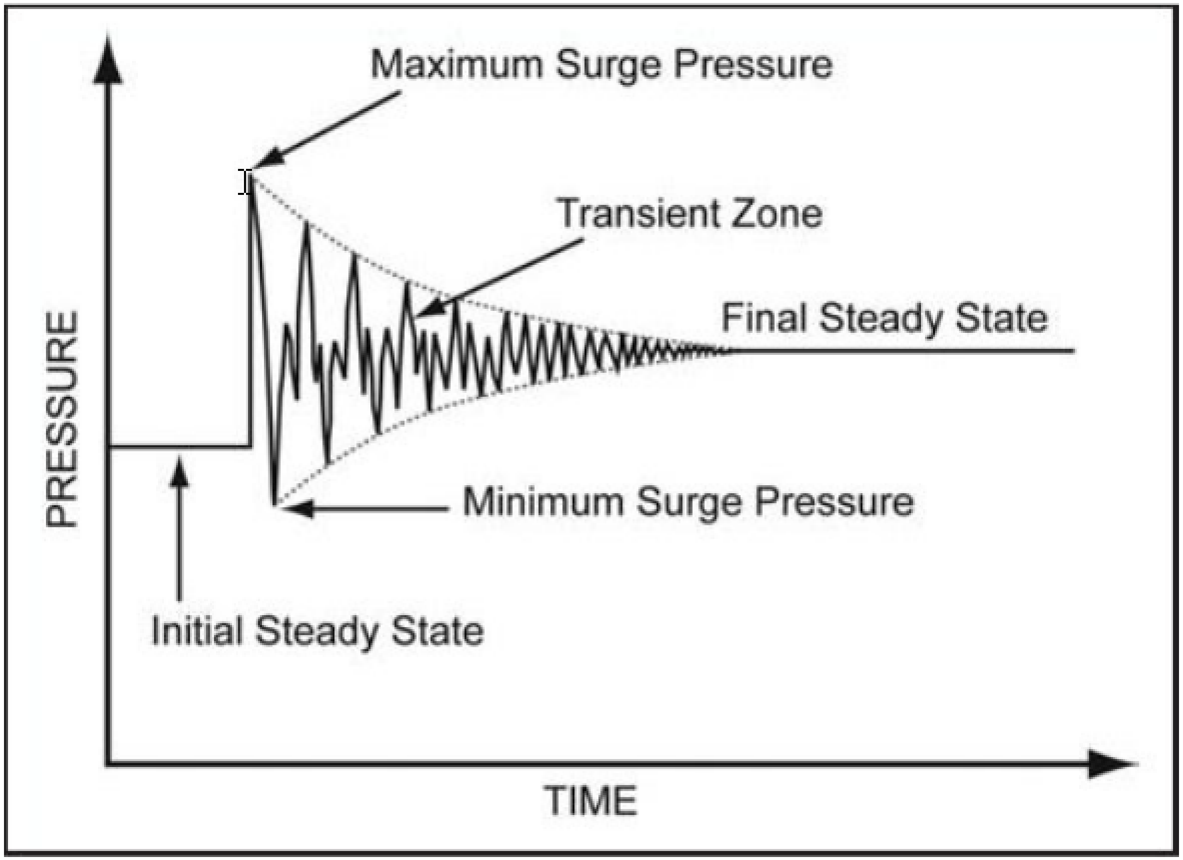
- Most severe effects at pump stations, control valves, in high-elevation areas, in locations with low static pressures, and in remote locations that are distanced from overhead storage



