Particle removal

Characterization of particles
Flocculation – Sedimentation – Filtration

Urs von Gunten

Would you drink this water?

drinking water?

For 1.1 billion people in the developing world,
this might be as good as it gets.
Unsafe water kills **6,000 people every single day.**

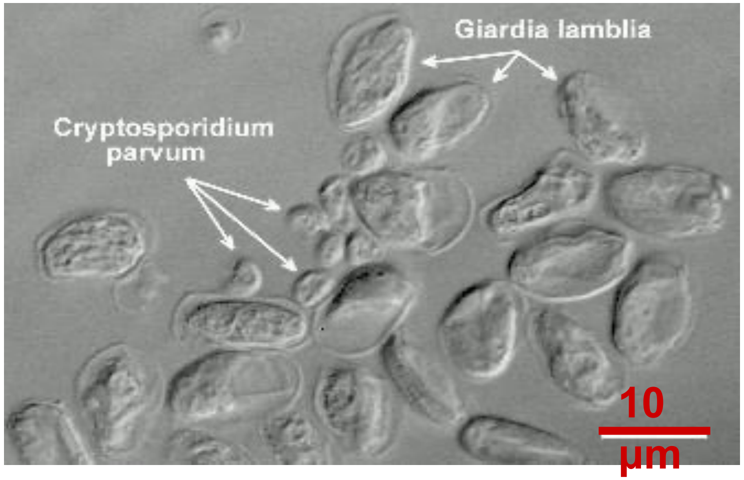
Learning objectives

- Calculation of particle size distributions from measured data
- Understanding coagulation/flocculation processes
- Assess sedimentation of particles and dimension sedimentation tanks
- Calculate head loss in media filtration systems

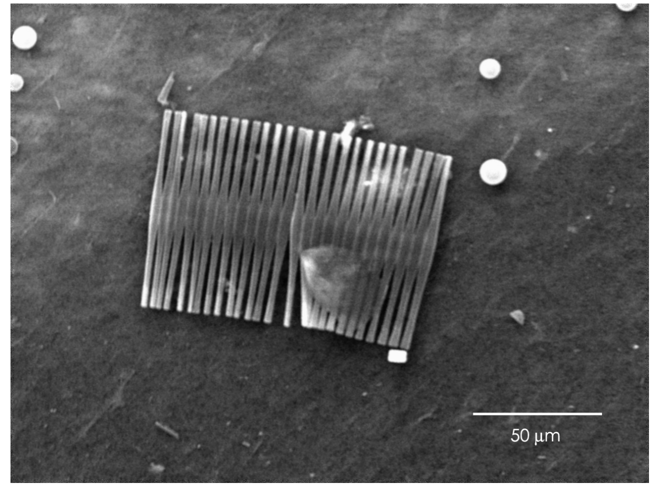
Particles in water resources

Biology

Microorganisms

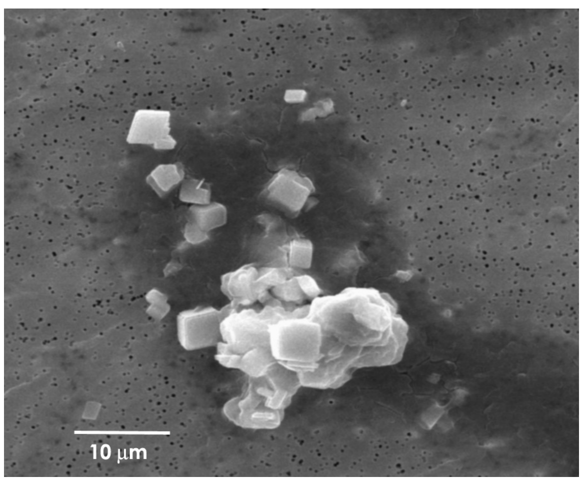


Algal material

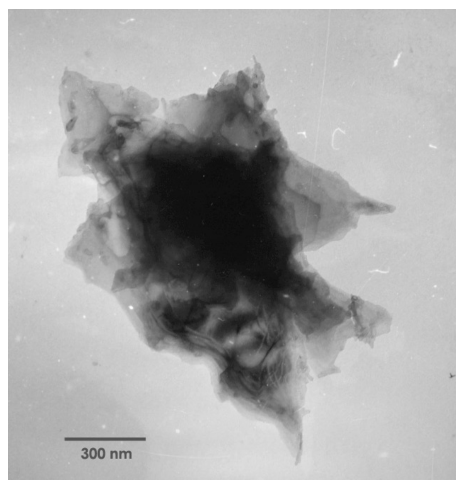


Minerals

Calcite

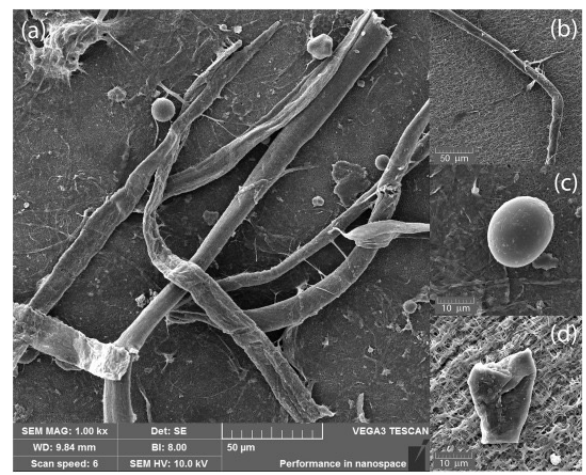


Clay

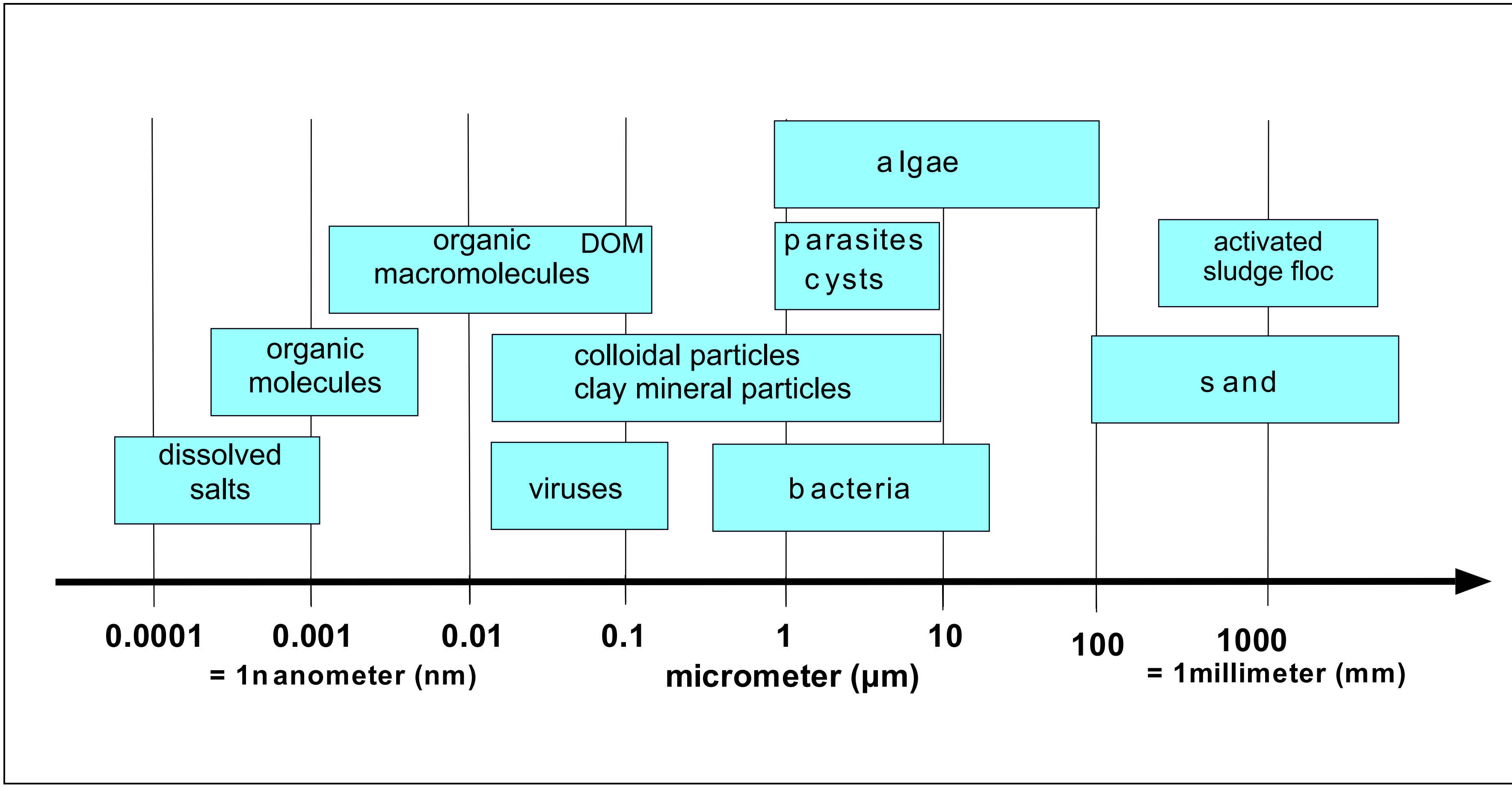


Anthropogenic contamination

Micro-/Nanoplastic

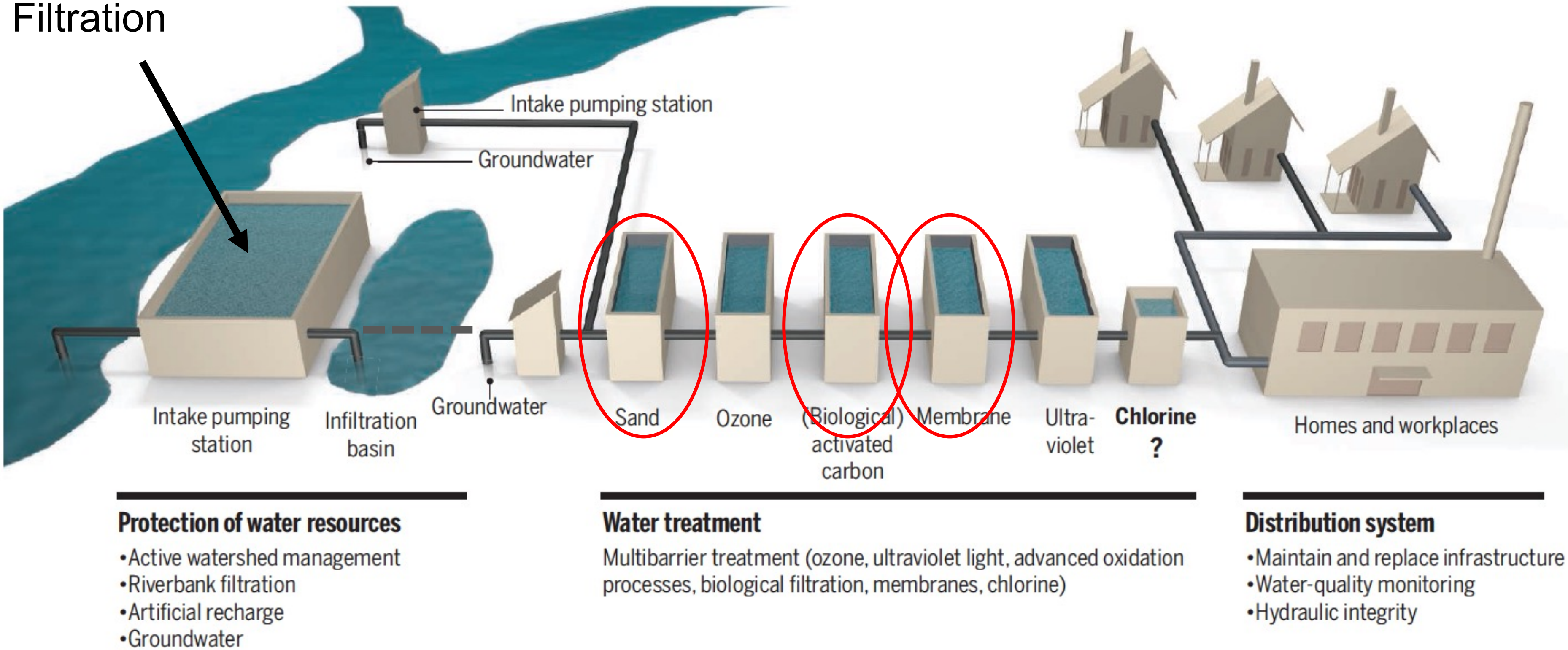


Size categories of particles



Treatment trains including particle separation

Flocculation
Sedimentation
Filtration



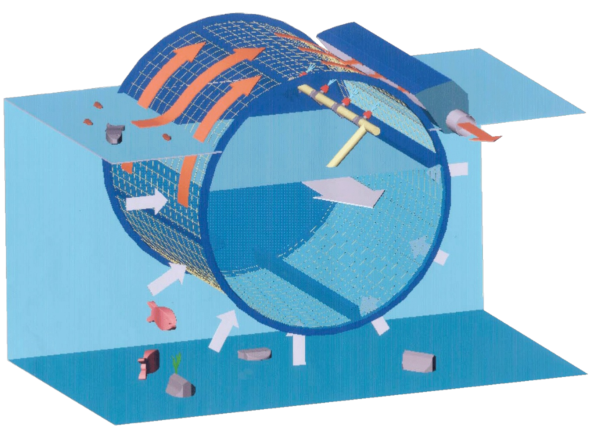
Article on moodle

Rosario-Ortiz, F., Rose, J., Speight, V., von Gunten, U. and Schnoor, J. (2016) How do you like your tap water? Science 351(6276), 912-914.

