Technische Universität
Bergakademie Freiberg
Institute of Geology
Department of Hydrogeology

Lecture notes for course

Hydrology III

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Foreword:

The course “Hydrology III” is a part of the module “Hydropedology”, which is ingredient of geoecology master degree course. Module “Hydropedology” is one of three obligatory modules within the area of specification called “River Basin Management”. Further more the module is a elective part of the master studies in „Climate and Environment“ and „Ecosystem and Landscape Management”.

The semester periods per week for the module “Hydropedology” in summary are: 3 for lectures, 5 for seminars and 3 for practical work. In total 12 credit points are acquirable. The semester periods per week for the hydrological part (“Hydrology III”) are: 1.5 for lectures, 3 for seminars and 3 for practical work. In connection with practical work the participants have to realize an individual project where field investigations, laboratory work and model application are included.

The course “Hydrology III” requires basic hydrologic knowledge from course “Hydrology I” which is held in bachelor degree course. Further more some practical aspects from module “Hydrology II” are needed.

The main contents of the course “Hydrology III” are the following:

- hydraulics, sediment transport and ice formation in rivers
- flood formation, flood calculation and flood protection
- low water formation and calculation
- hydrological design of reservoirs (dams and flood retention reservoirs)

The lecture notes is a guideline where the main topics, tables and figures are summarized. The lecture notes are an internal study material. Therefore you will not find any references except of pictures sources.

The lecture notes contain nearly the same literature references like in the scripts “Hydrology I” and “Hydrology II”. At the end of each chapter a list of special literature is given. In addition research results of the Freiberg Department of Hydrogeology are used.

I hope and wish that the lecture notes contribute to a better understanding of the communicated didactic topics.

I am grateful for any suggestions which are act the improvement of the lecture notes.

Freiberg, February 2014

Volkmar Dunger

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Appendix 1: Seminary tasks for “Hydrology III” course

Appendix 2: Instruction for project work

1. River hydraulics

1.1. Introduction, types of water movements

* **Aims:**

- Description of water movement by means of mathematic equations (for example BERNOULLI´s equation)
- River hydraulics = essential requirement for the description of other hydrologic processes → floods, sediment transport, ice formation

* **Types of water movement:**

i) Free, unregulated movement → falls (for example in reach of weirs and water falls)

ii) Within channels regulated movement → laminar (in surface waters practically never occurring) and turbulent flow (streaming and shooting flow)

► ***Streaming and shooting flow:***

- If flow velocity $v_{FLOW} \leq$ speed of a gravity wave v_{WAVE} → streaming (subcritical flow)
- If $v_{FLOW} > v_{WAVE}$ → shooting flow (supercritical flow)
- Speed of a gravity shallow water wave with a small amplitude is calculable:

$$v_{WAVE} = (g * h)^{0.5} \qquad (1.1)$$

where: v_{WAVE} – speed of a gravity wave [m/s]
g – gravitation constant [m/s^2]
h – water depth [m]

- Subcritical/supercritical flow calculable by FROUDE number Fr:

$$Fr = v_{FLOW} / v_{WAVE} \qquad (1.2)$$

Where: v_{FLOW} – flow velocity [m/s]
(all other symbols → see equation 1.1)

→ Fr > 1.0 → supercritical flow, Fr ≤ 1.0 → subcritical flow

- Estimation of v_{FLOW} by calculations or measurements → see lecture notes “Hydrology I”, **practical application of different measurement methods → see seminars 1 to 3**)
- Approximately calculation of FROUDE number Fr:

$$Fr = \frac{\alpha * v_m}{\left[\dfrac{g * A}{b_{Sp}}\right]^{0.5}} \qquad (1.3)$$

Where: α – flow velocity coefficient [ ] → standard values see table 1.1
v_m – average flow velocity [m/s]
g – gravitation constant [m/s^2]
A – cross-sectional area [m^2]
b_{Sp} – width of water level [m]

Table 1.1: Standard values for flow velocity coefficient α

Kind of stream	Flow velocity coefficient α [ ]
Canal	1.1 – 1.3 (mean: 1.2)
Natural river ground (stream, river)	1.2 – 1.6 (mean: 1.4)
Bursting of a rivers bank	1.5 – 2.0 (mean: 1.8)

- Examples for different flow conditions (longitudinal section) → see figure 1.1

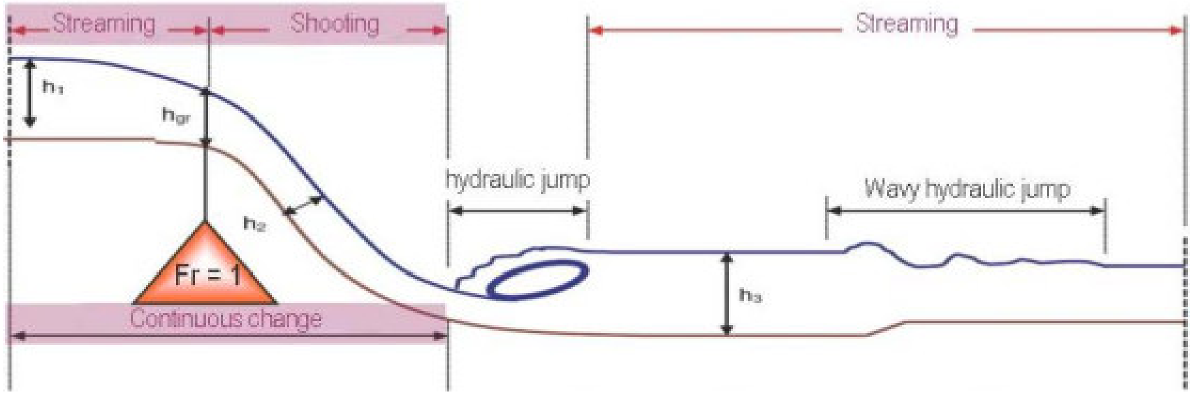


Fig. 1.1: Examples for different flow conditions in longitudinal section (after LfU, 2002)

- Types of hydraulic jumps in dependence on FROUDE number → see figure 1.2

Fr = 1.0 … 1.7 → wavy hydraulic jump	
Fr = 1.7 … 2.5 → low energy turnover	
Fr = 2.5 … 4.5 → oscillating hydraulic jump, heavily waves	
Fr = 4.5 … 9.0 → stationary hydraulic jump	
Fr > 9.0 → heavy turbulent hydraulic jump	