Note: the example presents a frictionless system

In real pipelines, transients are dissipated by friction

Dissipation of transients



Joukowsky relation

$$\sum F = \rho Q(v_2 - v_1)$$

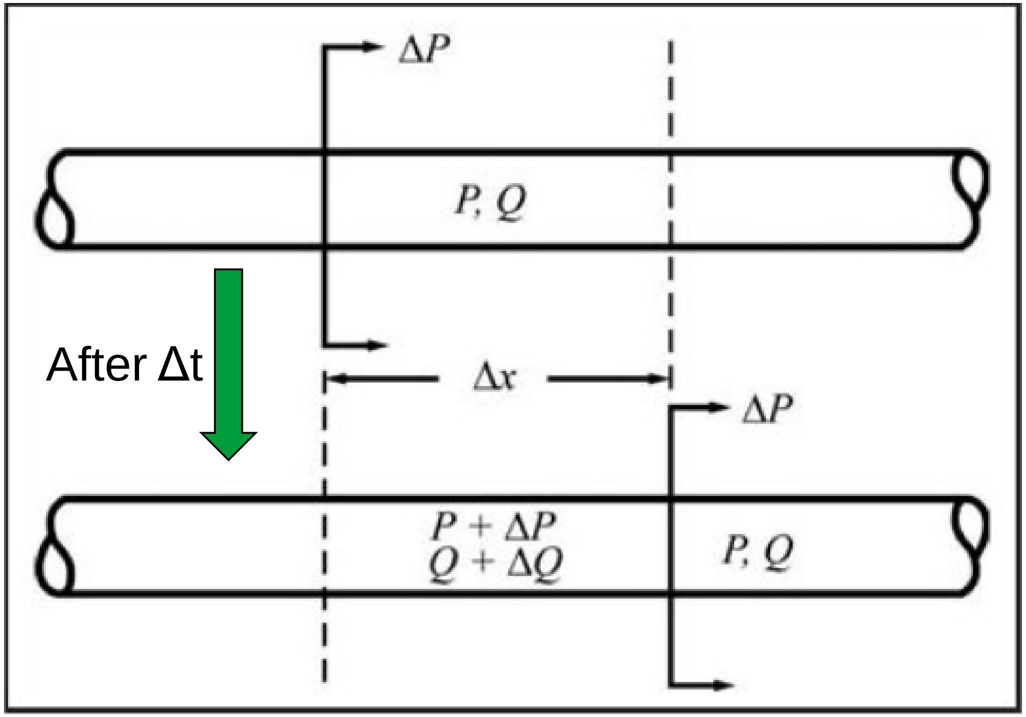
- According to the momentum principle,

$$\Delta PA = \rho \Delta x \, {}^{\Delta Q}/_{\Delta t}$$

$$\Delta P = \rho c \Delta v$$

where c=sonic speed (depends on pipe material and elasticity)

Δv = change in velocity



- Very simplified, applies only to simple cases where the valve is closed quickly compared to the time required for a pressure wave to travel the length of the pipe

Transient conditions mitigated through

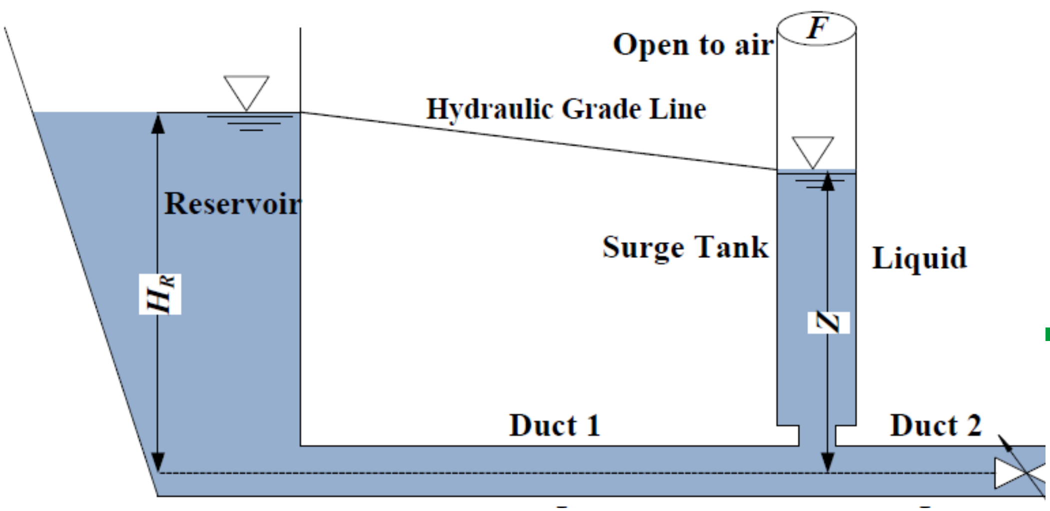
- Higher pressure class pipes
- Rerouting of pipes
- Improvement in valve and pump control/operation procedures
- Limiting the pipeline velocity
- Reducing the wave speed (e.g., different pipe material)
- Surge protection devices