Solids separation processes applied in water treatment

- Riverbank filtration
- Artificial recharge
- Screening, straining
- Sedimentation
- Flocculation
- Flotation
- Granular media filtration
- Contact filtration
- Membrane filtration

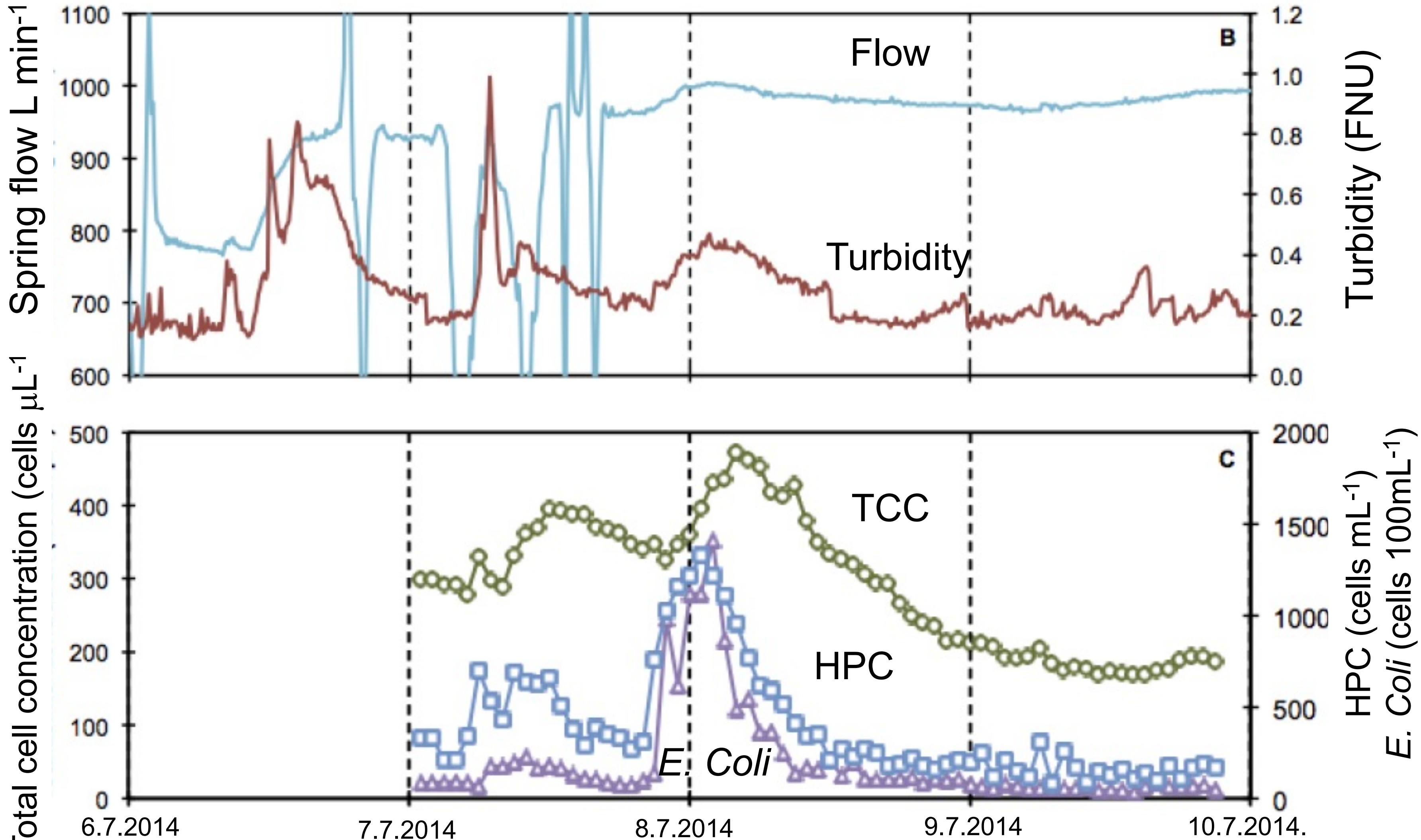
Micro strainer – drum filter



Definitions

- Total solids (TS): mass after drying the water sample (including mass of dissolved matter) in gTS/m^3
- Total suspended solids (TSS): expressing the dry weight of filtered solids mass (0.45 µm) in gTSS/m^3
- Turbidity: Light absorption or light scattering through a water sample; compared to standard turbidity of defined particles expressed as NTU (nephelometric turbidity), FTU (formazine turbidity) and others. Turbidity < 0.2 NTU is not visible to the human eye
- Drinking water standard typically 1 FTU, 0.2 recommended
- Particle number: Number of particles per volume (＃/mL), usually determined with laser diffraction methods

Dynamics of a karstic spring: Comparison between particles and microbiological parameters

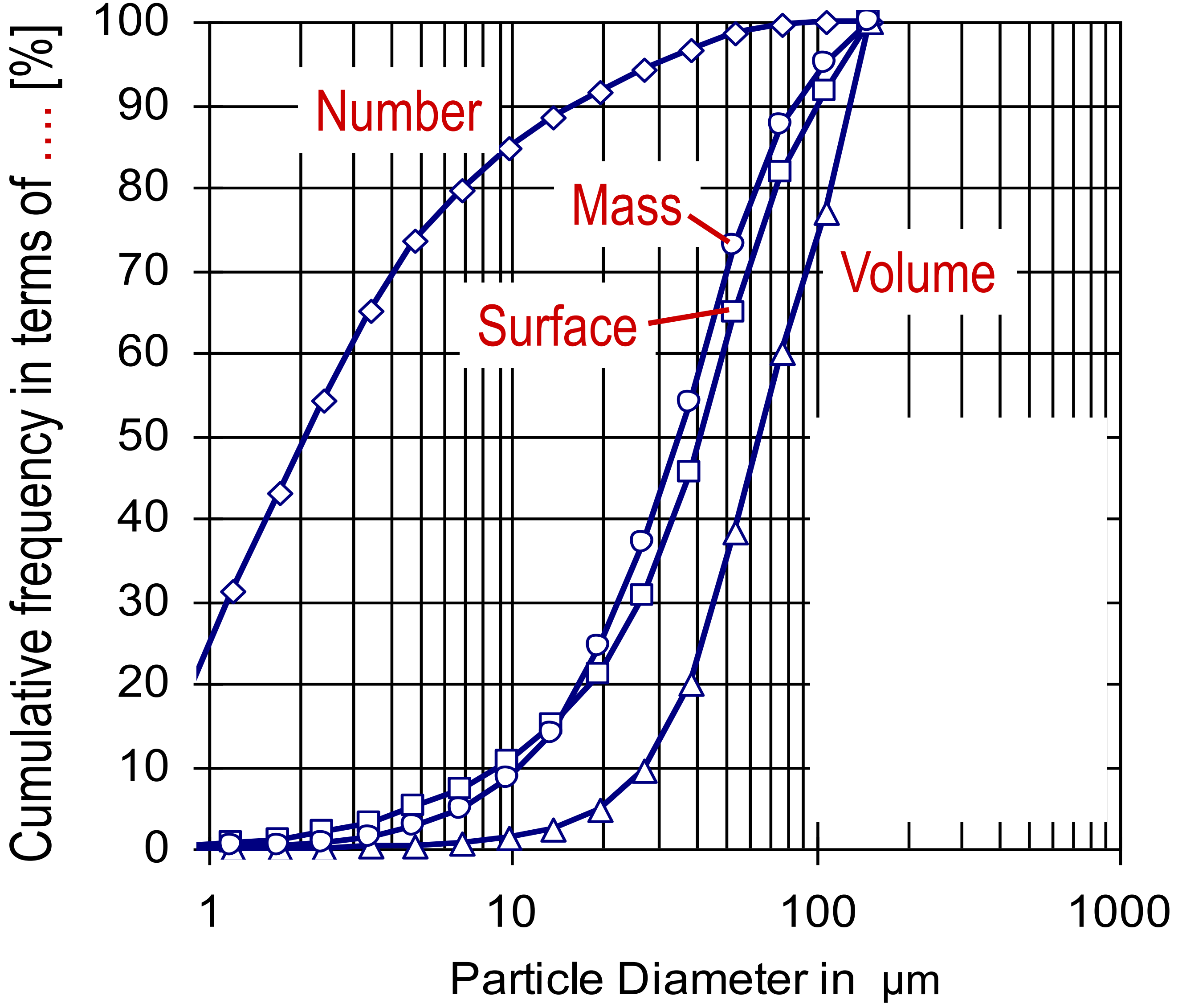


Wissenschaftlicher Bericht TP1, Regionale Wasserversorgung BL 21, 2016

Quantifying particles

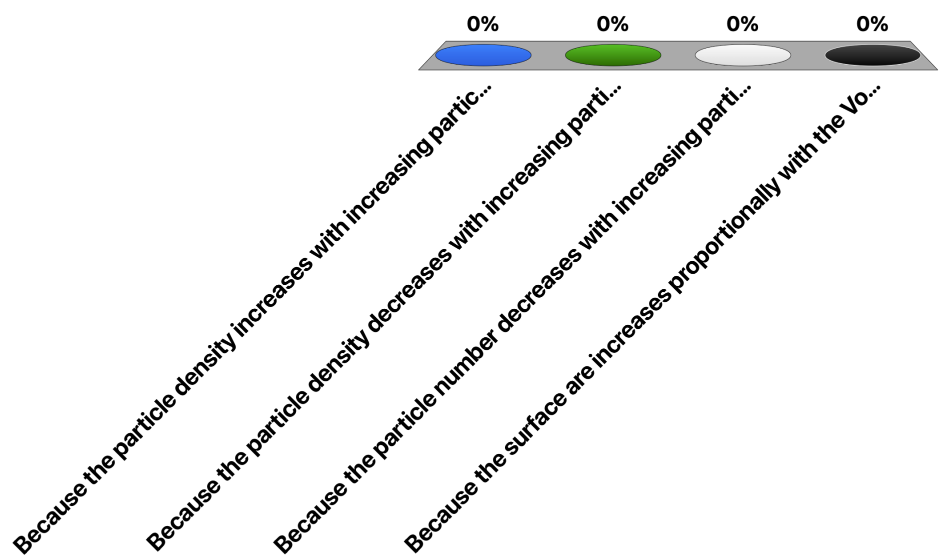
Particle **mass** →	Suspended solids
Particle **volume** →	Particle load of filters resulting in headloss buildup
Particle **surface** →	Adsorption capacities of heavy metals, synthetic organics
Particle **number** →	Hygienic criteria (e.g., number of pathogenic organisms)

Cumulative size distribution of particles in secondary wastewater effluent: *Number, surface, volume, and mass of particles*

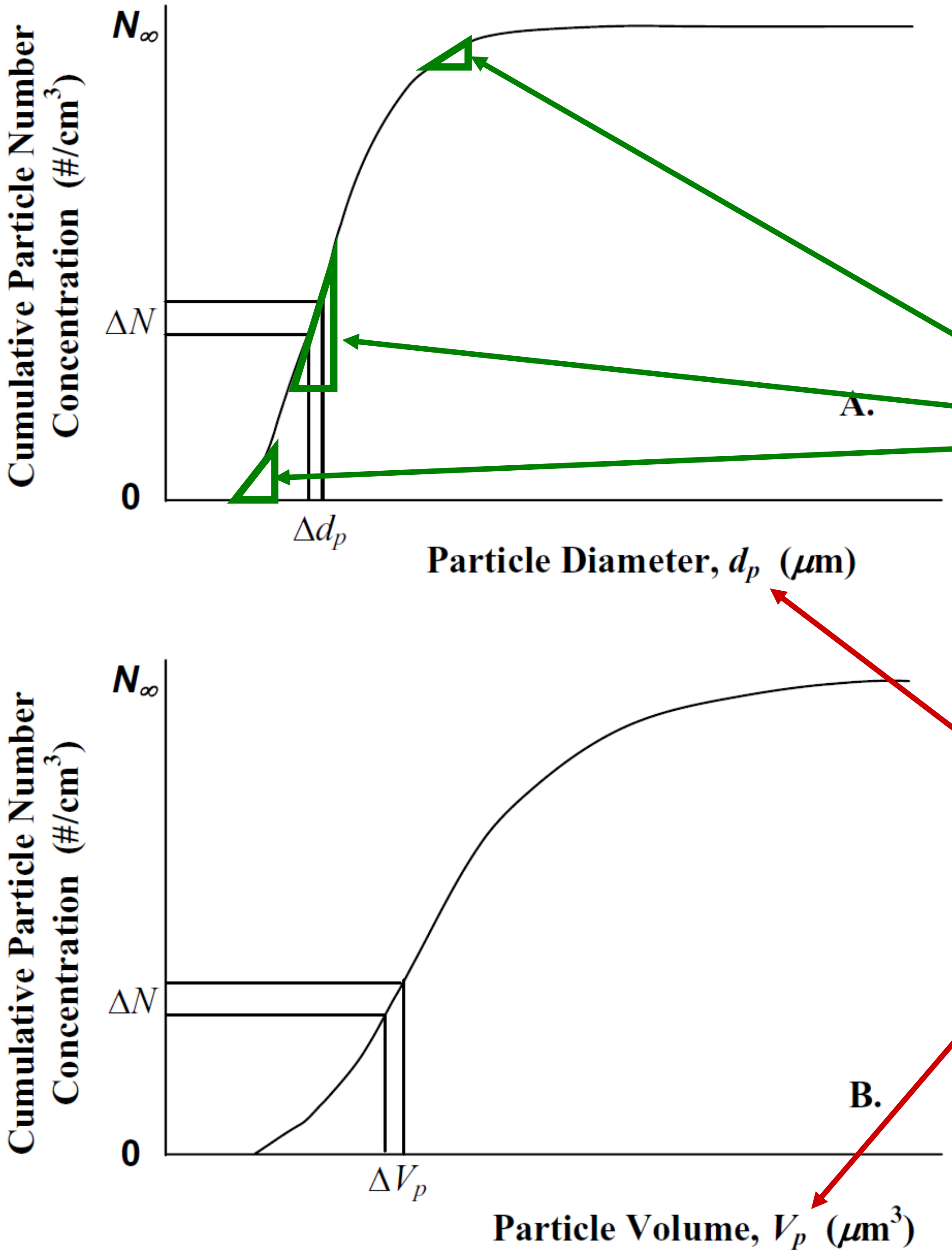


Why are the mass and the volume distribution not the same?

A. Because the particle density increases with increasing particle size
B. Because the particle density decreases with increasing particle size
C. Because the particle number decreases with increasing particle size
D. Because the surface are increases proportionally with the Volume



Cumulative particle size distributions based on number – size expressed as diameter or volume



Frequency distribution can be calculated from slopes

Particle size can be expressed as diameter or as volume

Lawler, D.F. and Benjamin, M.M. (in progress) Water Quality Engineering: Physical-Chemical Treatment Processes.

Size frequency distribution of particles

Size distribution of particles can empirically be described as a power law:

$$F_N\left(d_p\right) = \frac{\Delta N}{\Delta d_p} \approx A \cdot d_p^{-\beta}$$

log – form:

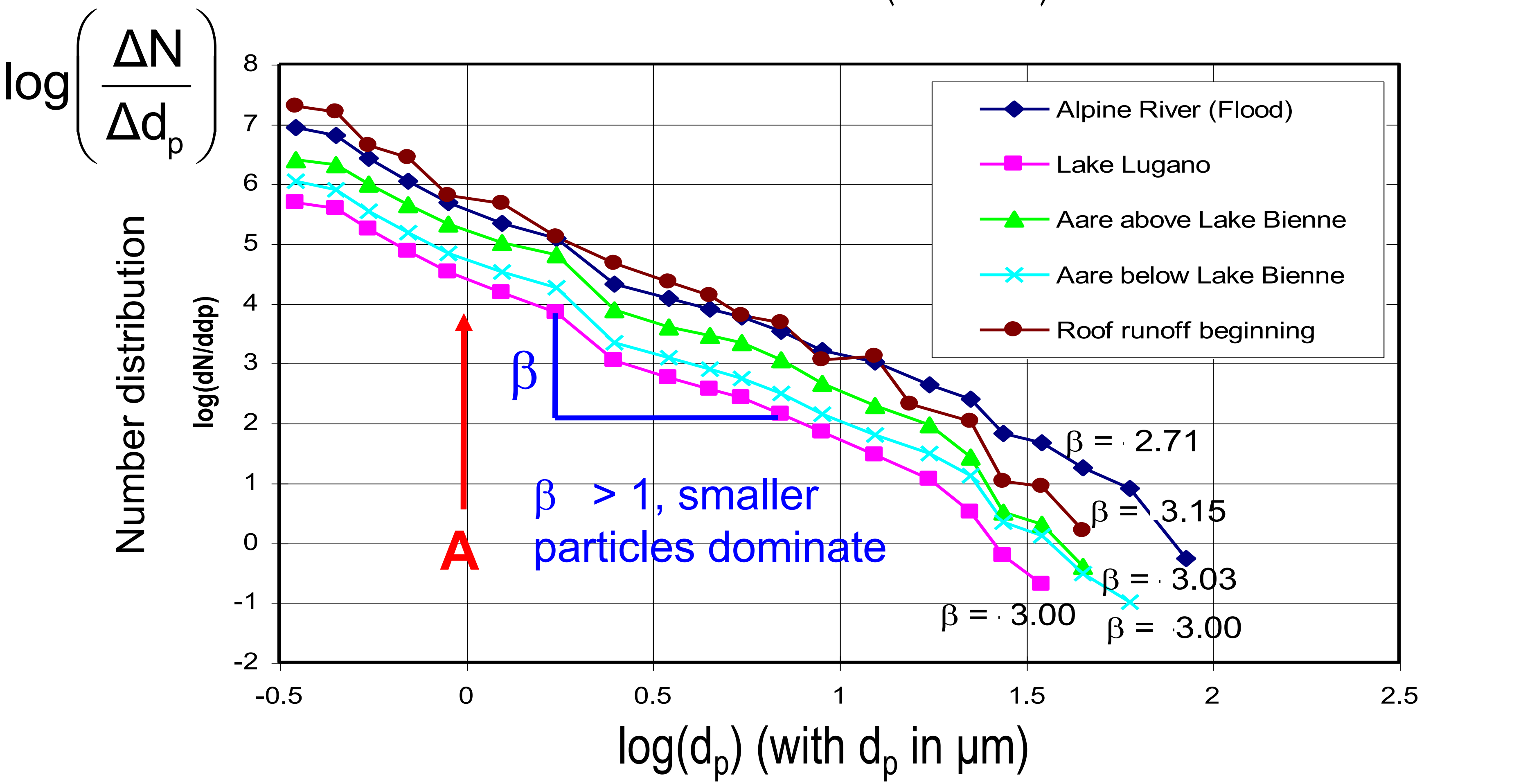
$$\log\left(F_N\left(d_p\right)\right) = \log\left(\frac{\Delta N}{\Delta d_p}\right) = \log A - \beta \cdot \log\left(d_p\right)$$