Fig. 1.2: Types of hydraulic jumps (LfU, 2002)

- Knowledge of hydraulic jump types, FROUDE numbers and flow conditions important for the assessment of building stress (energy release)
- **practical application → see seminar 6**

► ***Steady and non-steady flow:***

- Flow classifiable with regards to temporal and spatial change of flow velocity v and discharge Q along longitudinal section
- Overview → see figure 1.3

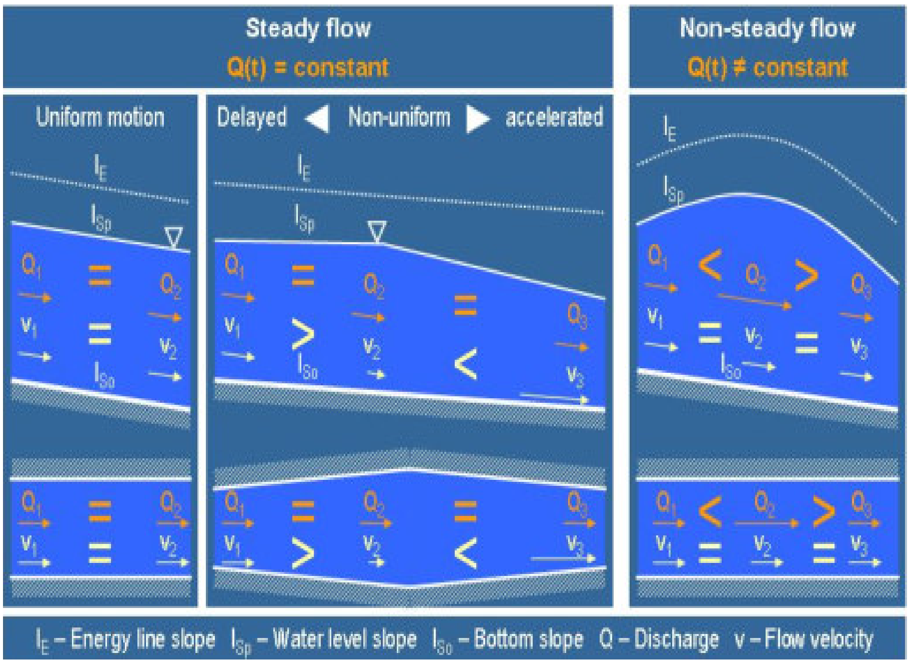


Fig. 1.3:

Steady and non-steady flow (after LfU, 2002)

- Conclusions in analysis of figure 1.3:
  → Strictly speaking all flow processes in flowing waters are non-steady.
  → In the most cases it is approximately possible to emanate from steady conditions. → calculation more simple, exceptions: mountain streams especially during flood periods and reservoir influenced streams and rivers in the case of heavy varying water releases

1.2. Methodology of hydraulic calculations for flowing waters

- basis: basic laws of physics:
  → mass conservation law (continuity equation)
  → energy conservation law (energy equation)
- continuity equation:

  For steady conditions is valid:

$$Q = v_1 * A_1 = v_2 * A_2 = \ldots = v_i * A_i \qquad (1.4)$$

where: Q – discharge [m^3/s]
v_i – mean flow velocity on cross-section A_i [m/s]
A_i – flow profile [m^2]

→ for illustration → see figure 1.4

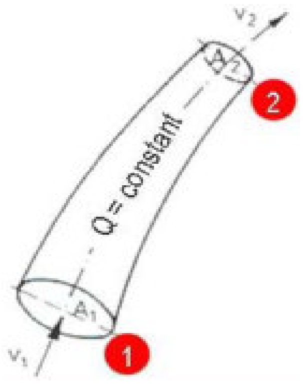


Fig. 1.4:

Principle of continuity (LfU, 2002)

- energy equation:

  For a frictionless steady system h_E can be expressed as:

$$h_E = k_K + h_D + z \tag{1.5}$$

where: h_E – energy potential [m]
h_K – kinetic potential [m]
h_D – pressure potential [m]
z – geodetic potential [m]

→ frictionless system → theory

→ more practical case: decrease of energy potential because of frictional losses in direction of the current (for example from point 1 to point 2 → for illustration → see figure 1.5)

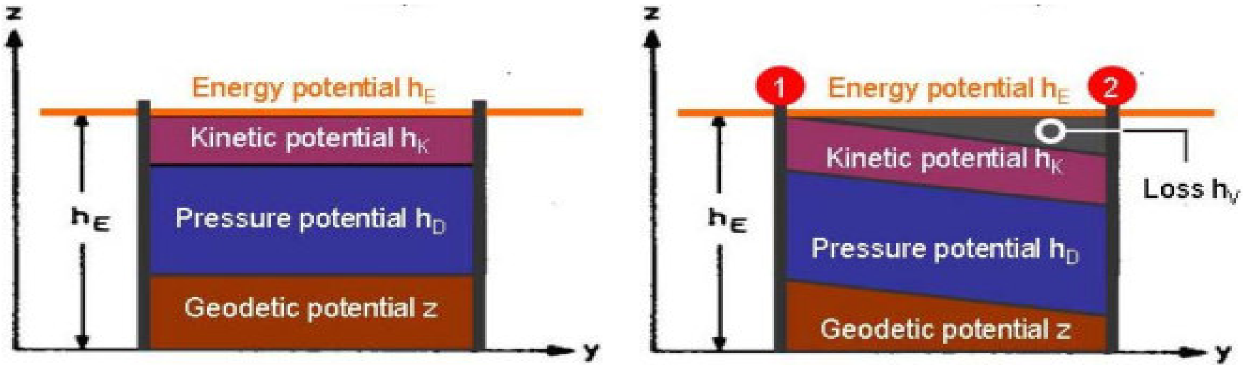


Fig. 1.5: Illustration of energy conservation equation (theoretical and practical case)

→ BERNOULLI equation for steady flow by considering of frictional losses:

$$h_{E,1} = h_{E,2} + h_V \tag{1.6}$$

$$z_1 + h_{D,1} + h_{k,1} = z_2 + h_{D,2} + h_{k,2} + h_V \tag{1.7}$$

where: h_E – energy potential [m]
h_V – loss potential [m]
z – geodetic potential [m]
h_D – pressure potential [m]
h_K – kinetic potential [m]

where: $$h_D = \frac{p}{\varrho g} \tag{1.8}$$

and: $$h_K = \frac{v^2}{2g} \tag{1.9}$$

with: p – pressure [Pa]
ϱ – water density [kg/m^3]
g – gravitation constant [m/s^2]
v – flow velocity [m/s]