Surge protection devices

- Minimize flow fluctuations e.g. by
  - delaying the change of flow (e.g. storing water)
  - discharging water from the line
- For both minimum & maximum pressures: surge tanks (e.g. storage tanks), air chambers, pump bypass lines
- Controlling maximum pressures:
  - pressure-relief valves, surge anticipation valves, or combinations
- Controlling minimum pressures:
  - increasing pump inertia, air-release/vacuum valves, or combinations

Open surge tank (aaltoilusäiliö)

- convert kinetic energy into potential energy
- at locations where normal static pressure heads are small (or tall tanks are acceptable)
- prevent both high and low pressures and cavitation
- water towers

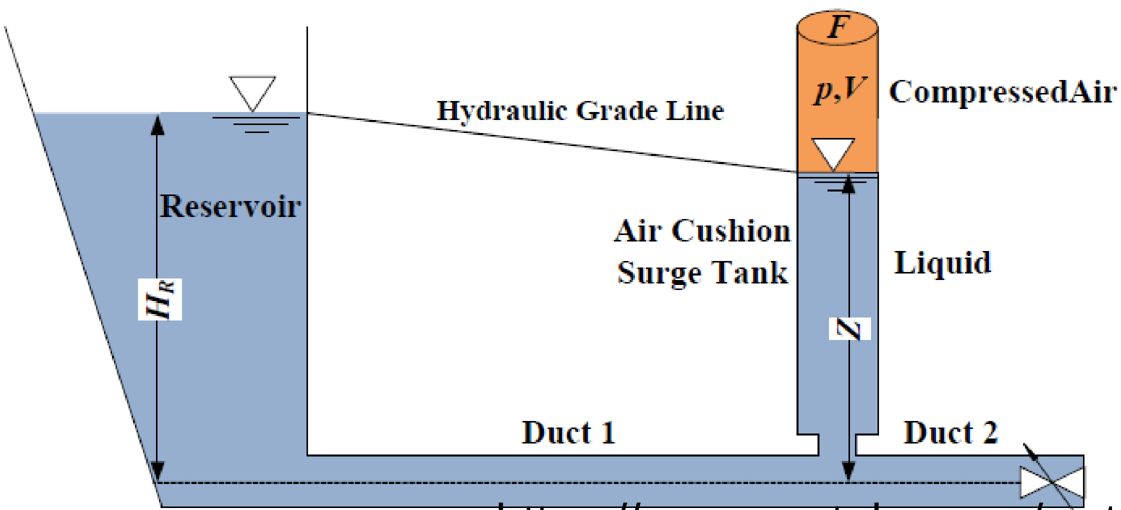


https://youtu.be/HIK5LYvDqYA

(Wang et al., doi:10.3390/w7084446)

Air chamber/ surge vessel/ closed surge tank

- a chamber in which air elastically compresses and expands to regulate the flow
- typically positioned downstream of pumps to protect the pumps against trips but can be installed anywhere along a line regardless of normal pressure head
- respond faster and allow a wider range of pressure fluctuation than open surge tanks