N: Particle number per volume, e.g. #/mL
d_p: particle size (usually diameter), e.g. in μm
A, β: constants for distribution function

A: With increasing A, the total number of particles increases
β <1 large particles dominate
β =1 all particles sizes are equally represented
β >1 smaller particles dominate

Examples of particle pize distribution functions in natural waters

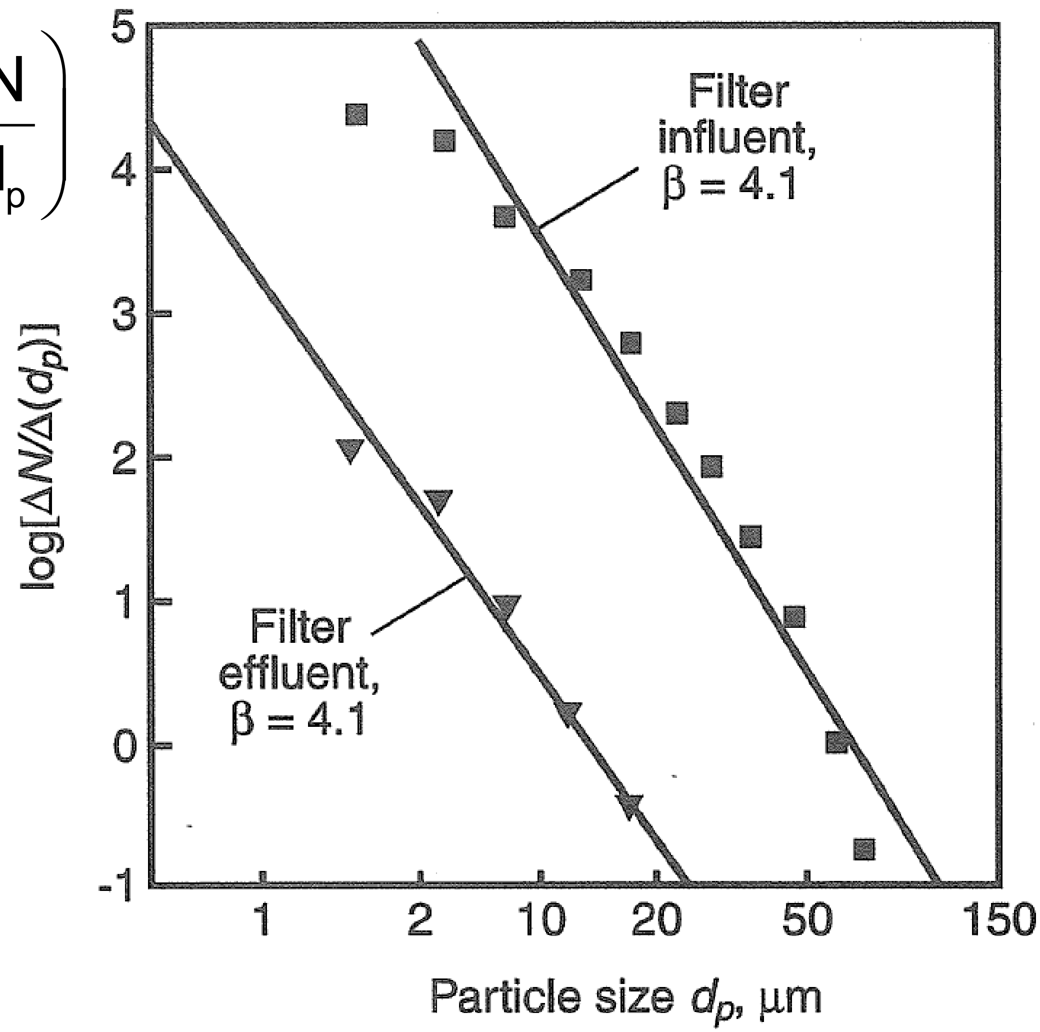
$$F_N(d_p) \approx A \cdot d_p^{-\beta} \longrightarrow \log\left(F_N\left(d_p\right)\right) = \log A - \beta \cdot \log\left(d_p\right)$$



Particle size distributions

Filter influent and effluent

$$\log\left(\frac{\Delta N}{\Delta d_p}\right)$$



Similar removal of all particle sizes by filtration

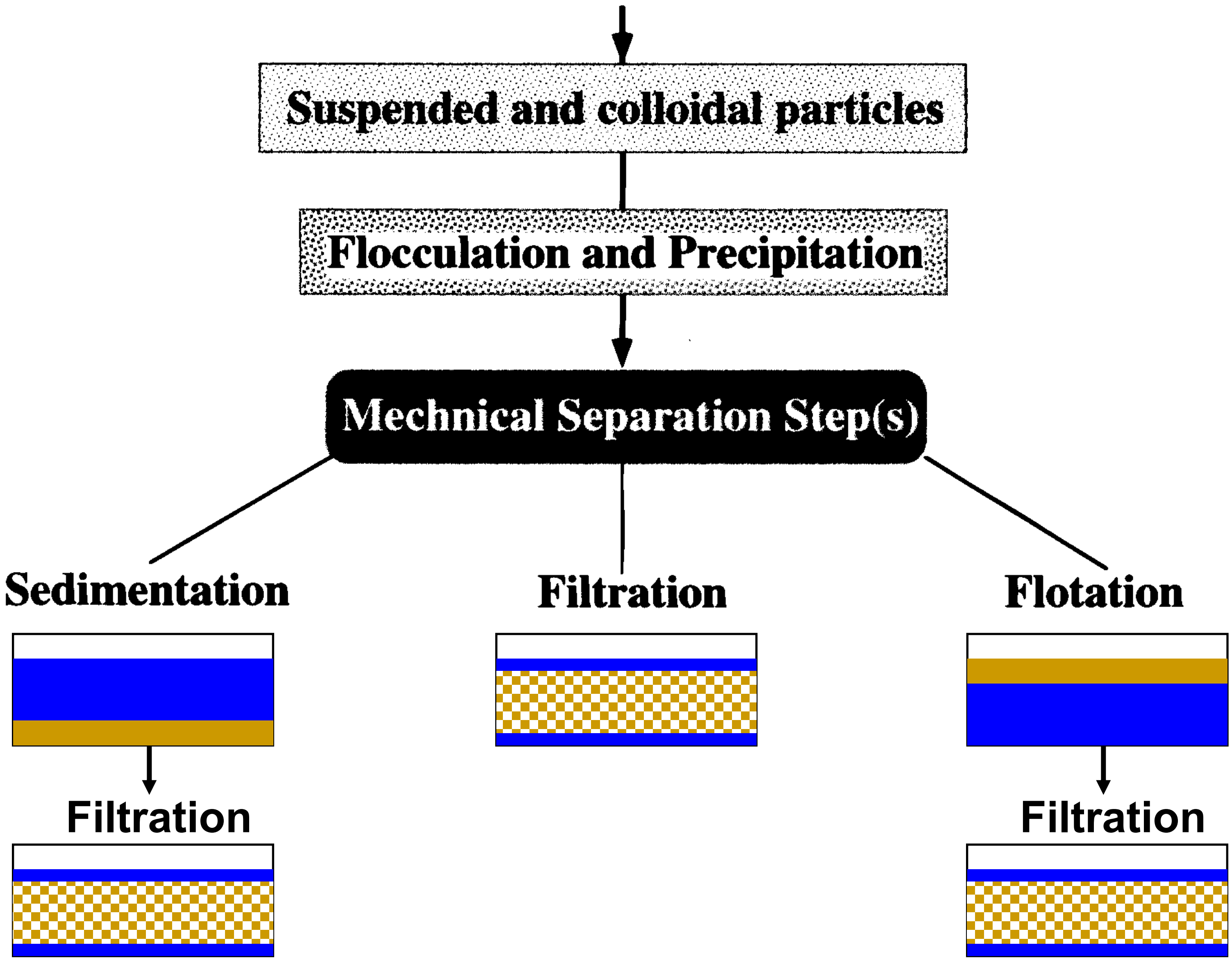
A gets smaller: number of particles decrease

$\beta > 1$, smaller particles dominate

β remains similar: particle distribution remains similar

Water Treatment: Principles and Design, Montgomery Watson Harza, John Wiley & Sons, 2005

Separation of particles from raw waters



Coagulation-Flocculation: Definitions

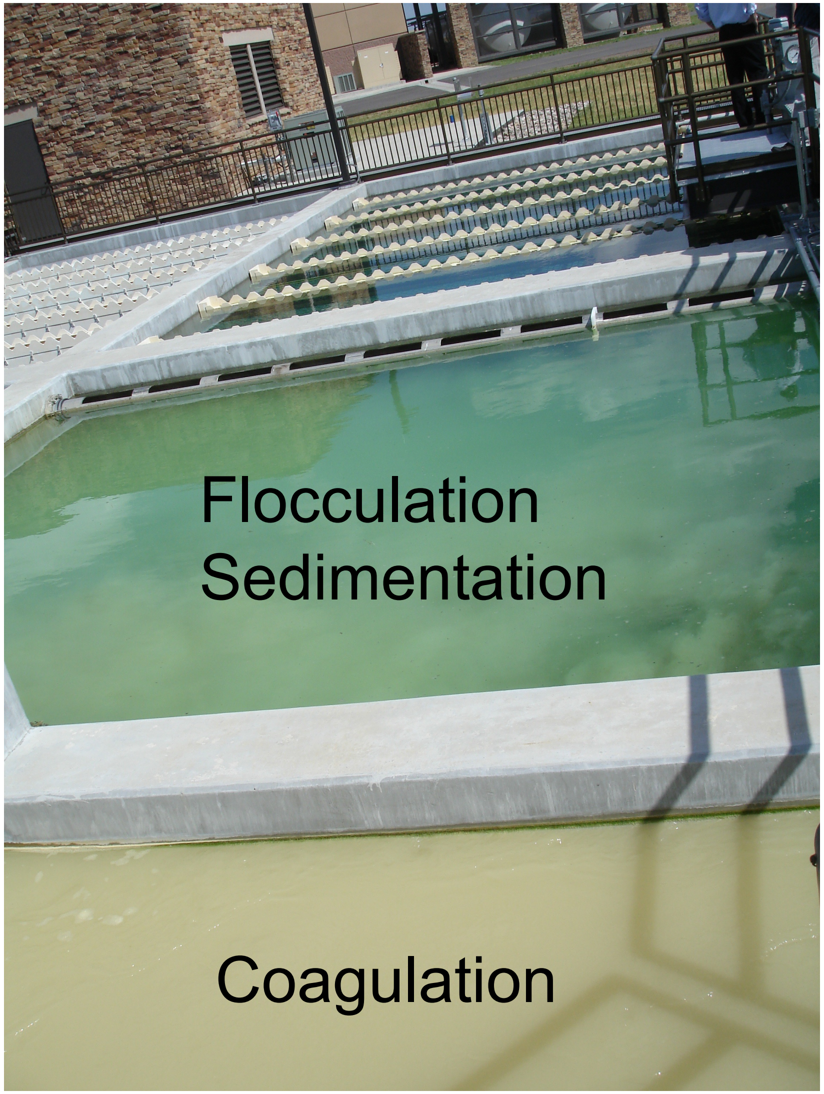
Coagulation:

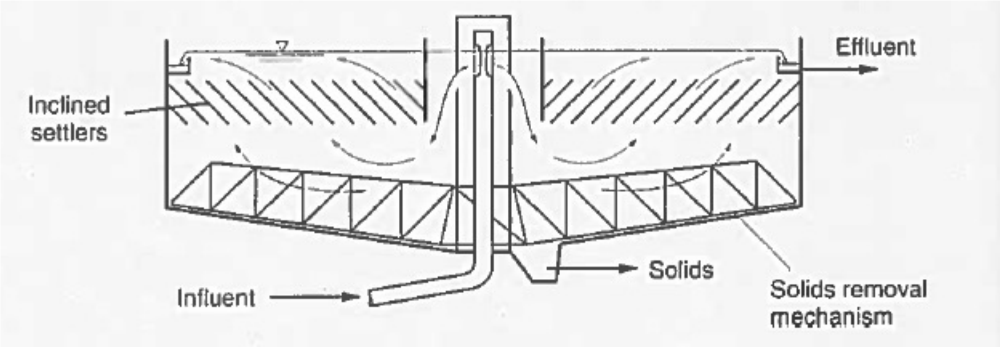
Condition particles, destabilize particles

Flocculation:

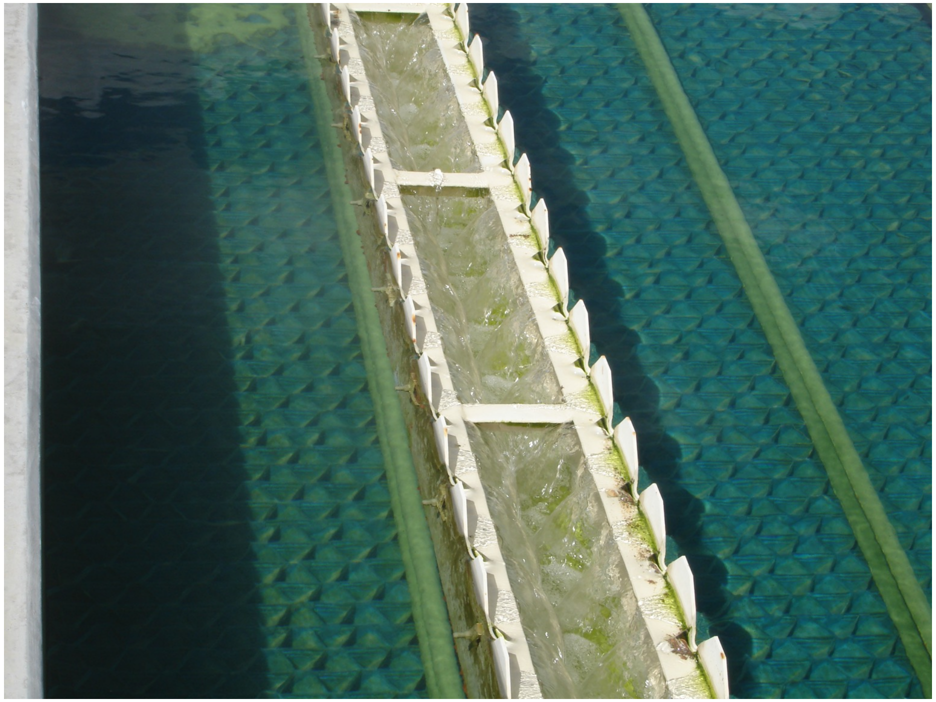
Aggregation of destabilized particles

Flocculation-Sedimentation

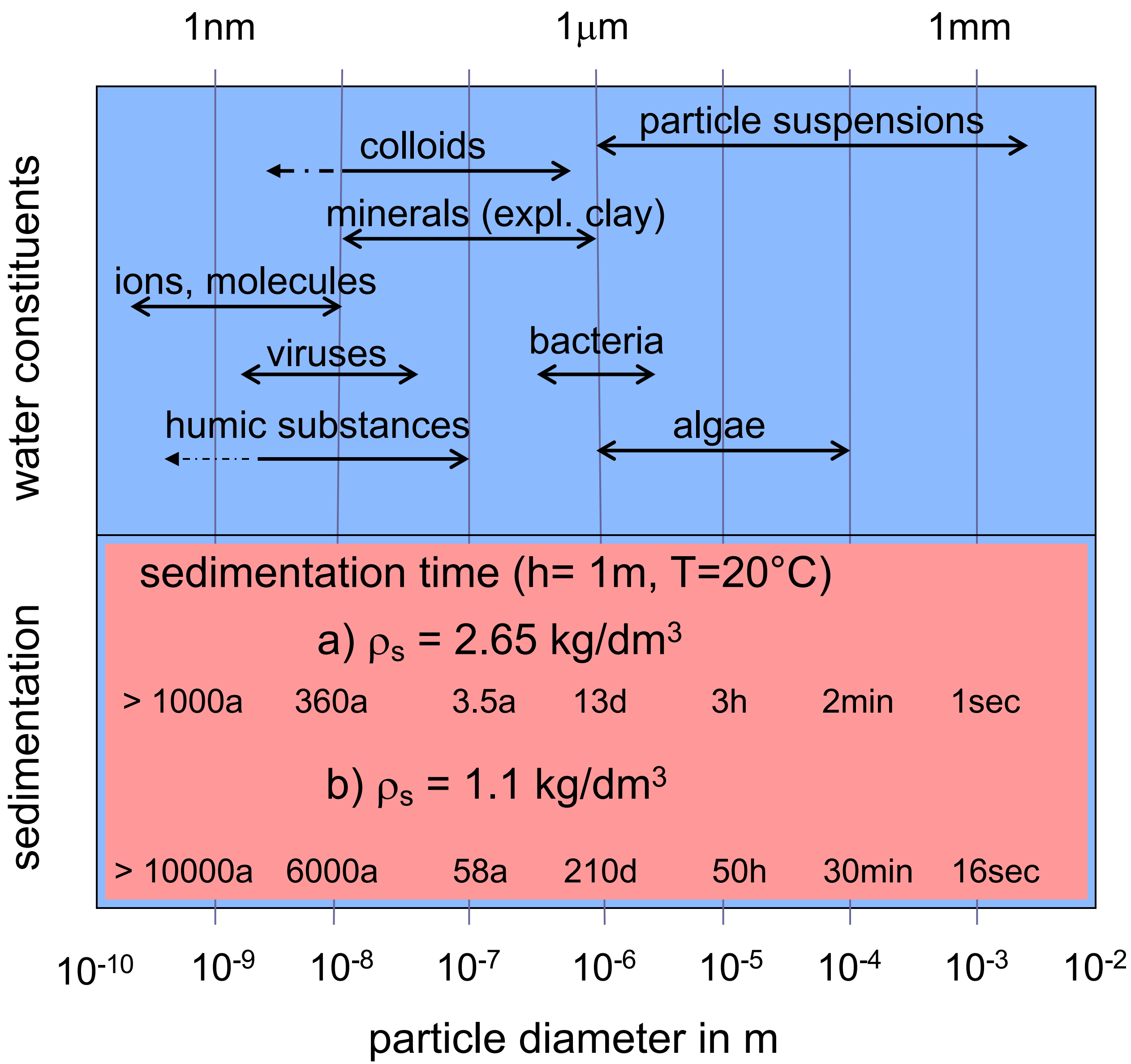




Upflow separation

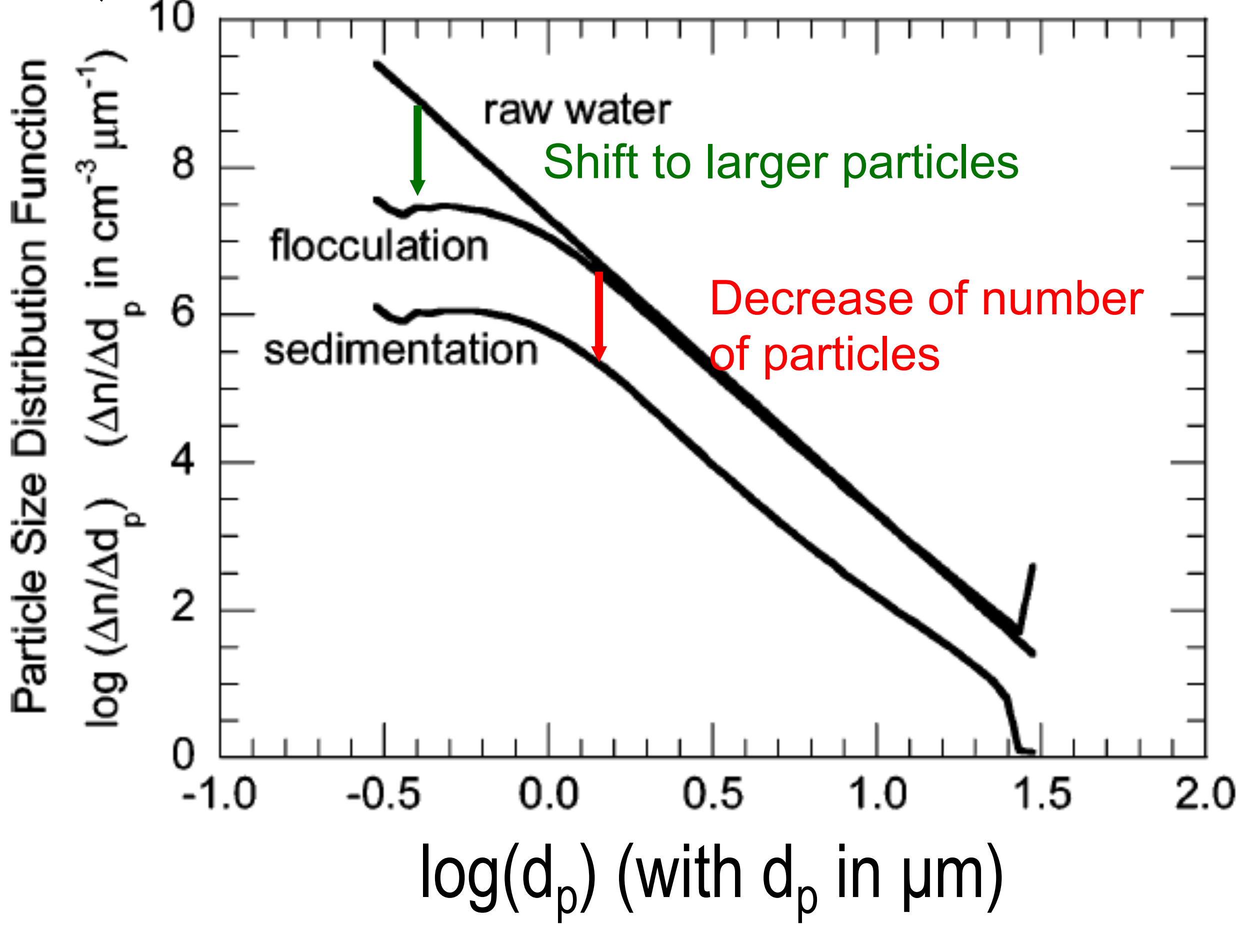


Sedimentation velocity of aquatic particles



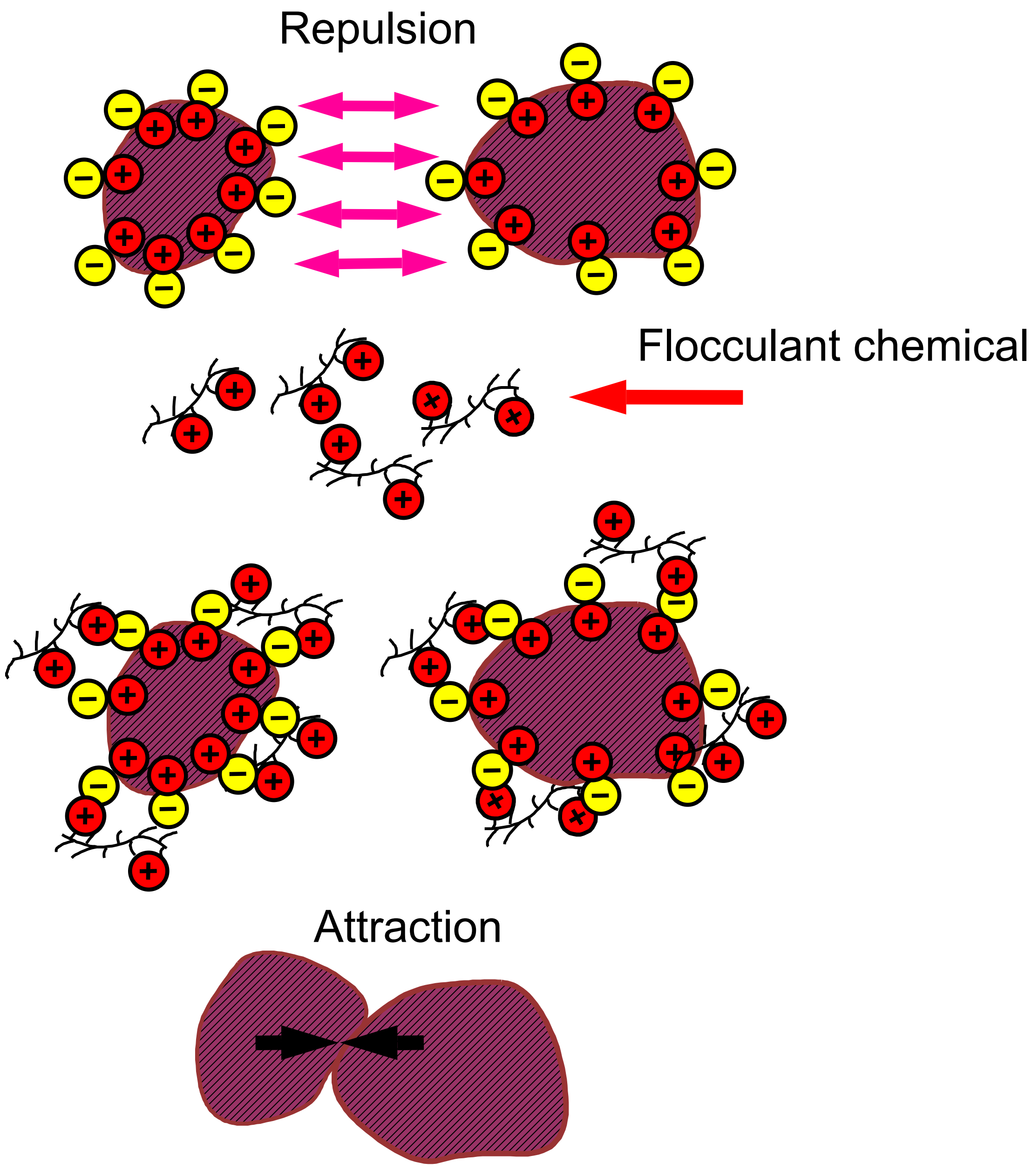
Desired effect of flocculation

$$\log\left(F_N\left(d_p\right)\right) = \log\left(\frac{\Delta N}{\Delta d_p}\right) = \log A - \beta \cdot \log\left(d_p\right)$$

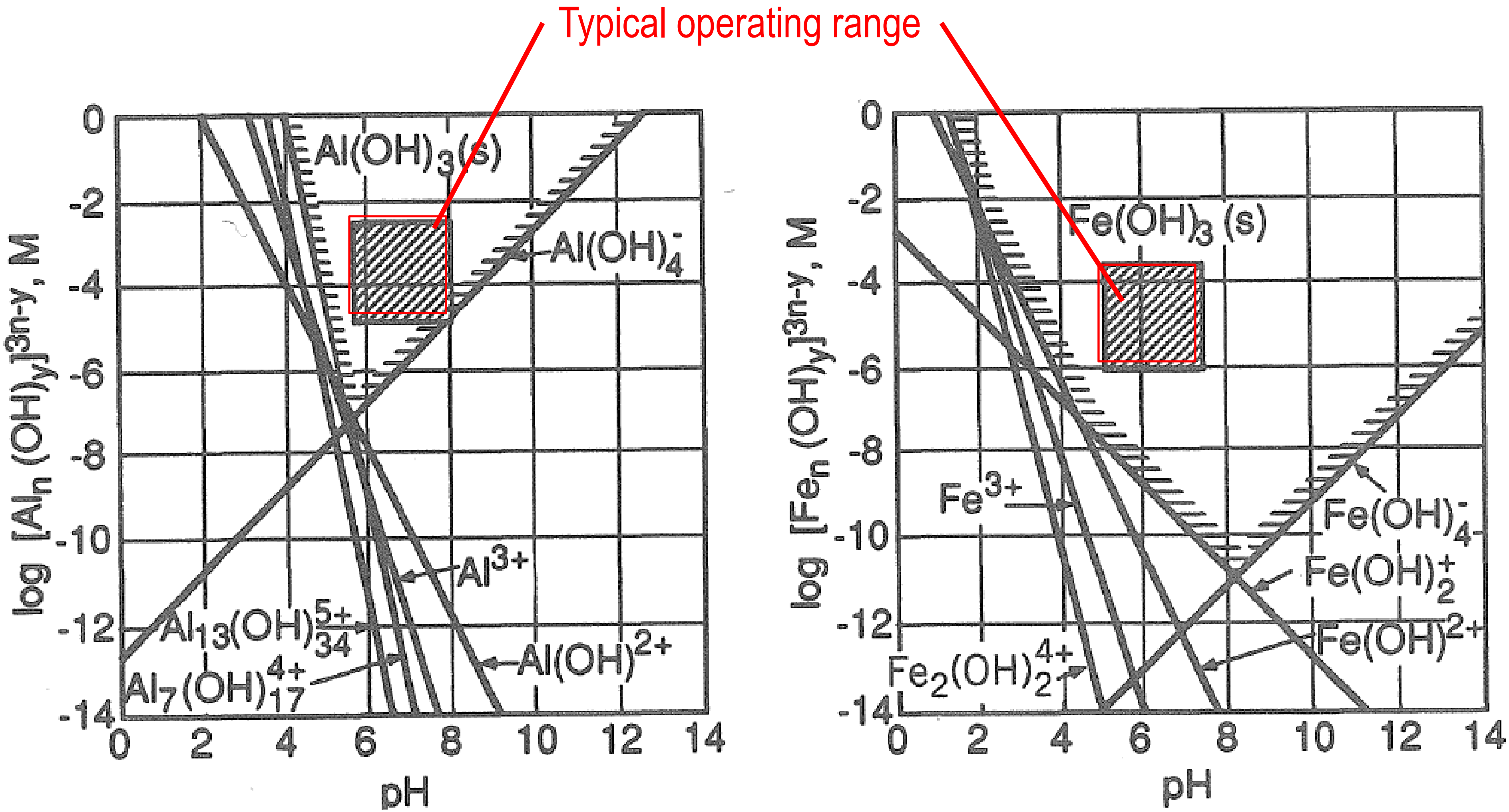


Lawler, D.F. and Nason, J.A. (2005) Integral water treatment plant modeling: Improvements for particle processes. *Environmental Science & Technology* 39(17), 6337-6342.