→ causes for frictional losses:
- inner friction of water particles
- river ground and riparian friction
- bending of river pathway
- cross-section changes
- vegetation

1.3. Hydraulic calculation basis for flowing waters

1.3.1. General stream formula for steady state uniform flow

- definition of a steady state uniform flow → see chapter 1.1
- consideration of a flow profile with uniform cross-section and constant slope → see figure 1.6

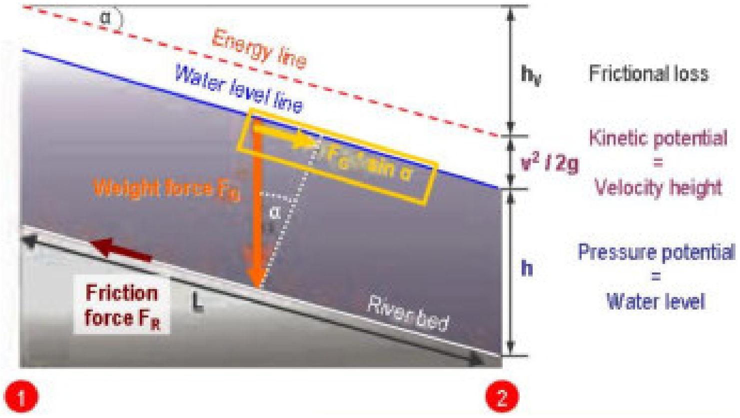


Fig. 1.6:

Potentials and forces in case of a steady state flow (after LfU, 2002)

- flow process caused by downhill component of weight force F_G → $F_G * \sin α$
- F_G results from water volume V between points 1 and 2:

$$F_G * \sin α = V * \varrho * g * \sin α = A * L * \varrho * g * \sin α \tag{1.10}$$

where: F_G – weight force [N]
V – water volume between points 1 and 2 [m^3]
A – cross-section area [m^2]
L – flow length [m]
g – gravitation constant [m/s^2]
ϱ – water density [kg/m^3]
α – river ground slope

- for small energy and river ground slopes regards approximately:

$$\sin \alpha \approx h_V / L \tag{1.11}$$

(all symbols → see equation 1.10)

- flow process causes friction force F_R an at river ground and embankment, which is react downhill force ($F_G * \sin \alpha$)
- F_R = arithmetic product from shear stress τ and wetted channel area between points 1 and 2:

$$F_R = \tau * l_U * L \tag{1.12}$$

where: F_R – friction force [N]
τ – shear stress [N/m^2]
l_U – wetted perimeter [m]
L – flow length [m]

- shear stress τ in pipe hydraulics analogy calculable:

$$\tau = \frac{\lambda}{8} \varrho * v^2 \tag{1.13}$$

where: λ – pipe coefficient [ ]
(all other symbols → see equations 1.9 and 1.12)

- connection of equations 1.12 and 1.13:

$$F_R = \frac{\lambda}{8} \varrho * v^2 * l_U * L \tag{1.14}$$

(all symbols → see equations 1.12 and 1.13)

- weight force F_G and friction force F_R hold the balance (energy conservation law) → connection of the equations 1.10. 1.11 and 1.14:

$$A * g * h_V = \frac{\lambda}{8} v^2 * l_U * L \tag{1.15}$$

(all symbols → see equations 1.10. 1.11 and 1.14)

- solving the average flow velocity v:

$$v = \left[\frac{8\,g}{\lambda}\right]^{0.5} \left[\frac{A}{l_U}\right]^{0.5} \left[\frac{h_V}{L}\right]^{0.5} \tag{1.16}$$

(all symbols → see equations 1.10. 1.11 and 1.14)

$\left[\frac{8\,g}{\lambda}\right]^{0.5}$ → flow velocity coefficient C $[m^{1/2}/s]$ → equal to roughness coefficient

$\left[\frac{A}{l_U}\right]$ → hydraulic radius r_{hy} [m]

$\left[\frac{h_V}{L}\right]$ → energy line slope I_E [m/m] → approximately = ground slope I_{So} [m/m]

→ change of equation 1.16:

$$v = C * \sqrt{r_{hy} * I_{So}} \tag{1.17}$$

where: v – flow velocity [m/s]
C – flow velocity coefficient $[m^{1/2}/s]$
r_{hy} – hydraulic radius [m]
I_{So} – river slope [m/m]

→ at about the same time mathematically derived by BRAHMS und DE CHEZY respectively DARCY and WEISBACH for pipe flows

- disadvantages of universal flow formula (after BRAHMS, DE CHEZY, DARCY, WEISBACH):
  - → C originally estimated for different rough pipes
  - → v = average flow velocity → influence of open channel geometry on velocity distribution by the hydraulic radius r_{hy} not satisfactory recorded

- transformation of the universal flow formulae for pipes to open channels → see chapter 1.3.2

1.3.2. Practicable calculation methods for open channels

* **overview:**

- at present used calculation formulae for open channel flow:
  - → empirical flow formula by GAUCKLER-MANNING-STRICKLER
  - → physically determined equation by COLEBROOK-WHITE

- application of GAUCKLER-MANNING-STRICKLER´s empirical formula in all cases where:
  - → ordinary accuracy is acceptable
  - → forelands are insignificant or have a little importance only or parametrisation is difficult
  - → parameter condition is not good

- application of physically determined equation by COLEBROOK-WHITE in all cases where:
  - → high accuracy is necessary
  - → forelands are important
  - → availability of information which allows a resilient parameter estimation

- **empirical formula by GAUCKLER-MANNING-STRICKLER (GMS formula):**
  - description of the flow velocity coefficient C by empirical roughness coefficients (in German-speaking countries STRICKLER coefficient k_{St} respectively in English-speaking countries MANNING coefficient n) and hydraulic radius r_{hy}:

$$C = k_{St} * r_{hy}^{1/6} = \frac{1}{n} * r_{hy}^{1/6} \qquad (1.18)$$

where: C – flow velocity coefficient [$m^{1/2}/s$]
r_{hy} – hydraulic radius [m]
k_{St} – STRICKLER coefficient [$m^{1/3}/s$]
n – MANNING coefficient [$s/m^{1/3}$]