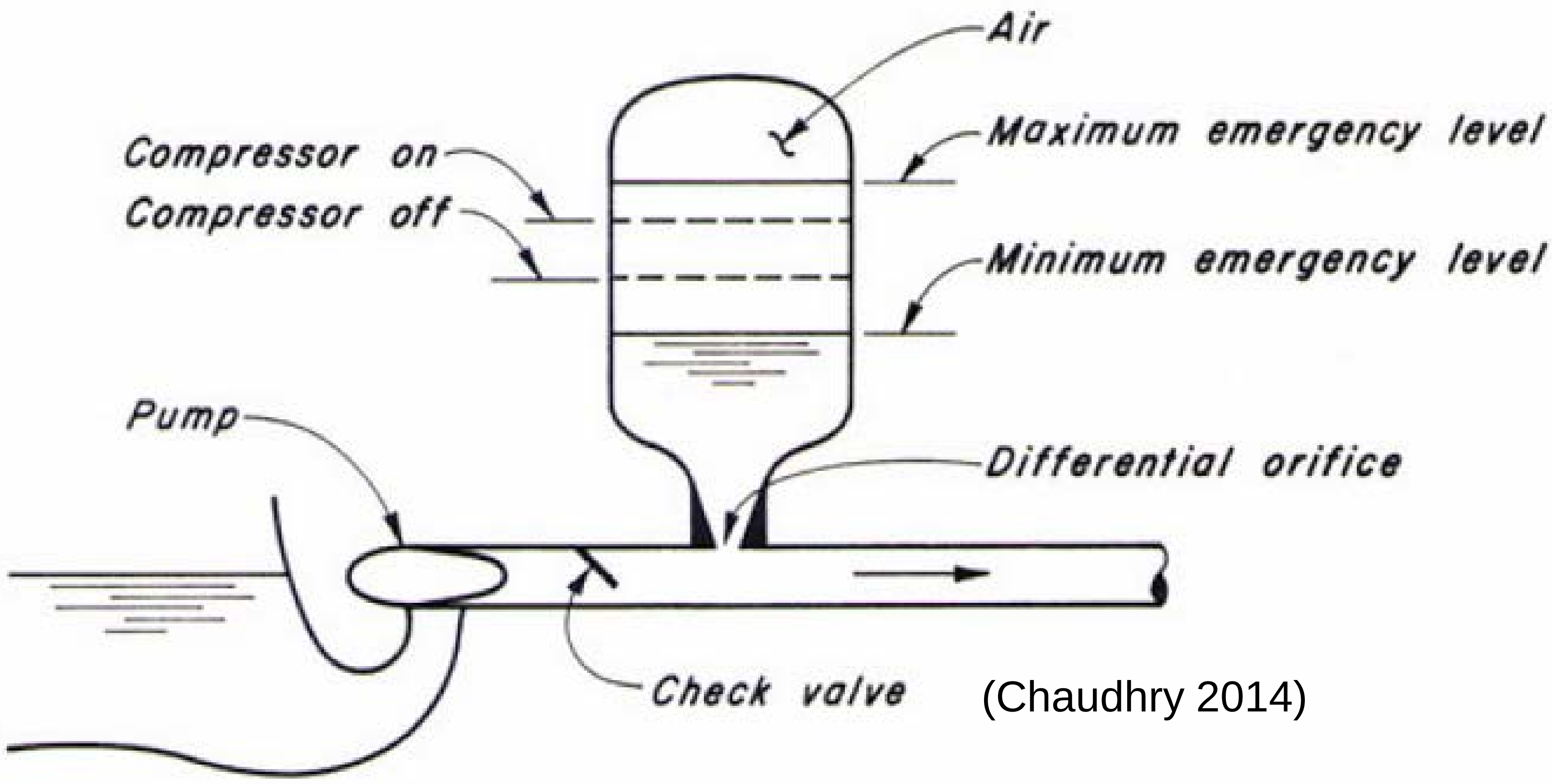
(Wang et al., doi:10.3390/w7084446)

https://www.youtube.com/watch?v=YgjRZq70GR4

Check valves (takaiskuventtiili)

- allow flow only in one direction and closes when flow reversal is impending
- a substantial backflow may occur before closure
- together with other devices
- can be used to isolate high pressure waves from reaching a section of a pipeline

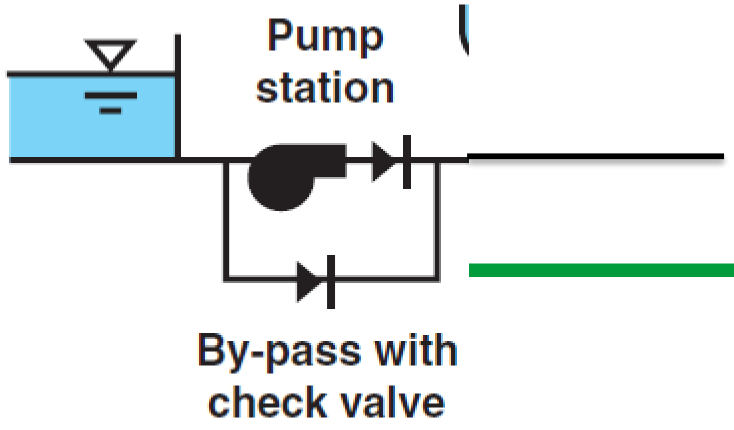
Air chamber at pump discharge side



(Chaudhry 2014)