Aggregation of particles by flocculants

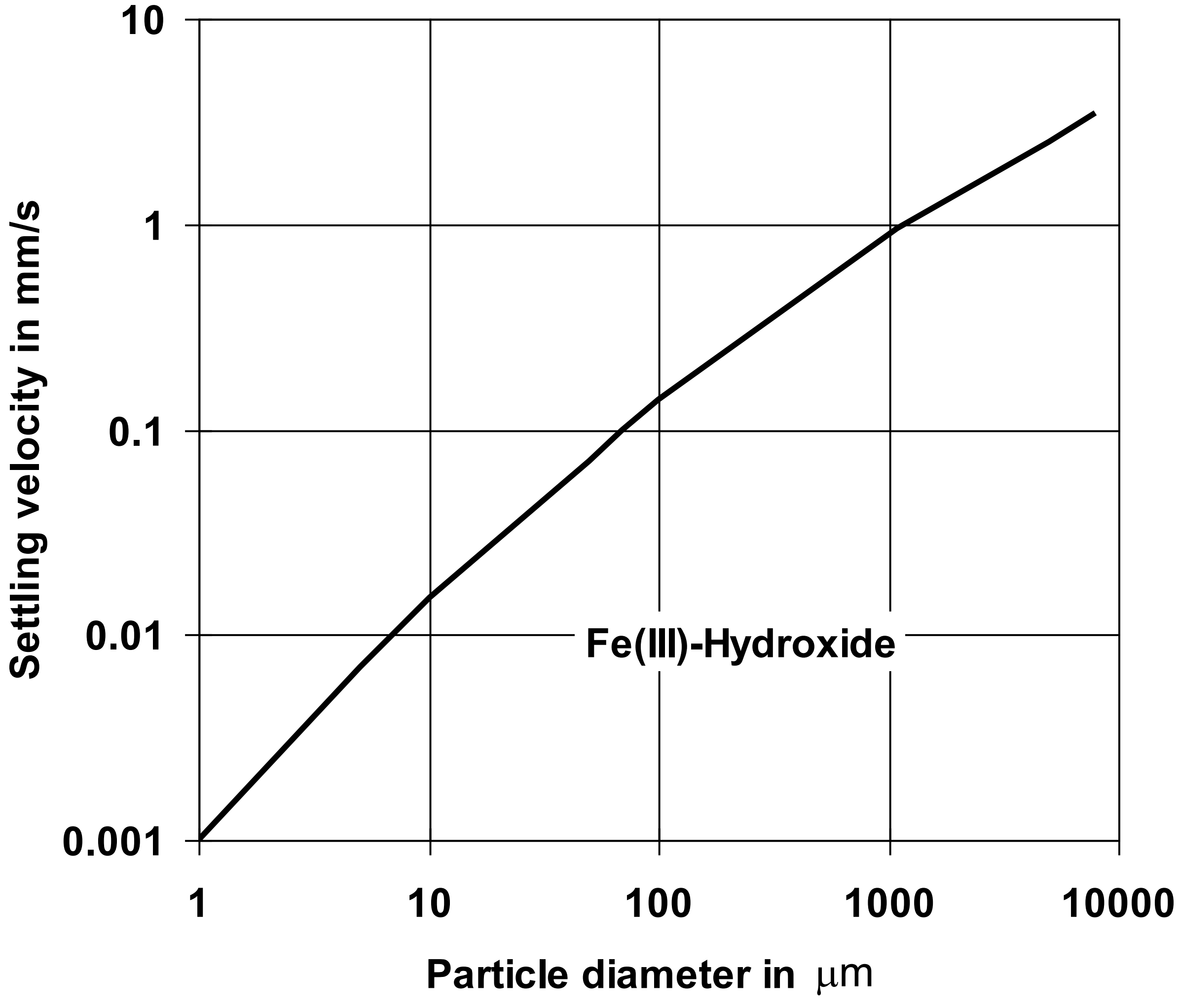


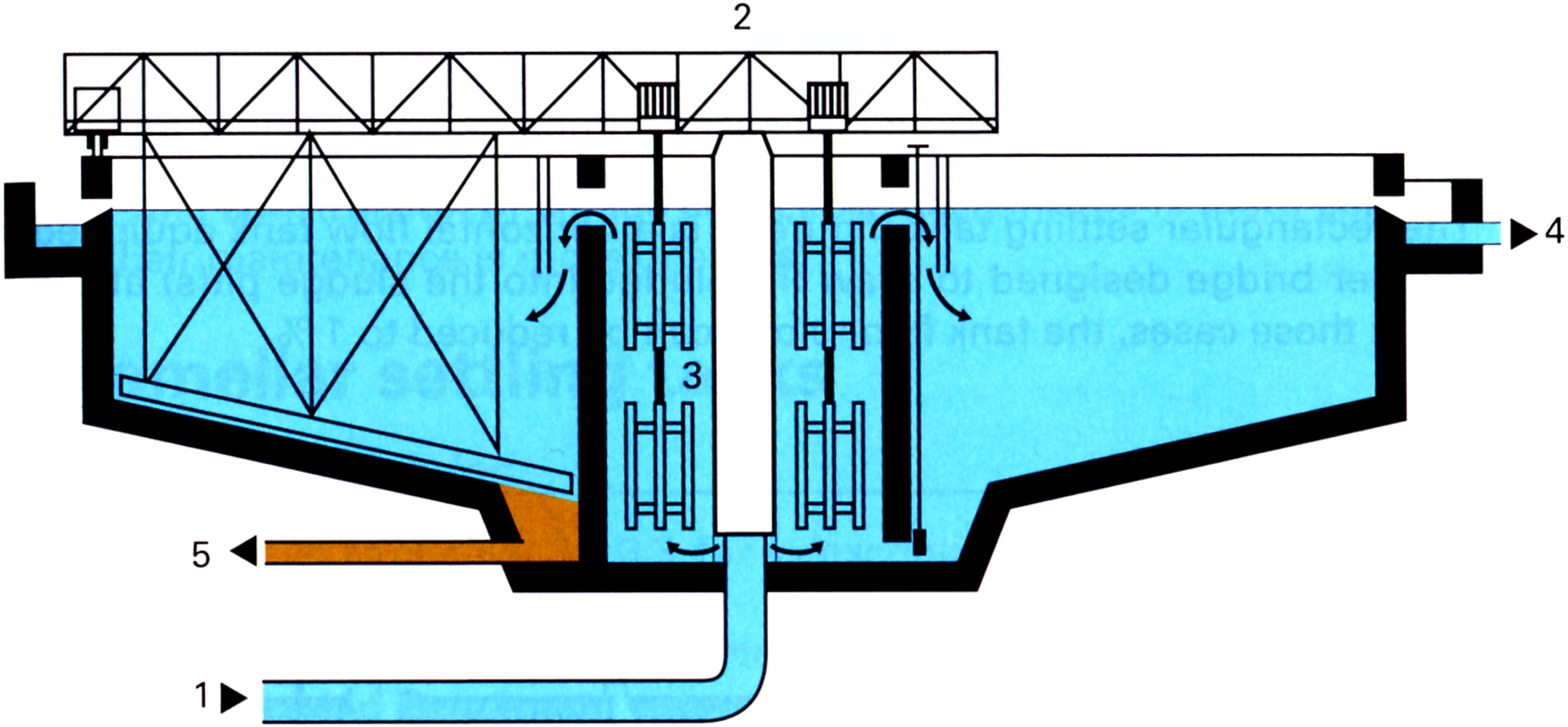
Precipitation of $Al(OH)_3(s)$ and $Fe(OH)_3(s)$



Stumm and Morgan, 1996

Settling velocity





1. Raw water inlet
2. Scraper bridge
3. Flocculation zone
4. Clarified water outlet
5. Sludge removal

Figure 7. ***Static settling tank with integrated flocculation area***

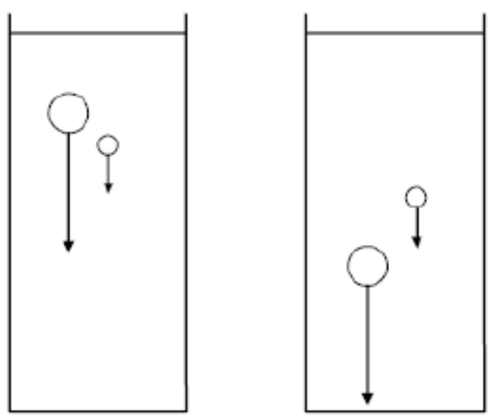
Water Treatment Handbook, Degremont-Suez, 2007, vol. 2

Settling behaviour of a continuous distribution of particles

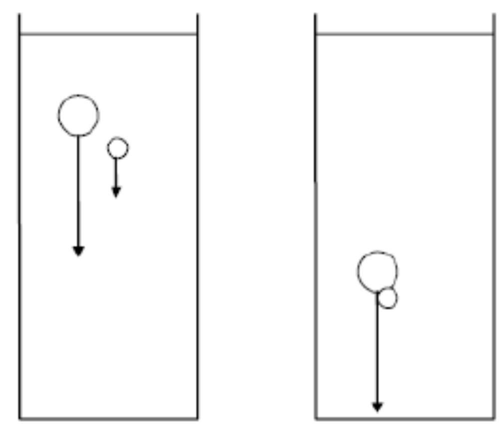
For low particle concentrations, distinguish between two types of sedimentation:

Type I sedimentation (discrete particle settling): particles do not aggregate as they settle

Type II sedimentation (flocculant settling): settling with simultaneous flocculation

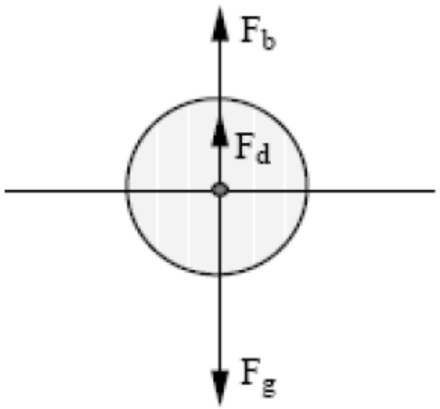


Type 1: removal of grit and sand



Type 2: Sedimentation after flocculation

Theory of discrete settling



Gravity force: $$F_G = V_p g \rho_p$$

Buoyancy force: $$F_B = V_p g \rho_W$$

Drag force: $$F_D = C_D A_p \rho_W \frac{v^2}{2}$$

At v = v_s (= terminal settling velocity, usually reached within 1s):

$F_G = F_B + F_D$

$$V_p g(\rho_p - \rho_W) = C_D A_P \rho_W \frac{v_s^2}{2}$$

$$v_s = \sqrt{\frac{(\rho_P - \rho_W)}{\rho_W} \cdot \frac{g}{C_D} \cdot \frac{2V_P}{A_P}}$$

V_p = particle volume (m^3)
ρ_p = particle density (kg/m^3)
ρ_W = density of water (kg/m^3)
C_D = drag coefficient
A_p = crossectional particle area (m^2)
v = particle settling velocity (m/s)
v_s = terminal particle settling velocity (m/s)

The terminal settling velocity v_s is a key parameter. It determines how fast particles will settle, and thus how much residence time the sedimentation tank requires.

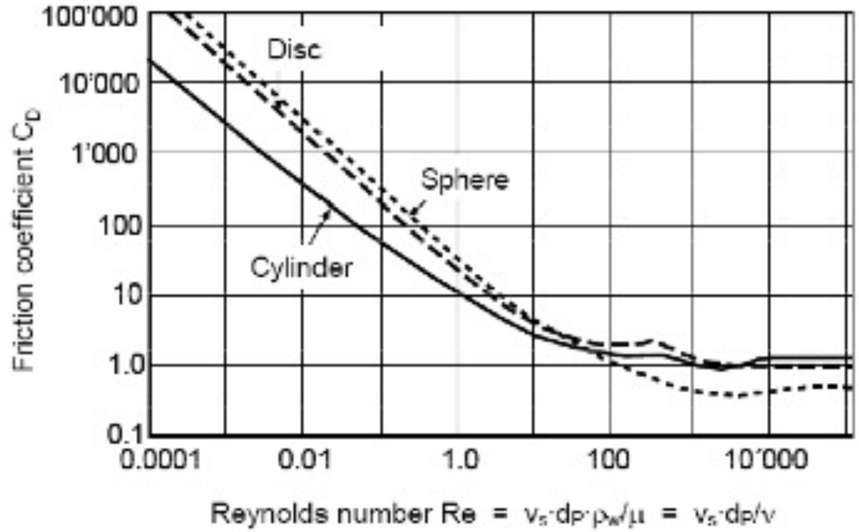
Calculation of the drag coefficient C_D

C_D depends on Re: $$Re = \frac{d_p v_s \rho_W}{\mu_W}$$

Re < 1 (laminar flow) $$C_D \approx \frac{24}{Re} = \frac{24 \mu_W}{d_p v_s \rho_W}$$

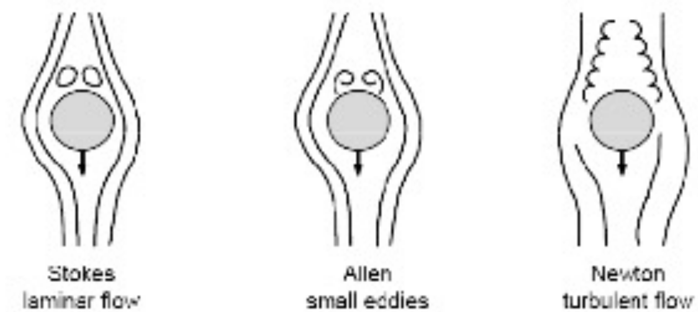
1 < Re < 2000 (transient range) $$C_D \approx \frac{24}{Re} + \frac{3}{\sqrt{Re}} + 0.34$$

Re > 2000 (turbulence) $$C_D \approx 0.4$$



Temperature °C	Dynamic viscosity μ kg m^{-1} s^{-1}
0	0.0017938
5	0.0015518
10	0.0013097
15	0.0011592
20	0.0010087
25	0.0009046
30	0.0008004
35	0.0007270
40	0.0006536

T-dependence



μ_W = dynamic viscosity of water (kg/(m*s))
d_P =particle diameter

Discrete settling at Re < 1 (laminar)

Consider:

- For most particles v_s is small enough that Re < 1, so $C_D \approx \frac{24}{Re} = \frac{24\,\mu_W}{d_p v_s \rho_W}$
- Spherical particles, so V = $\frac{\pi}{6} d_p^3$, and A_P= $\frac{\pi}{4} d_p^2$

Recall: $F_G=F_B+F_D$ (or $F_G-F_B=F_D$)

$$V_p g(\rho_p - \rho_W) = C_D A_P \rho_W \frac{v_s^2}{2}$$

Substitute V_P, A_P and C_D:

$$\frac{\pi}{6} d_P^3 g(\rho_p - \rho_W) = 3\pi\mu_W d_p v_s$$

$$v_s = \frac{d_p^2 g(\rho_p - \rho_W)}{18\mu_W}$$ **(Stoke's Law)**

For a given calculation of v_s it has to be tested if Re < 1

$$\mathrm{Re} = \frac{d_p v_s \rho_W}{\mu_W}$$

Discrete settling at Re > 1

For particles with Re > 1, the Stoke's equation is no longer valid.