→ modification of equation 1.16:

$$v = k_{St} * (r_{hy})^{2/3} * (I_{So})^{1/2} \qquad (1.19)$$

where: v – mean flow velocity [m/s]
r_{hy} – hydraulic radius [m]
k_{St} – STRICKLER coefficient [$m^{1/3}/s$]
I_{So} – river ground slope [m/m]

- parameter of GMS formula (details → see chapter 1.4):
  - → river slope I_{So}: from longitudinal channel analysis respectively topographic maps
  - → hydraulic radius r_{hy}: estimation by means of quotient A / l_U (cross-section area / wetted perimeter), requires the knowledge of cross-sections (levelling, sound the depth, ultrasonic measurement, …)
  - → STRICKLER coefficient k_{St}: characterises the hydraulic roughness of the stream (0 → rough, 100 → smooth), k_{St} is independent on channel greatness but dependent on water level
- essential difference of GMS formula in comparison to equation by COLEBROOK-WHITE:
  - → roughness coefficients (STRICKLER coefficient k_{St} respectively MANNING coefficient n) are estimated by empirical methods
  - → large number of investigations at very different open natural and artificial channels concerning river ground, riparian design, vegetation, … necessary

- **physically determined equation by COLEBROOK-WHITE:**
  - formula:

$$v = \frac{1}{(\lambda)^{1/2}} * (8\,g)^{1/2} * (r_{hy} * I_{So})^{1/2} \qquad (1.20)$$

where: v – mean flow velocity [m/s]
λ – hydraulic resistance coefficient [ ]
r_{hy} – hydraulic radius [m]
g – gravitation constant [m/s^2]
I_{So} – river ground slope [m/m]

- parameter r_{hy} and I_{So} analogue GMS formula
- hydraulic resistance coefficient λ depends on hydraulic radius r_{hy}, on cross sectional figuration, meander degree (straight, meandering) and on equivalent sand roughness k_S
- definition of equivalent sand roughness k_S [mm]:
  - → aim: comparison of hydraulic roughnesses of different pipes (about 1930)
  - → preparation of pipes with different, but precise defined roughnesses by coating with sand → grain size = roughness characteristics → see figure 1.7
  - → empirical estimation of flow behaviour for different grain diameter

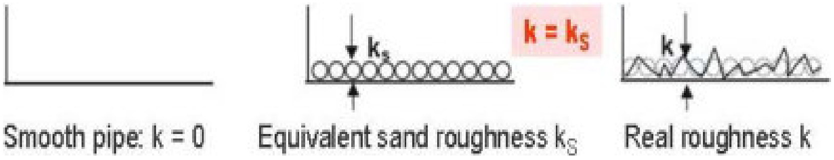


Fig. 1.7:

Illustration of equivalent sand roughness (Figures from M. Eisenhauer)

- estimation of k_S for open channels (details → see chapter 1.4):
  - → on basis of current velocity measurements → calibration
  - → calculation under consideration of grain size distribution parameters

* **ranges of validity of GMS formula and equation by Colebrook-White:**

- subcritical flow (no shooting), quantifiable by die Froude number → see chapter 1.1
- big channel width in comparison to depth of water body
  - → is valid properly speaking for rectangle flow profile with fall away sharply riverside embankments only
  - → practical accepted in case of the following criteria:
    - channel width / depth of water body ≥ 5 → if river embankments are considerable amount more smooth than river ground
    - channel width / depth of water body ≥ 10 → if river embankments and river bank are nearly equal rough
    - channel width / depth of water body ≥ 25 → if river embankments are much more rough as river bank
  - → depth of water body h [m] > 3 * equivalent sand roughness k_S [m]

1.4. Data estimation for hydraulic calculations

1.4.1. Cross-section geometry of open channels

* **criteria regarding the determination of representative cross-sections** (see figure 1.8)**:**

- construction of cross-sections vertical to river axis
- fixed point installation (benchmarks) outside of the flooding area → extension of the cross-sections longer than flooding area
- longitudinal distances accordingly the local conditions:
  - → about 100 m in case of open channels of first order
  - → consideration of distinctive river points: bridges, channel narrowing, weirs

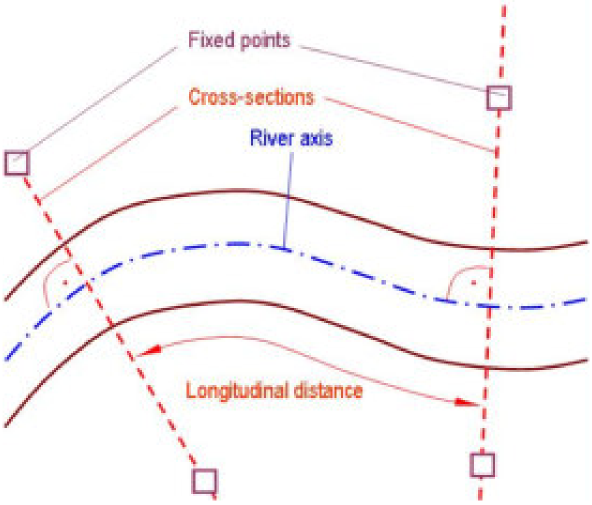


Fig. 1.8:

Construction of cross-sections for hydraulic investigations on open channels (after LfU, 2002)

* **channel segmentation (practical application→ see seminar 5):**

- channel classification from the cross-sectional aspect (for illustration → see figure 1.9):
  - → compacted (not segmented) channel: straight lines from any points of the embankments to the river ground no cut the embankments → practical effect: nearly the same flow velocities within the flow profile
  - → segmented channel: straight lines from the embankments to the river ground cut the embankments (on one side or on both sides) → practical effect: big differences in flow velocities in the central part of the channel in comparison to forelands (flow velocity in the central part > flow velocity in the forelands)

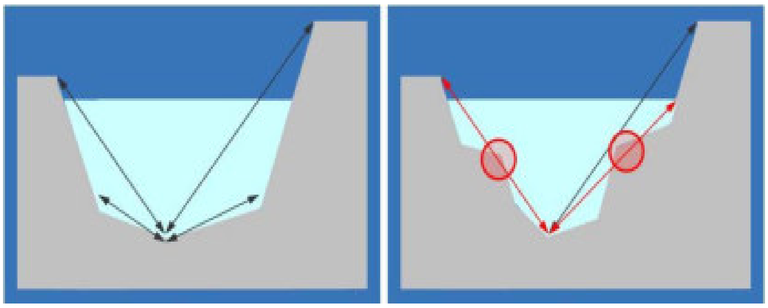


Fig. 1.9:

Compacted and segmented cross-sections (after LfU, 2002)

* **cross-section measurement:**

- measurement methods:
  - → terrestrial measuring by levelling
  - → echo sounder
  - → aerial photo analysis
  - → by means of global position system GPS

  } high precision: approximately ± 5 cm (terrestrial measuring by levelling, echo sounder); at present more inaccurate than ± 5 cm (aerial photo analysis, GPS)
- measurement results:
  - → cross-section length to distinctive points (riversides, control structures, …)
  - → terrain altitudes along the cross-section
  - → water level (with date and time)
  - → river ground
- measurements of bridges: cross-section measuring on upstream side, in the middle and downstream

- measurement of bottom steps and weirs: detailed recording of bottom steps (bottom step = not dammed fall) respectively weirs (weir = dammed fall) if necessary downstream of water uptake facilities / stilling basins
- other matters: notes of further conditions (especially vegetation), photo documentation

* **aims of the analysis of cross-sectional data:**

- estimation of geometric parameters by means of cross-section measurement:
  → cross-section area A
  → wetted perimeter l_U
  → mean water level h
- helpful related to the estimation of hydraulic roughness (STRICKLER or MANNING coefficients)
- Approximation of irregular flow profiles by trapezes / triangles → simplification regarding the estimation of geometric parameters → see figure 1.10

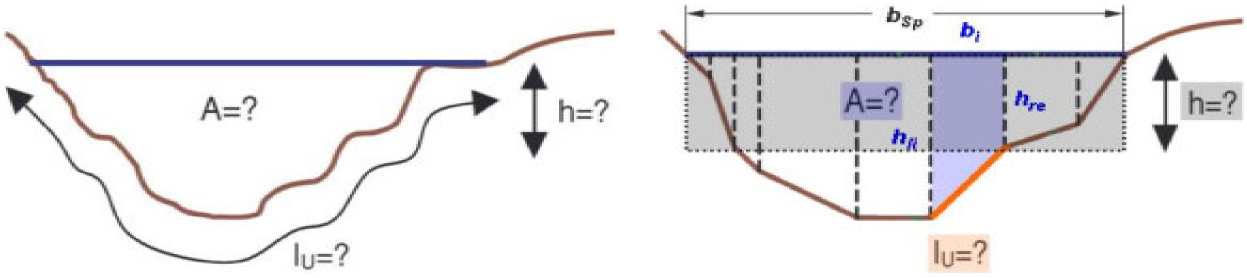


Fig 1.10: Estimation of geometric parameters (after LfU, 2002)

- simplified estimation of cross-section area A (see figure 1.10) → segmentally → superposition:

→ cross-section area A:

$$A_i = \frac{h_{li} + h_{re}}{2} * b_i \tag{1.21}$$

where: A_i – area of segment i [m^2]
h_{li} – water level on the left segment border [m]
h_{re} – water level on the right segment border [m]
b_i – segment width [m]

$$A = \sum_{i=1}^{n} A_i \tag{1.22}$$

where: A – total cross-section area [m^2]
A_i – area of segment i [m^2]

→ wetted perimeter l_U:

$$l_{U,i} = [\, b_i^{\,2} + (h_{li} - h_{re})^2 \,]^{1/2} \tag{1.23}$$

(symbol explanation → see next page)

where: $l_{U,i}$ – wetted perimeter of segment i [m]
h_{li} – water level on the left segment border [m]
h_{re} – water level on the right segment border [m]
b_i – segment width [m]

$$l_U = \sum_{i=1}^{n} l_U \tag{1.24}$$

where: l – total wetted perimeter [m]
$l_{U,i}$ – wetted perimeter of segment i [m]

→ mean water level h:

$$h = \frac{A}{b_{Sp}} \tag{1.25}$$

where: h – mean water level [m]
A – cross-section area [m^2] → after equation 1.22
b_{Sp} – river width [m]

- **practical estimation of cross-sectional dada → see seminars 4, 5 and 8**

1.4.2. Water levels (water marks and drift material marks after flood events)

* **drift material marks:** traces of deposits (flotsam, substances in water, …) respectively degradation (leaves, sediments, …) as an indicator for flood peak

* **drift material:** also called flotsam, jetsam

* **necessity of drift material marks recording:**

- important information during the process of reconstruction of maximum flood peak
  → recording of drift material marks immediately after a flood event
  → drift material marks are representative for flood peak because during flood decrease less drift material is moved than sedimented and the maximum erosion occurs on flood peak
- basis for calibration of hydraulic calculation methods

* **examples for observations immediately after flood events:**

- lay down of vegetation
- drift material deposition (trees, branches, filth, leafs) on barriers
- sedimentation of fine-sandy suspended matter (river erosion, foreland sedimentation)
- removal of leaves and green waste in the foreland in case of smaller flood events
- soil erosion on river narrowing in case of very large flood events
- soiling and water logging phenomena at buildings

- * **drift material mark recording methods:**
  - terrestrial annotation → see above (observations immediately after flood events)
  - aerial photo (photogrammetry) → at present not very precise

- * **please note related to drift material mark analysis:**
  - drift material marks rarely sharp lines
  - reasons for unsharpness:
    - → slight terrain slope
    - → wave action
    - → conditioned by vegetation → lay down of vegetation due to flood events because of big flow velocity and tilt-up movement after flood ending

- * **analysis of water marks at buildings:**
  - important information regarding calibration of hydraulic calculations
  - mostly more exact then drift material marks
  - **please note:** possibly influencing of flood peak by water-surface ascent of the building!