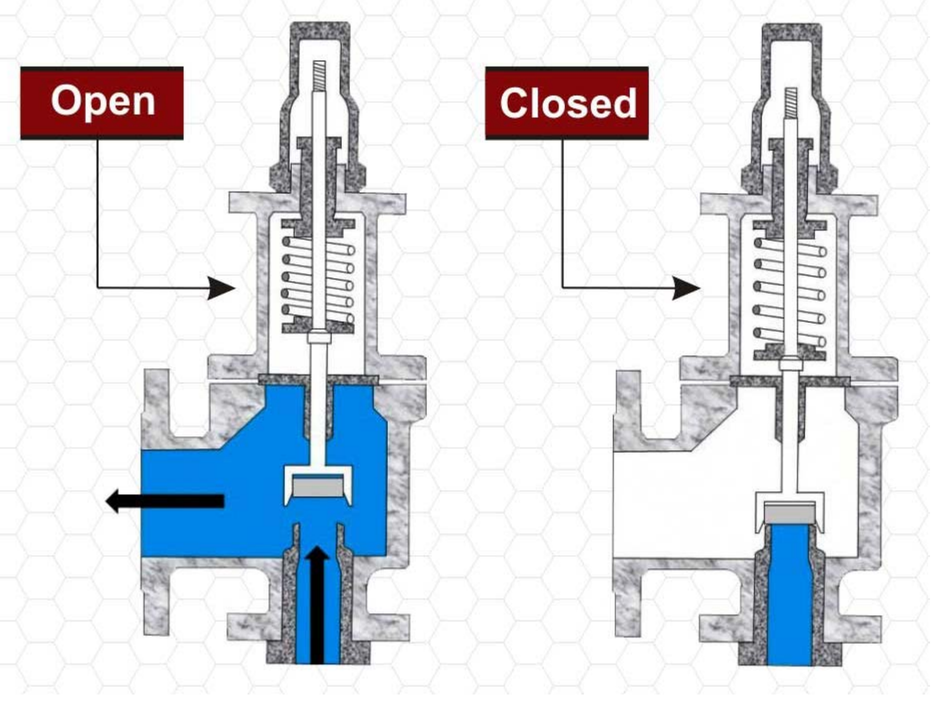
Pump bypass lines (ohitukset)

- can be installed around the pumps in low-head pumping systems that have a positive suction head
  - allow water to be drawn into the discharge line following power failure
  - activated when the pump suction head exceeds the discharge head
- prevent high-pressure buildup on the pump suction side and cavitation on the pump discharge side



Pressure-relief valves (varo/ylipaineventtiili)

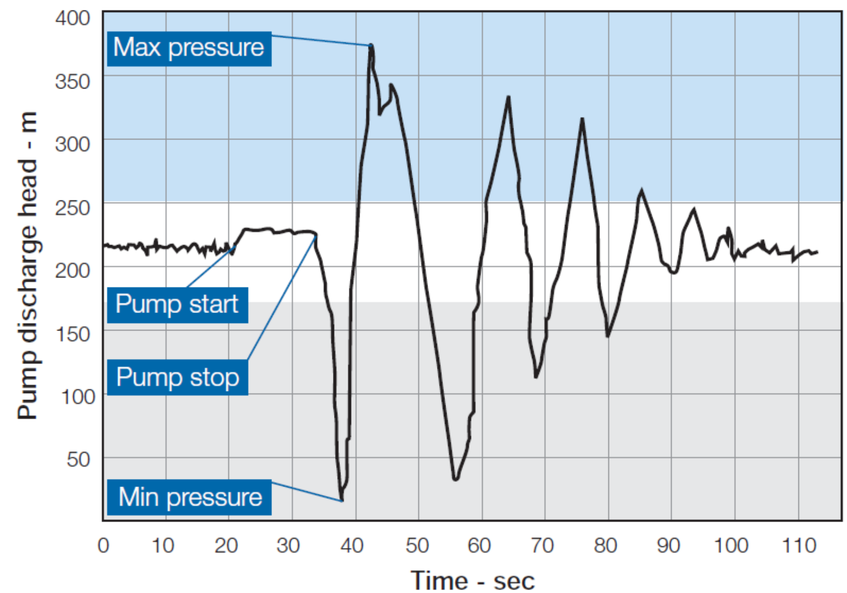
- eject water out of the system when the pressure reaches a preset value -> prevent excessive high-pressure surges
- open and close at prescribed rates over which the designer may have some control