In this case, the final settling velocity is given by Newton's law:

$$v_s = \sqrt{\frac{4g(\rho_p - \rho_w)d_p}{3C_D\rho_w}}$$

In this case an iterative approach has to be taken:

1. Calculate vs by Stoke's law
2. Calculate the Reynolds number from v_s. If Re > 1, calculate C_D.
3. Based on C_D, v_s can be recalculated by Newton's law and the procedure continues until v_s converges.

Critical settling velocity or overflow rate

A sedimentation tank needs to be designed to remove a desired fraction of particles before the water leaves the tank. It has to be considered that there is a certain velocity distribution for settling of particles.

The critical settling velocity v_c of a particle is given by: $v_c = \frac{h_o}{\tau}$

v_c: particle settling velocity such that the particle at the surface of the inlet is removed in sludge zone just before the outlet

h_o: depth of sedimentation basin

τ: hydraulic detention time of sedimentation basin

v_c can also be expressed as follows:

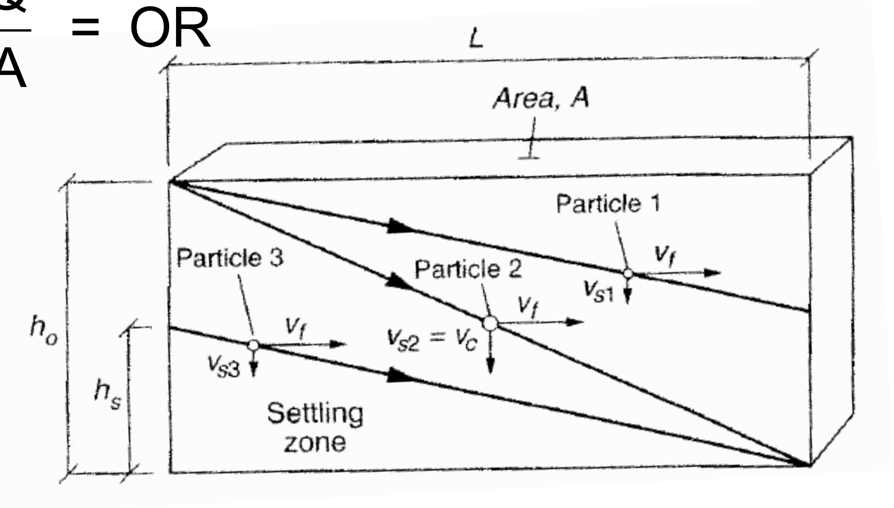
$$v_c = \frac{h_o}{\tau};\ \tau = \frac{V}{Q} = \frac{h_o \times A}{Q} \Rightarrow v_c = \frac{h_o \times Q}{h_o \times A} = \frac{Q}{A} = OR$$

OR: overflow rate $m^3/m^2/h$

V: Volume of tank

A: Area of top of basin settling zone m^2

Q: process flow rate m^3/h

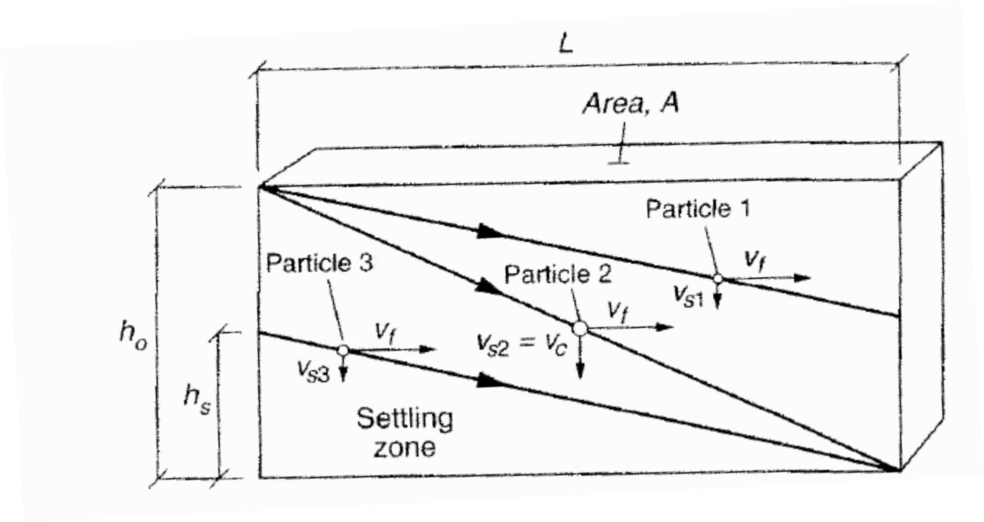


Removal efficiency

Particles with a settling velocity < v_c may also be removed, if they enter the sedimentation tank at a height $h_s < h_o$

The fraction f of the particles removed can be expressed by:

$$f = \frac{h_s}{h_o} = \frac{h_s/\tau}{h_o/\tau} = \frac{v_s}{v_c}$$



v_s: particle settling velocity smaller than v_c, m/h

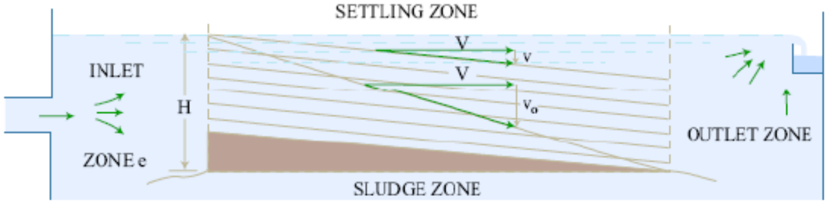
h_s: height of particle from the bottom of tank at position entering the settling zone, m

Design considerations for type 1 settling tanks

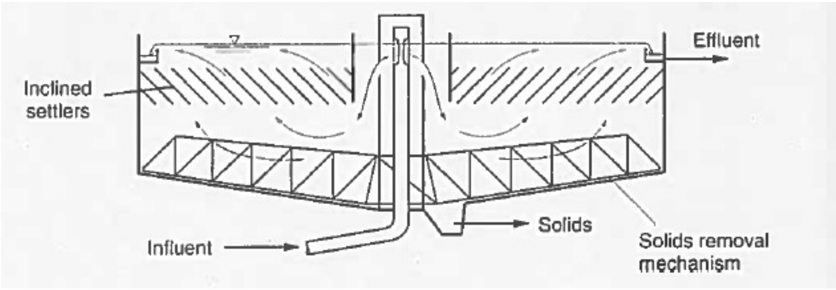
Design variable that determines η: v_{so} (=Q/A, H/τ).

Depth H is irrelevant! If H doubles, so does τ, and the ratio stays the same.

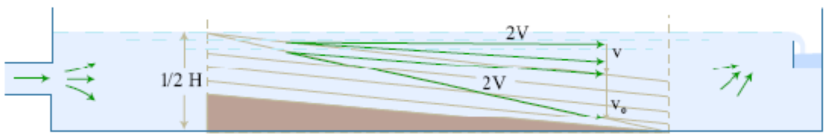
Q is usually fixed → surface area A is the only variable



Zones of a rectangular, horizontal, continuous-flow sedimentation basin.



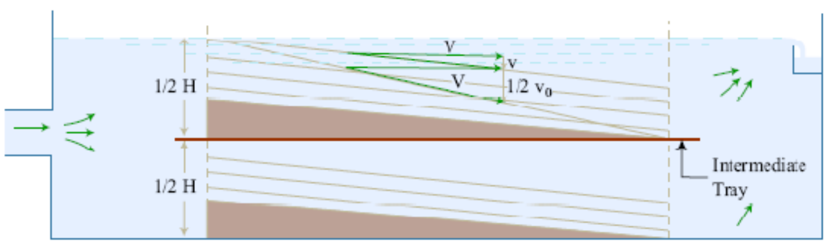
Upflow separation



Reduced tank depth does not increase removal ratio.

Shallow tanks: not a good approach!

- linear velocity of water in the system increases, causing resuspension of particles
- plug flow difficult to maintain

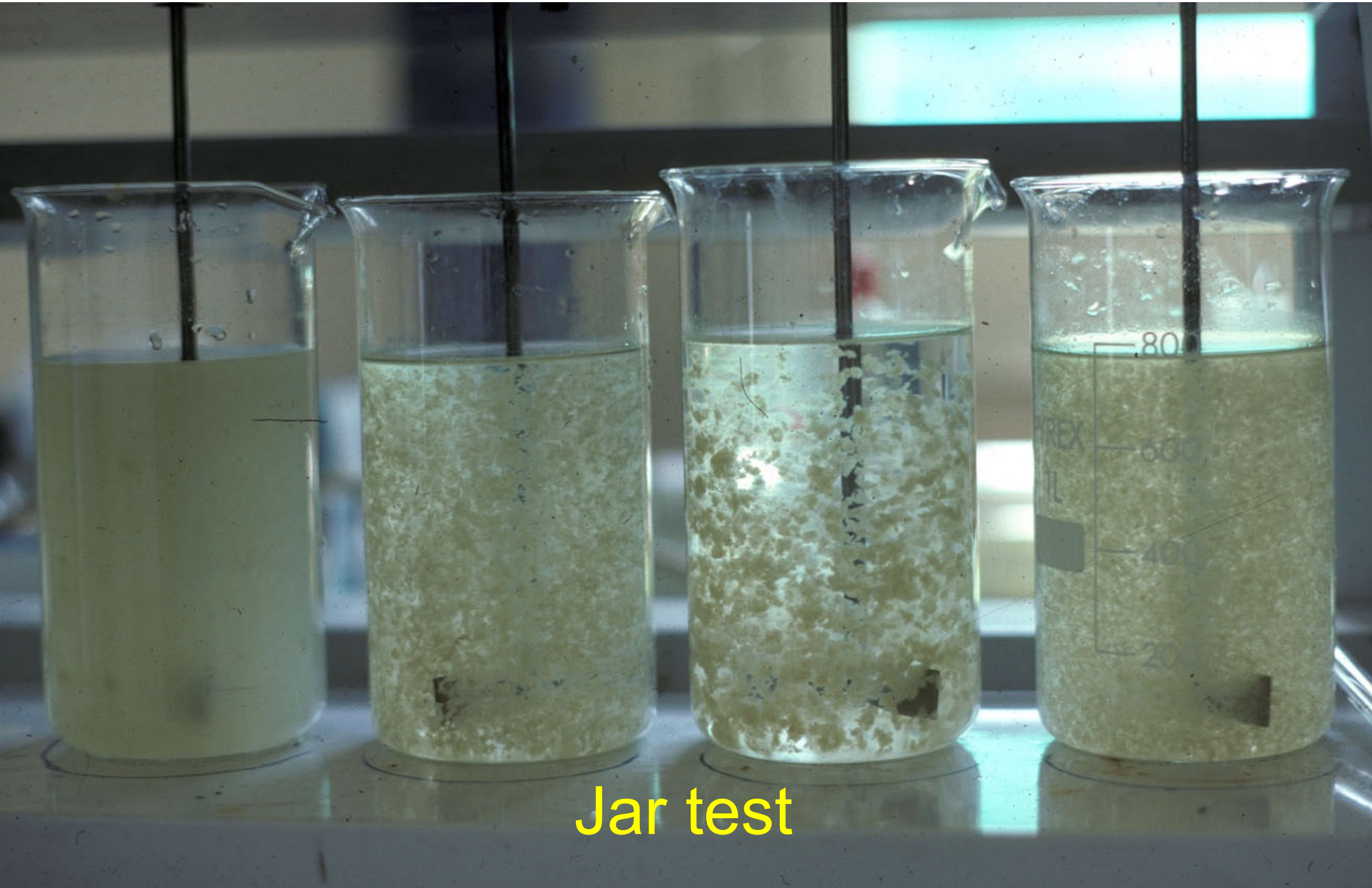


Tray in tank provides added floor area & increases solids removal

False bottom tank:

- false bottom collects particles at one-half the full depth
- Q and A (and τ) do not change
- v_H does not change.
- H that any particle must fall before it hits a surface and is removed is halved.
- v_{so} has been cut in half

Coagulation-Flocculation-Sedimentation



Jar test

Summary settling

Re < 1: Stoke's law

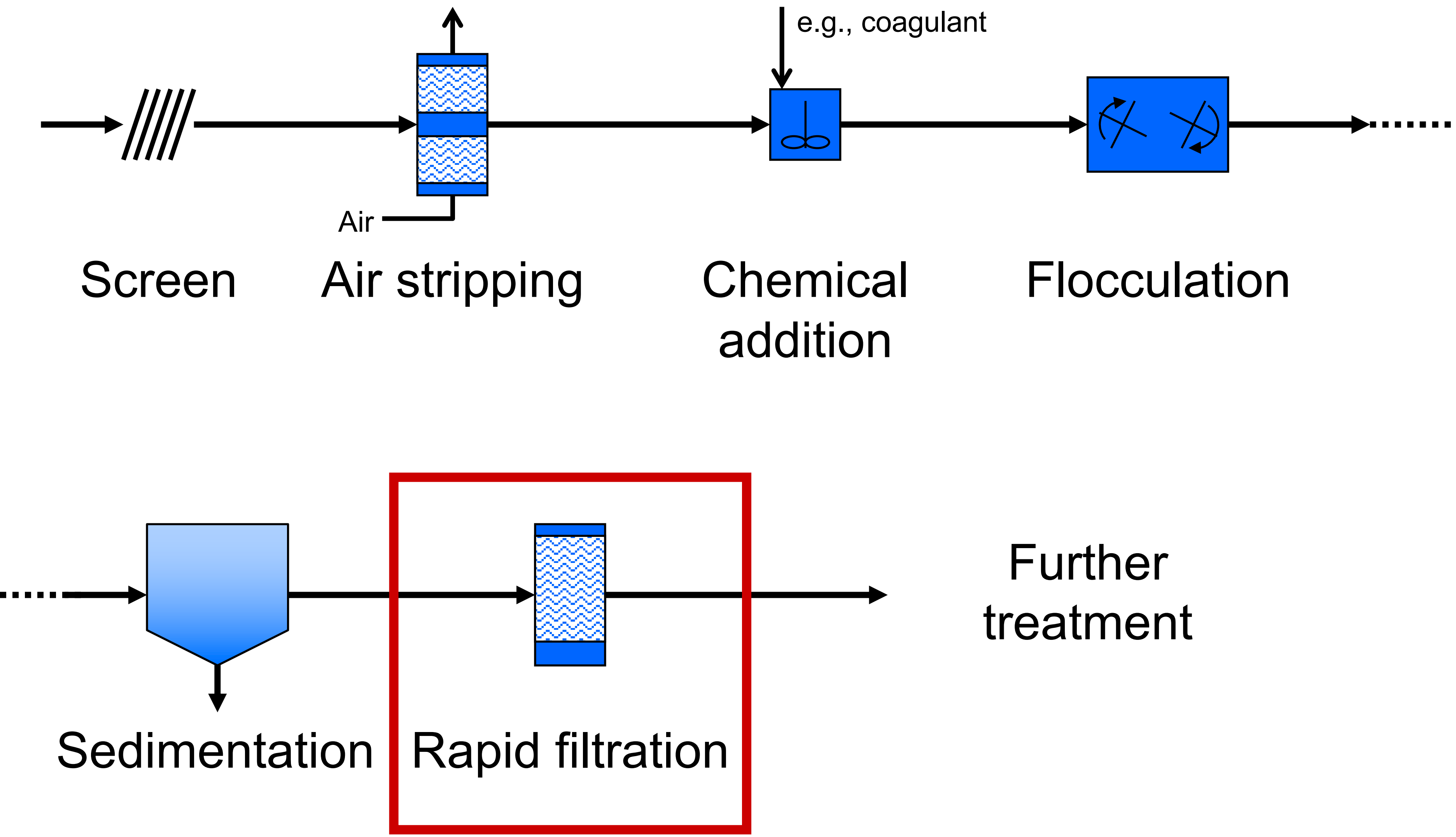
Re > 1: Newton's law

For calculated settling velocity (v_s), check which regime is valid

For settling tanks, the critical settling velocity v_c is crucial and defined by the ratio of Q/A (process flow rate/surface area of settling tank)