1.4.3. Data for hydraulic roughness characterization

1.4.3.1. Roughness coefficients for GAUCKLER-MANNING-STRICKLER formula

- * **possibilities for the estimation of STRICKLER coefficient k_{St}:**
  - overview on possibilities for the estimation of STRICKLER coefficient → see figure 1.11

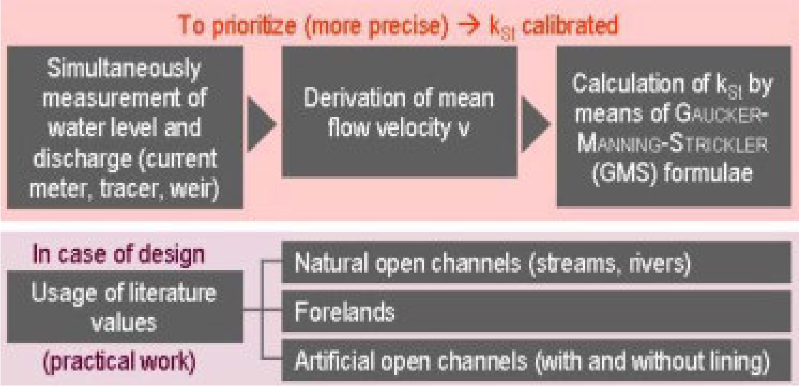


Fig 1.11:

Options for the estimation of STRICKLER coefficient k_{St}

- * **examples for k_{St} calibrations (practical application → see seminar 10):**

► ***example 1 – flood peak (nearly no bursting of a river´s bank):***

- current meter measurement: discharge $Q = 2.4\ m^3/s$
- cross-sectional area $A = 1.6\ m^2$ → flow velocity $v = Q / A = 1.5\ m/s$
- by means of cross-section: wetted perimeter $l_U = 2.1\ m$
- by means of longitudinal profile: river ground slope $I_{So} = 0.015\ m/m$

→ conversion of GMS formula 1.19 to k_{St}:

$$k_{St} = \frac{v}{r_{hy}^{2/3} * I_{So}^{1/2}} = \frac{v}{(A / l_U)^{2/3} * I_{So}^{1/2}} = \frac{1.5\ m/s}{(1.6\ m^2 / 2.1\ m)^{2/3} * (0.015\ m/m)^{1/2}} \approx 15\ m^{1/3}/s$$

► ***example 2 – flood peak (bursting of a river´s bank, otherwise analogous to example 1):***

- current meter measurement: discharge $Q = 15.8\ m^3/s$
- cross-sectional area $A = 17.4\ m^2$ → flow velocity $v = Q / A = 2.1\ m/s$
- by means of cross-section: wetted perimeter $l_U = 8.1\ m$
- by means of longitudinal profile: river ground slope $I_{So} = 0.015\ m/m$

→ conversion of GMS formula 1.19 to k_{St}:

$$k_{St} = \frac{v}{r_{hy}^{2/3} * I_{So}^{1/2}} = \frac{v}{(A / l_U)^{2/3} * I_{So}^{1/2}} = \frac{2.1\ m/s}{(17.4\ m^2 / 8.1\ m)^{2/3} * (0.015\ m/m)^{1/2}} \approx 10\ m^{1/3}/s$$

→ $k_{St} = f$ (water level)
→ hydraulic roughness = f (water level)
→ calibration for different (if necessary for many) water levels

* **literature values for k_{St}:**

- hundreds of literature values available → for better illustration → see chapter 1.7
- before assignment of literature values → exact analysis of hydraulic conditions of the open channel necessary → assignment of literature values to ones of these 3 groups:
  → natural open channels (stream, rivers)
  → artificial open channels (ditches, channels)
  → foreland (areas subject to flooding in case of flood events)
- literature values for natural open channels (stream, rivers) → see table 1.2

Table 1.2: Roughness coefficients k_{St} for natural channels

Kind of open channel	k_{St} [$m^{1/3}/s$]
Plain river with a width of < 30 m in case of floods:	
Straightforward, small ditches, no shoals	30 – 40
Meandered, some troughs and shoals	22 – 30
With stagnant water zones, with weedage, deep troughs	13 – 20
Very high weediness, forelands with many trees and undergrowth	ca. 10
Mountain rivers, streambed not vegetated, steep embankments, riverbank vegetation:	
Gravelly bottom of the water body, stony, single boulders	20 – 35
Rough bottom of the water body with stones and boulders	15 – 25
Ocean of boulders (felsenmeer)	ca. 10

- literature values for artificial open channels → see table 1.3

Table 1.3: Roughness coefficients k_{St} for artificial channels

Kind of open channel	k_{St} [$m^{1/3}/s$]
Concrete channels:	
Smooth down cement, smoothes concrete	85 – 100
Asphalt concrete	72 – 77
Coarse, rough surface	50 – 55
Stonewalled channels:	
Brick wall, fine grouted	70 – 80
Dry-stone or natural stone walls, coarse hewn	45 - 60
Soil channels:	
With loamy river ground	30 – 60
With coarse stones coated respectively with high weediness	< 20 – 30

- literature values for forelands → see table 1.4

Table 1.4: Roughness coefficients k_{St} for forelands

Kind of foreland	k_{St} [$m^{1/3}/s$]
Foreland, flooded in case of flood events:	
Meadow, short grass, no scrub	30 – 40
Meadow, high grass, no scrub	20 – 33
Dense grass / herb vegetation,, several bushes	14 – 29
Medium to dense scrub during winter time	9 – 22
Medium to dense scrub during summer time	6 – 14
Tree population, few undergrowth, water level during flood events below branches	8 – 13
Tree population, few undergrowth, water level during flood events within branches	6 – 10

* **conclusions for segmented channels:**

- in case of measurements of water levels or discharges or in case of calculation of k_{St} as a remainder of GMS formula → flow profile segmentation and water level depended discharge measurements and k_{St} estimations → **practical application → see seminar 10**
- in cas of the usage of literature values → also water level depended segmentation of the flow cross-section and weighting of the k_{St} literature values respectively the proportion of the total cross-sectional area → **practical application → see seminar 9**

1.4.3.2. Roughness coefficients for COLEBROOK-WHITE method

* **hydraulic parameters of COLEBROOK-WHITE method** (see also equation 1.20)**:**

- hydraulic perimeter r_{hy}
- cross-sectional figuration
- meander degree
- equivalent sand roughness k_S

* **estimation of equivalent sand roughness k_S:**

- analogous to roughness coefficient k_{St} of GMS formula by calibration on base of water level and discharge measurements respectively from literature (estimation by means of grain size distribution parameters)
- estimation by means of grain size distribution parameters → see table 1.5

Table 1.5: Calculation formulae for equivalent sand roughness k_S by means of grain size distribution curves

Author		Formula
KAMPHIUS (1974)		$k_S = 2 * d_{50}$
DVWK (1990)		$k_S = 3.6 * d_{50}$
MERTENS (1997)		$k_S = 2.5 * d_{50}$
DITTRICH (1998)	Gravel	$k_S = 3.5 * d_m$
	Coarse gravel, stones	$k_S = 3.5 * d_{84}$

k_S - equivalent sand roughness [mm]
d_i - particle size [mm] at i percent material passing
d_m - mean particle diameter [mm]

- estimation of d_{50}, d_{84} and d_m:
  - → d_{50} and d_{84} by means of grain size distribution curves → for example see figure 1.12
  - → **practical application → see seminar 11**