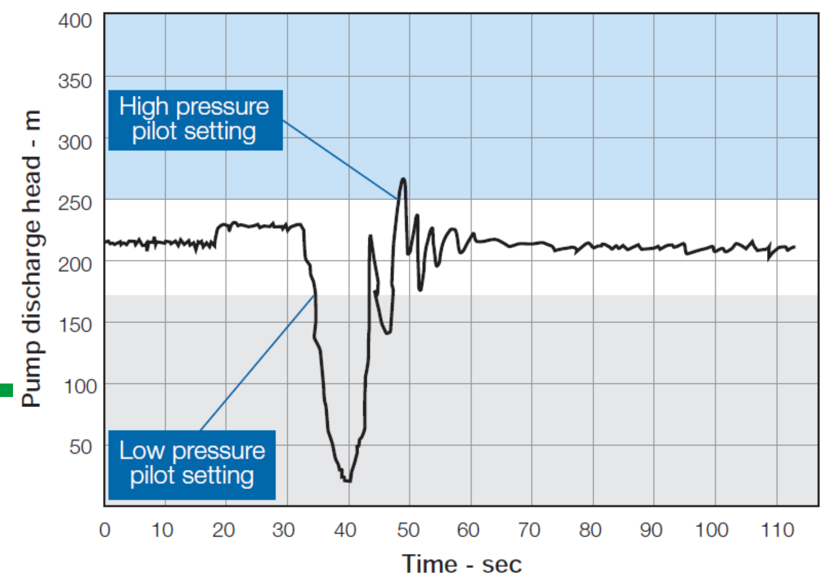


(Industrial Valve Store)

Surge anticipation valves

- similar to pressure relief valves, but also open when a downsurge occurs -> accommodate the following upsurge
- complete a cycle of opening and closing
- can solve the problem of upsurge at the pump due to reverse flow or wave reflection
- can worsen low pressure conditions



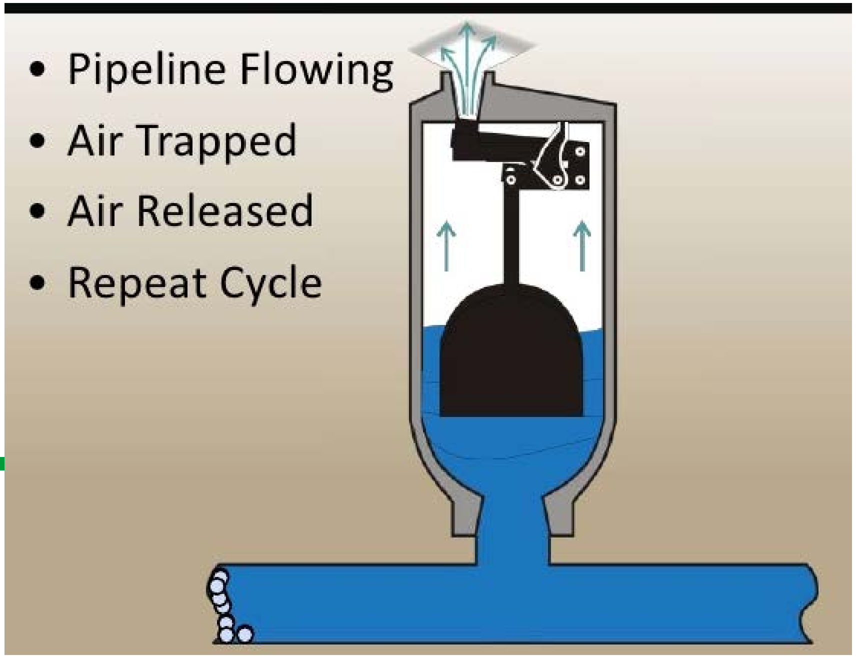


Vacuum + air-release valves

- Can be installed at high points in a pipeline
- Vacuum valves admit air into the pipe when pressure drops below atmospheric