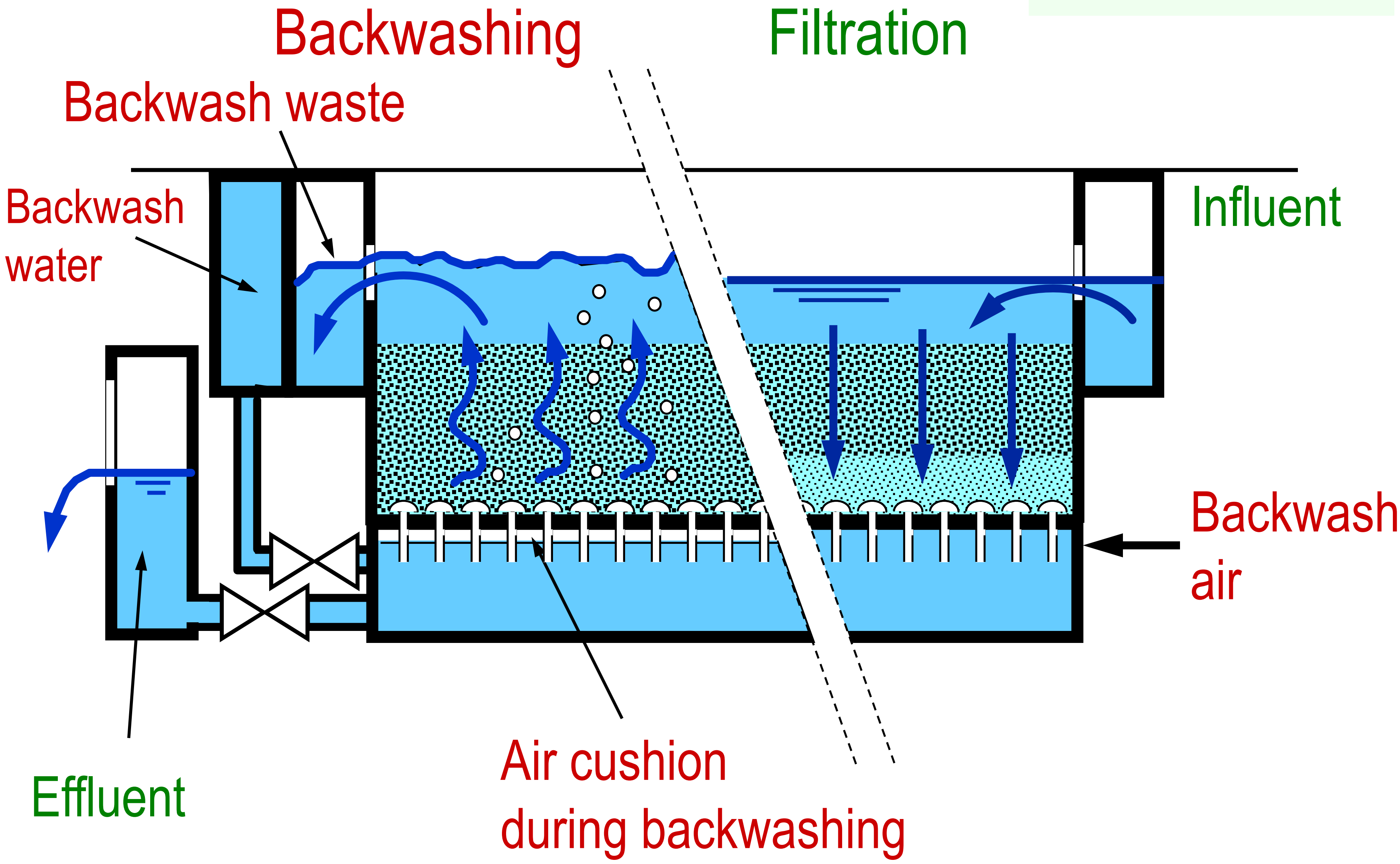
The fraction of particles which are removed is v_s/v_c

Filtration in surface water treatment

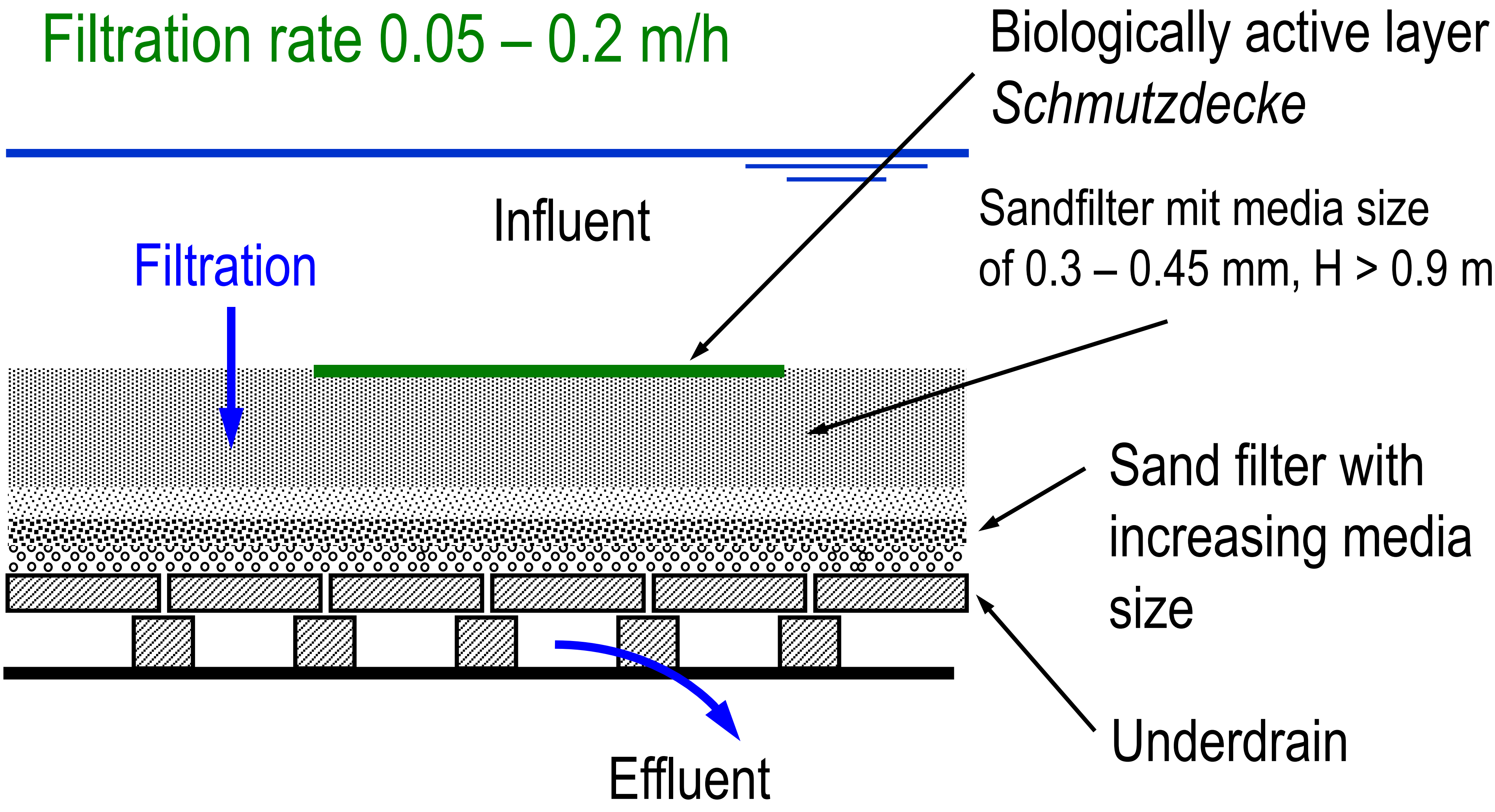


Rapid filtration

Filtration
Rate 5-10 m/h



Slow sand filter



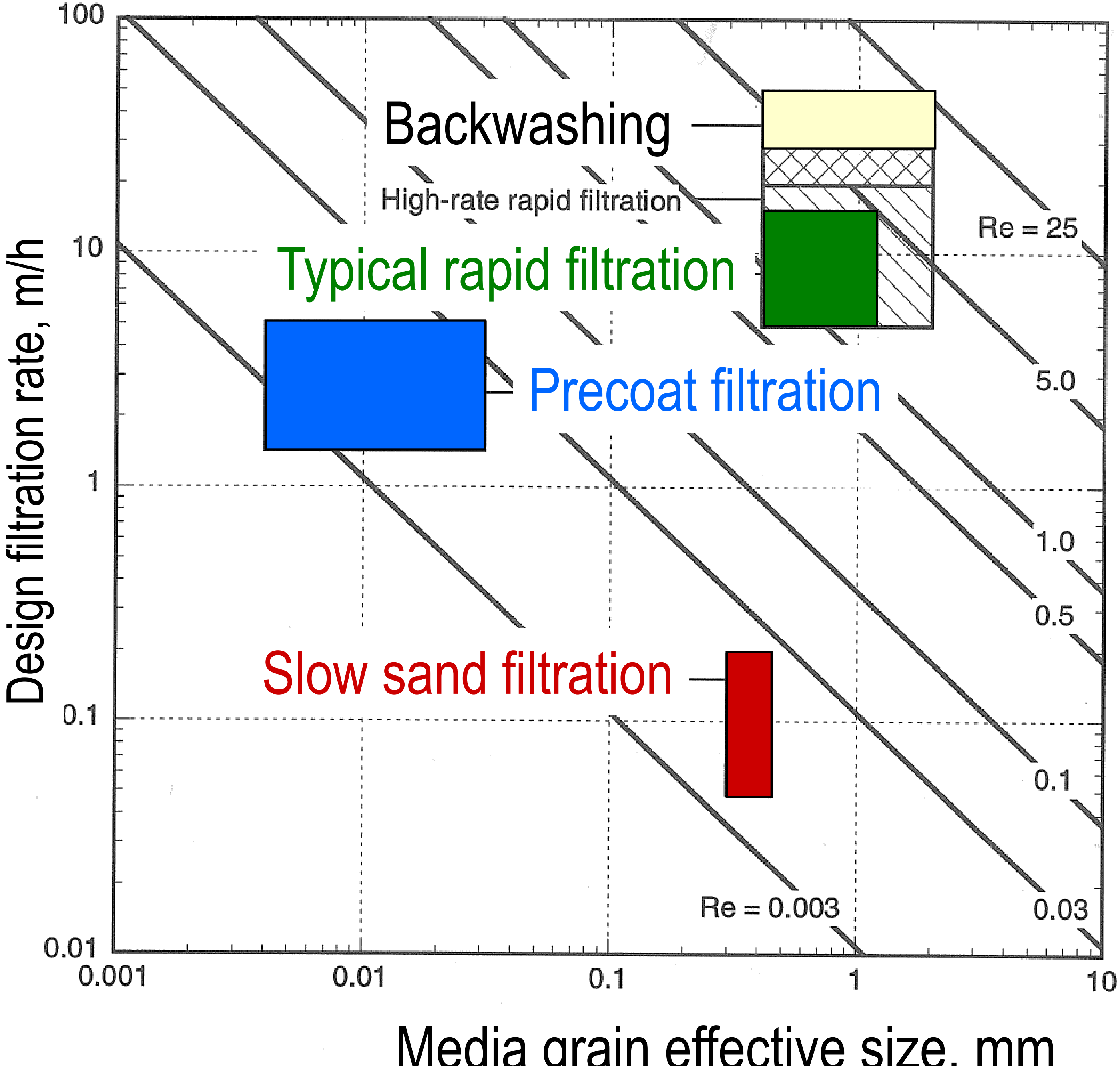


Figure 11-5
Ranges of Reynolds numbers for granular filtration technologies.

$$Re = \frac{\rho_W v d}{\mu}$$

ρ_W = Fluid density, kg/m^3

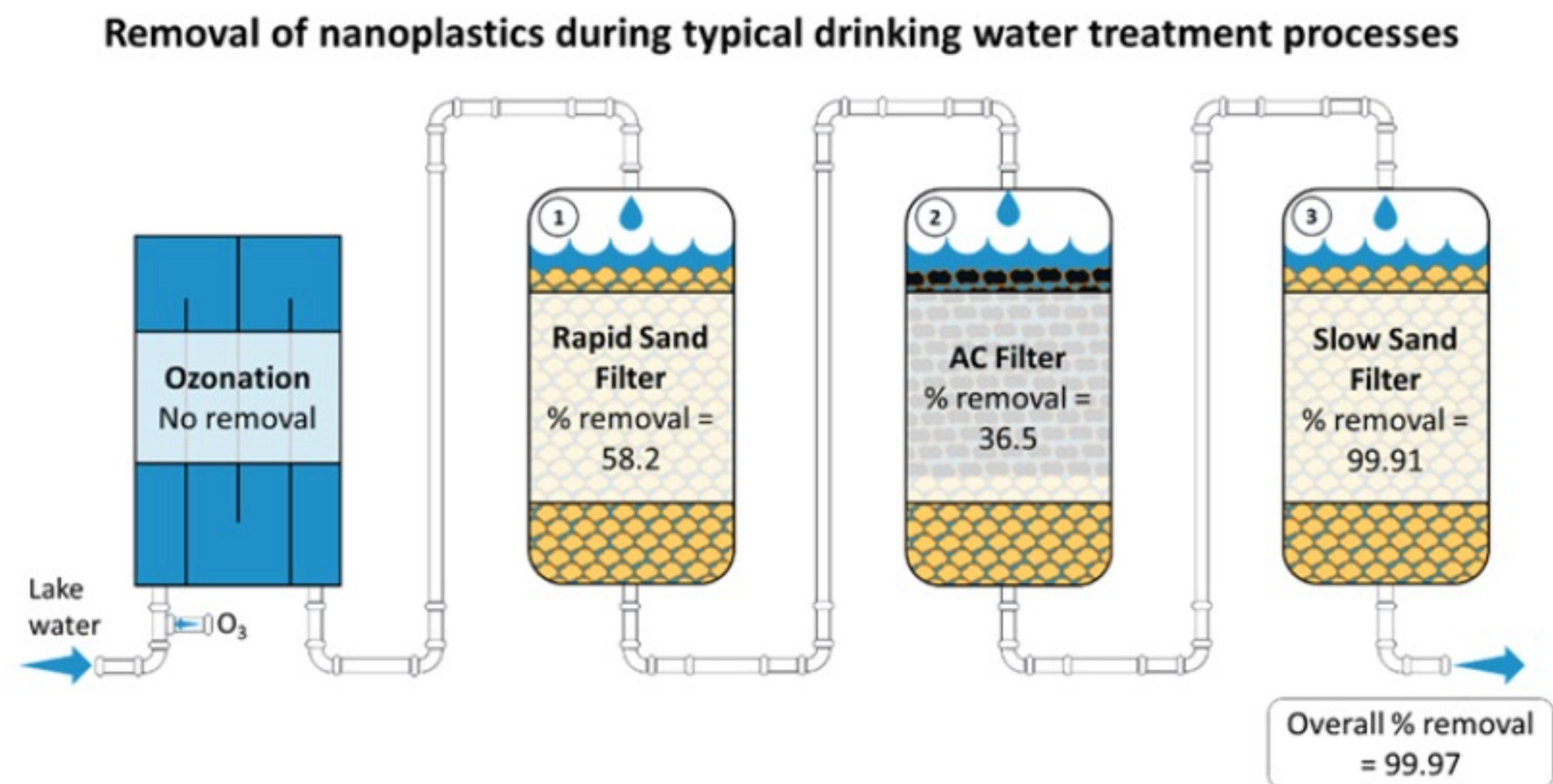
v = Filtration rate (superficial velocity), m/s

d = media grain diameter, m

μ = dynamic viscosity of fluid, kg/m.s

MWH book

Combination of different filters for plastic nano-particle removal (diameter: 160-220