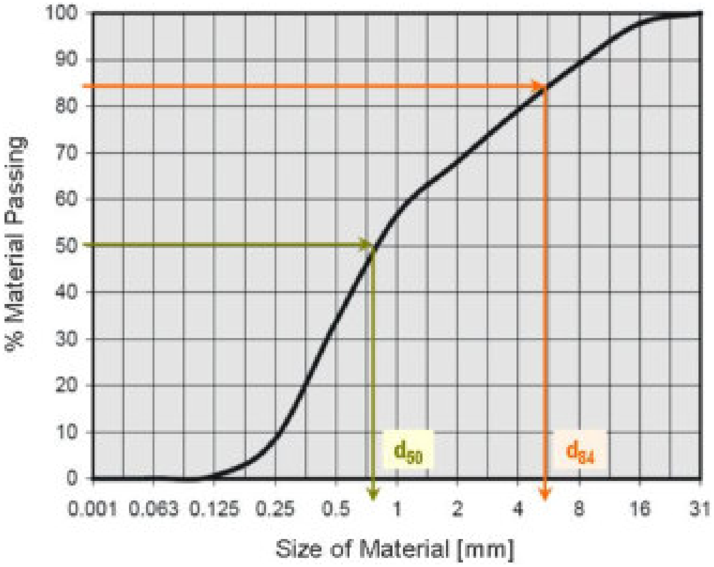


Fig. 1.12:

Example for the estimation of d_{50} and d_{84} by means of a grain distribution curve

→ d_m after equation 1.26:

$$d_m = \sum_{i=1}^{n} (d_i * \Delta p_i) \qquad (1.26)$$

where: d_m - mean particle diameter [mm]
d_i - particle diameter [mm] at i percent material passing
Δp_i - percentage share of grain fraction i [%]

→ mean particle diameter d_m is often nearly equal to particle diameter at 50 % material passing: $d_m = d_{50}$

- most common approaches: DVWK, MERTENS (see table 1.5)

1.4.4. Vegetation parameters

* **vegetation characteristics:**

- vegetation geometry → horizontal and vertical extension → mapping
- roughness → depending on vegetation structure → 2 different groups from hydraulic point of view

* **vegetation group 1: grasses, herbs, young reeds:**

- only little resistance against running water
- snap in case of floods
- no separate consideration in determination of STRICKLER coefficient k_{St}

* **vegetation group 2: perennial trees and bushes:**

- can set resistance against running water
- two hydraulic different cases:
  - → case 1: single trees with distances > 2 m
    - population with great openings
    - watered during floods
    - nearly no blockage danger
    - no or only small additional hydraulic resistance

    → no separate k_{St} coefficient
  - → case 2: dense tree or bush vegetation
    - compact population
    - flowed round during floods
    - high blockage danger
    - consideration of an additional hydraulic resistance
    - from hydraulic point of view = vertical barrier

    → separate k_{St} coefficient respectively cross-section area deduction (example → chapter 1.5)
- **practical vegetation mapping → see seminar 7**

1.5. Application examples for hydraulic calculations

1.5.1. Examples for GAUCKLER-MANNING-STRICKLER formula

1.5.1.1. Weighting methods for the estimation of STRICKLER coefficient

* **necessity of weightings:**

- main problem of GMS formula = estimation of k_{St} coefficient → see chapter 1.4.3.1
- k_{St} depends on: wetting degree, meander degree, bed-load transportation, river bank vegetation, weediness, discharge barriers)
- k_{St} → depend on water level and on season (in case of river ground and bank vegetation only)
- capture of influencing factors on k_{St} by weighting factors
- three at present usual weighting methods:
  - → COWAN weighting method
  - → EINSTEIN / HORTON weighting method
  - → SELLIN weighting method

* **COWAN weighting method (practical application → see seminar 12):**

- is originated from English-speaking region → usage of MANNING coefficient instead of STRICKLER coefficient k_{St} (usually in German-speaking regions)→ $n = 1 / k_{St}$ (see chapter 1.3.2)
- calculation equation:

$$n = (n_0 + n_1 + n_2 + n_3 + n_4) * m_5 \tag{1.27}$$

where: n – weighted MANNING coefficient [$s/m^{1/3}$]
n_i – COWAN factor [$s/m^{1/3}$] → see table 1.6
m_5 – meander degree [ ] → see also table 1.6

Table 1.6: Factors for COWAN method

Indication of COWAN factor	Factor [ ]
n_0: Characterisation of river bed material (further values see chapter 1.4.3.1):	
Soil	0.020
Fractured gravel	0.025
Fine gravel	0.024
Coarse gravel	0.028
n_1: Irregularities of river bed material:	
Negligible	0.000
Slight	0.005
Medium	0.010
Heavy	0.020
n_2: Changes in cross-sectional figuration:	
Not existent	0.000
Occasional	0.005
Often	0.010 – 0.015
n_3: Influence of barriers within the watered cross-section:	
Negligible	0.000
Slight	0.010 – 0.015
Medium	0.020 – 0.030
Heavy	0.040 – 0.060
n_4: Vegetation influences:	
Negligible	0.000 – 0.005
Slight	0.005 – 0.010
Medium	0.010 – 0.025
Heavy	0.025 – 0.050
Very heavy	0.050 – 0.100
m_5: Meander degree of the open channel:	
Slight	1.000
Medium	1.150
Heavy	1.300

► *application example 1: slight meander stream, river bed: soil, very low vegetation:*

- flow profile → see figure 1.13 a

a) Application example 1 b) Application example 2 c) Application example 3

Fig 1.13: Flow profiles of application examples 1 to 3 (Photos: LfU, 2002)

- weighting factors according to table 1.6:
  - → river bed material: $n_0 = 0.020$ s/m$^{1/3}$
  - → irregularities of river bed material: $n_1 = 0.000$ s/m$^{1/3}$
  - → changes in cross-sectional figuration: $n_2 = 0.000$ s/m$^{1/3}$
  - → influence of barriers: $n_3 = 0.000$ s/m$^{1/3}$
  - → vegetation influences: $n_4 = 0.005$ s/m$^{1/3}$
  - → meander degree: $m_5 = 1.150$
- n and k_{St}: $n = (0.020 + 0.005) * 1.150 = 0.0288$ m/s$^{1/3}$ → $k_{St} = 1 / n = 35$ m$^{1/3}$/s