-> prevent cavitation

- Air release valves expel air when pressure exceeds atmospheric pressure

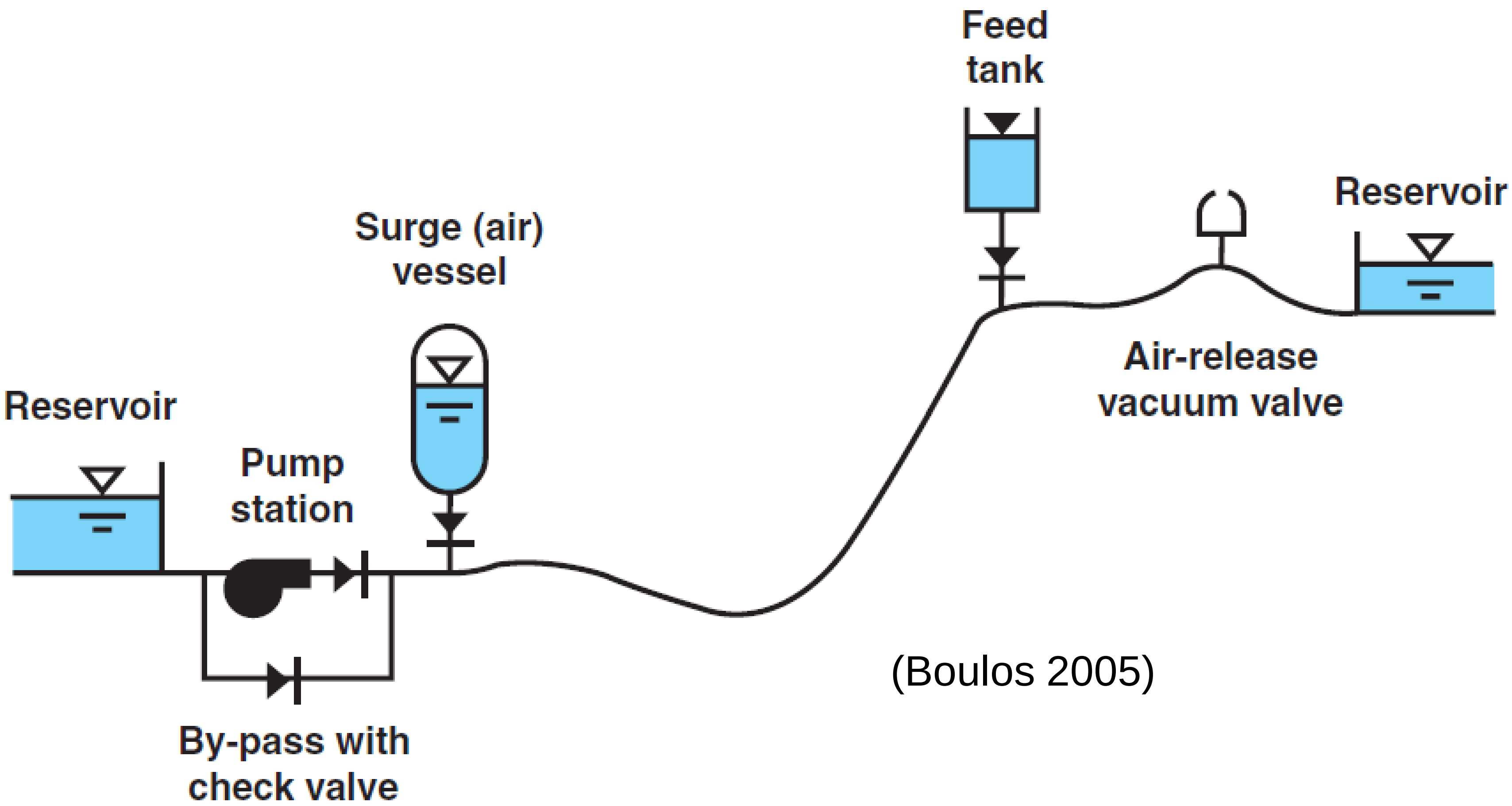


Control of change in pump speed

- frequency controlled pumps (taajuusmuuttaja)
  – have variable frequency drives = a system for controlling the pump speed

- flywheels = e.g. a large-diameter steel plate attached to the pump motor
  – the provided rotational energy reduces pump speed gradually

Locations for surge control devices



(Boulos 2005)

Further materials…

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