Biofilm is very effective for nanoparticle removal

Pulido-Reyes, G. et al. (2022) Journal of Hazardous Materials ***436****: 129011.*

Typical filter run

Rapid filtration

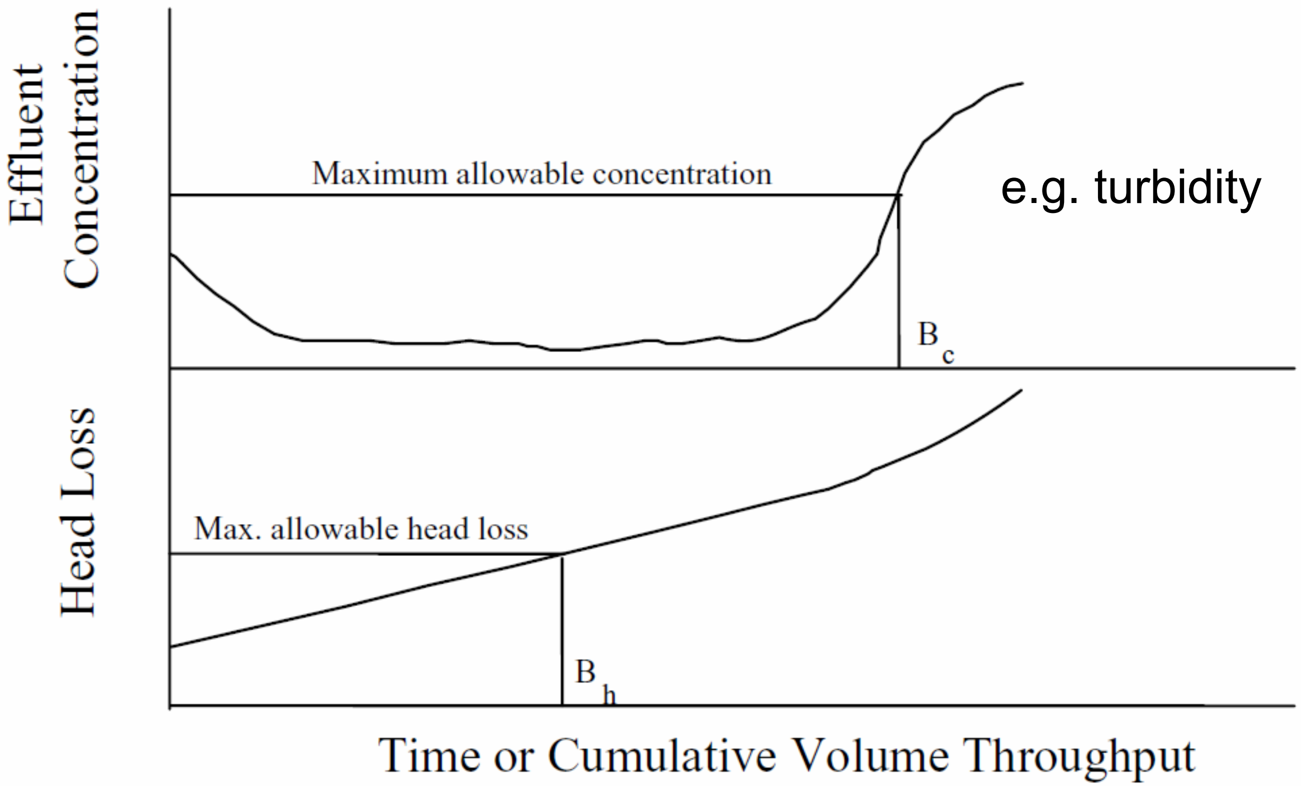
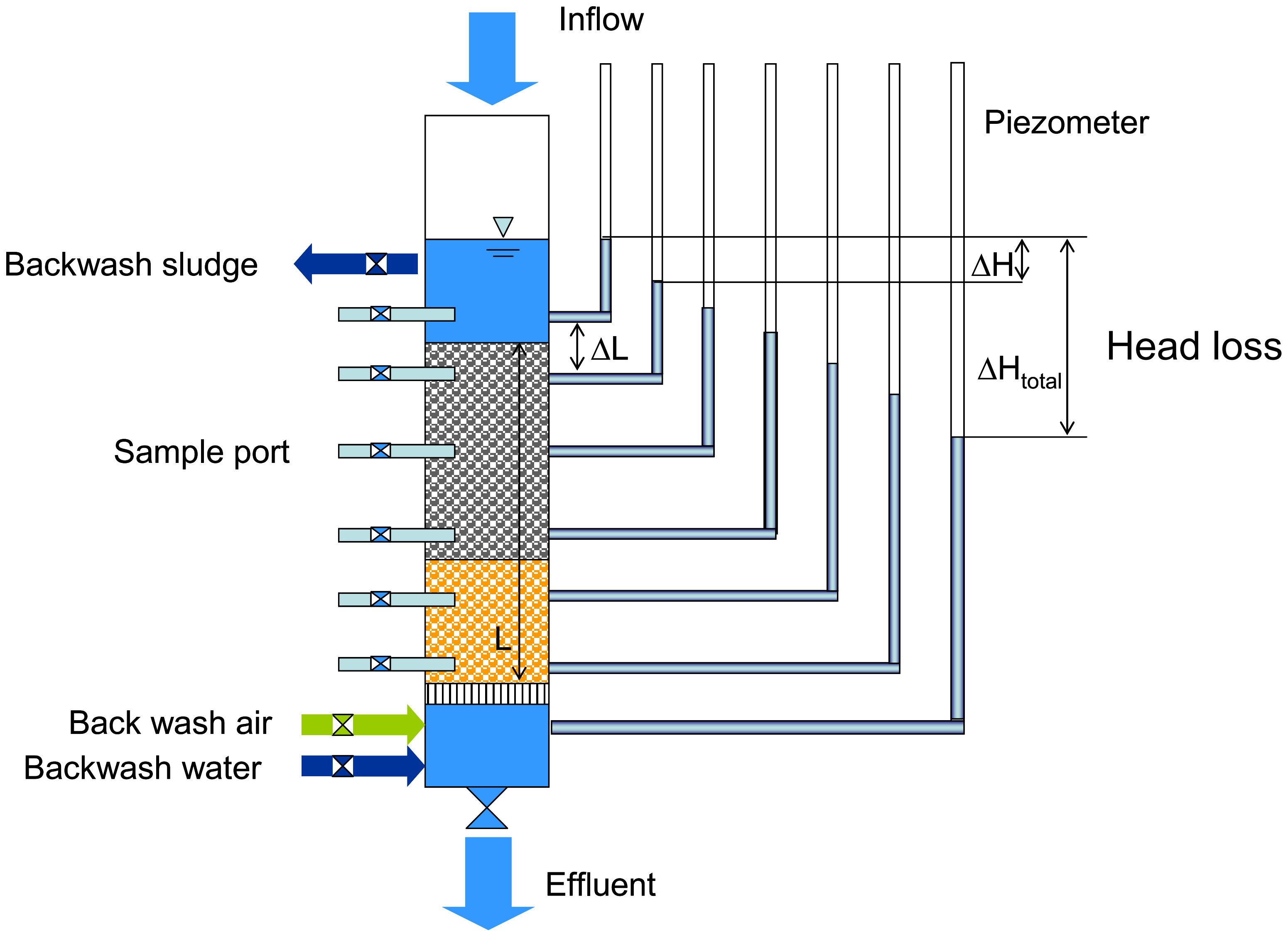


Figure 14-2. Typical Filter Performance in a Single Run. (The points B_c and B_h represent the times or throughput volumes at which backwashing is necessary because the maximum allowable effluent concentration or maximum allowable head loss is exceeded, respectively. A normal filter run would be terminated when the first of these points was reached, but the graph is extended here for illustrative purposes.)

Filtration Experiments - Head Loss



Head loss through clean granular filters

The development of an equation for calculation of head loss includes both theoretical considerations and empirical approaches (for more details see MWH book). Since the Reynolds number is typically above 1 in rapid filtration steps, the following Equation has been developed, based on work from Forchheimer:

$$\text{head loss } h_L = \kappa_V \frac{(1-\varepsilon)^2}{\varepsilon^3} \frac{\mu L v}{\rho_w g d^2} + \kappa_I \frac{1-\varepsilon}{\varepsilon^3} \frac{L v^2}{g d}$$

kinetic energy loss

viscous energy loss

κ_V: head loss coefficient due to viscous forces, unitless

κ_I: head loss coefficient due to inertial forces, unitless

ε: porosity, dimensionless

μ: dynamic viscosity of water, kg/ms

L: depth of granular media, m

v: filtration rate, m/s

ρ_w: density of water kg/m^3

g: gravity constant, 9.81 m/s^2

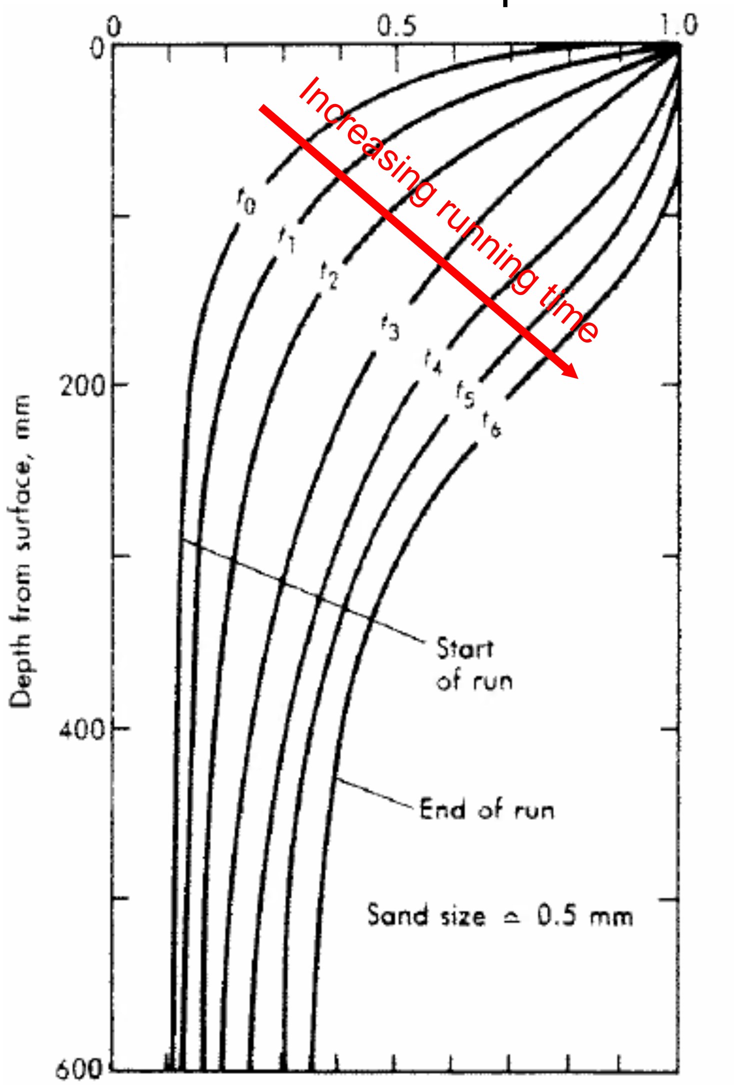
d: diameter of granular media

Recommended values for head loss equation

Medium	κ_V	κ_I	ε, %
Sand	110-115	2.0-2.5	40-43
Anthracite	210-245	3.5-5.3	47-52

Filter run time: accumulation of solids

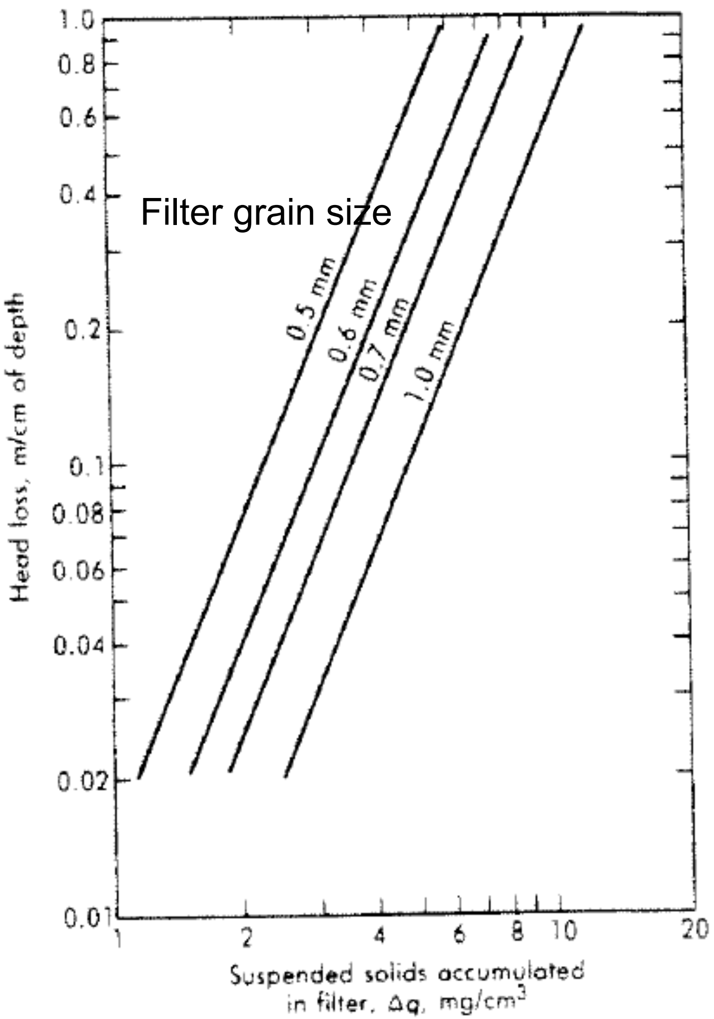
Relative concentration of particles c/c_o



Suspended solid removal curves as a function of depth and time
⇒ No homogeneous deposition of solids in filter bed

Water Quality, Characteristics, Modeling, Modification; Addison, 1987

Filter run time: Build-up of head loss



Head loss build-up as a function of the medium size and the amount of material removed

For each layer, the mass balance equation can be formulated as:

$$v\frac{\Delta C}{\Delta x} = \frac{\Delta q}{\Delta t}$$

v: filtration velocity, $m^3/m^2 \cdot min$

$\Delta C = (C_{x-1} - C_x)$, kg/m^3

$\Delta x = (x_{x-1} - x_x)$, m

$\Delta t = (t_2 - t_1)$, min

$\Delta q = (q_2 - q_1)$, kg/m^3 (SS accumulated)

Head loss at time t:

$$H_t = H_o + \sum_{i=1}^{n} h_i(t)$$

H_t: total head loss at time t

H_o: total initial clean water head loss

$(h_i)_t$: head loss in the *i* th layer of the filter at time t

Water Quality, Characteristics, Modeling, Modification; Addison, 1987

Conclusions

- Particle removal is still a very empirical science
- Assessment of particle size distribution allows to characterize separation processes
- Coagulation/flocculation processes are determined by an optimum range (pH, flocculant dose)
- Sedimentation rates can be used to design settling tanks: Laboratory jar tests
- Filtration is difficult to be described theoretically, many empirical approximations

➔ Piloting for particle removal processes!