► *application example 2: slight meander stream, river bed: fine gravel, low – medium vegetation:*

- flow profile → see figure 1.13 b
- weighting factors according to table 1.6:
  - → river bed material: $n_0 = 0.024$ s/m$^{1/3}$
  - → irregularities of river bed material: $n_1 = 0.000$ s/m$^{1/3}$
  - → changes in cross-sectional figuration: $n_2 = 0.000$ s/m$^{1/3}$
  - → influence of barriers: $n_3 = 0.000$ s/m$^{1/3}$
  - → vegetation influences: $n_4 = 0.010$ s/m$^{1/3}$
  - → meander degree: $m_5 = 1.100$
- n and k_{St}: $n = (0.024 + 0.010) * 1.100 = 0.0374$ m/s$^{1/3}$ → $k_{St} = 1 / n = 27$ m$^{1/3}$/s

► *application example 3: straight line stream, river bed: soil, very dense vegetation:*

- flow profile → see figure 1.13 c
- weighting factors according to table 1.6:
  - → river bed material: $n_0 = 0.020$ s/m$^{1/3}$
  - → irregularities of river bed material: $n_1 = 0.000$ s/m$^{1/3}$
  - → changes in cross-sectional figuration: $n_2 = 0.005$ s/m$^{1/3}$
  - → influence of barriers: $n_3 = 0.005$ s/m$^{1/3}$
  - → vegetation influences: $n_4 = 0.080$ s/m$^{1/3}$
  - → meander degree: $m_5 = 1.000$
- n and k_{St}: $n = (0.11) * 1.000 = 0.110$ m/s$^{1/3}$ → $k_{St} = 1 / n = 9$ m$^{1/3}$/s

* **EINSTEIN / HORTON weighting method (practical application → see seminar 12):**

- identification of different roughnesses by weighting of STRICKLER coefficient k_{St} by means of wetted perimeter l_U
- calculation equation:

$$k_{St} = \left[\frac{\sum_{i=1}^{n} l_{U,i}}{\sum_{i=1}^{n} \frac{l_{U,i}}{k_{St,i}^{3/2}}} \right]^{2/3} \qquad \textbf{\textit{(1.28)}}$$

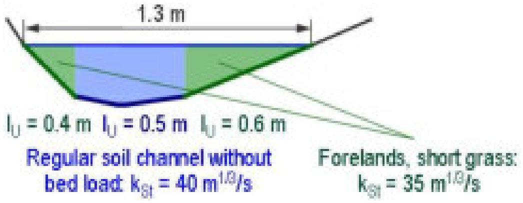
where: k_{St} – weighted STRICKLER coefficient [m$^{1/3}$/s]
$l_{U,i}$ – wetted perimeter of the subsection i [m]
$k_{St,i}$ – STRICKLER coefficient of the subsection i [m$^{1/3}$/s]

- applicability of EINSTEIN / HORTON method in case of compacted, <u>not segmented</u> channels with distinctive hydraulic roughness differences within the cross-section (for example roughness of left river side ≠ right river side ≠ river bed roughness)
- advantage of EINSTEIN / HORTON method: more precise in comparison to COWAN method → EINSTEIN / HORTON method includes river width and condition at river embankments and river bed
- disadvantage of EINSTEIN / HORTON method: costlier in comparison to COWAN method → strict cross-sectional information necessary → requires cross-sectional measurement

► *application example: slight meander stream, river bed: soil, very low vegetation → analogue example 1 of COWAN method:*

- cross-sectional profile → see figure 1.13 a, however with two different widths: 1.3 m and 2.5 m
- COWAN method results: $n = 0.0288\ \mathrm{m/s}^{1/3}$ and $k_{St} = 35\ \mathrm{m}^{1/3}/\mathrm{s}$
- cross-sections, vegetation and STRICKLER roughness coefficients → see figure 1.14

Flow profile with a width of 1.3 m:



Flow profile with a width of 2.5 m:

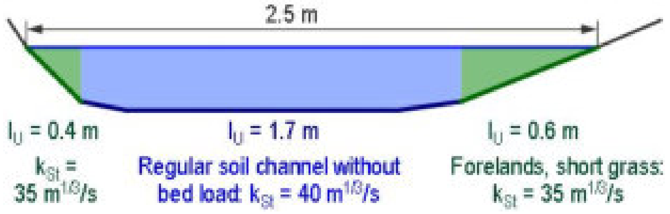


Fig. 1.14:

Cross-sections, vegetations and STRICKLER roughnesses fort he application example

Calculation of k_{St} by means of equation 1.28 for b = 1.3 m:

$$k_{St} = \left(\frac{0.4\ \mathrm{m} + 0.5\ \mathrm{m} + 0.6\ \mathrm{m}}{\dfrac{0.4\ \mathrm{m}}{(35\ \mathrm{m}^{1/3})^{3/2}} + \dfrac{0.5\ \mathrm{m}}{(40\ \mathrm{m}^{1/3})^{3/2}} + \dfrac{0.6\ \mathrm{m}}{(35\ \mathrm{m}^{1/3})^{3/2}}} \right)^{2/3} = 36.5\ \mathrm{m}^{1/3}/\mathrm{s}$$

→ result approximately comparable with COWAN method

Calculation of k_{St} by means of equation 1.28 for b = 2.5 m:

$$k_{St} = \left(\frac{0.4\ \mathrm{m} + 1.7\ \mathrm{m} + 0.6\ \mathrm{m}}{\dfrac{0.4\ \mathrm{m}}{(35\ \mathrm{m}^{1/3})^{3/2}} + \dfrac{1.7\ \mathrm{m}}{(40\ \mathrm{m}^{1/3})^{3/2}} + \dfrac{0.6\ \mathrm{m}}{(35\ \mathrm{m}^{1/3})^{3/2}}} \right)^{2/3} = 38.0\ \mathrm{m}^{1/3}/\mathrm{s}$$

→ result is noticeably different from COWAN method

→ for a infinite river width would be calculated: $k_{St} = 40\ m^{1/3}/s$

→ $k_{St} = f$ (river width, river depth)

→ advantages compared to COWAN method

*** SELLIN weighting method (practical application → see seminar 12):**

- application for segmented open channels with very big differences in regard to flow velocities (open channel, forelands):
  → turbulences at boundaries
  → transformation of kinetic energy to heat
  → delayed flow in the central (deep) part of the open channel
  → accelerated flow in the shallow part of the channel

- methodology: consideration of the boundary height during calculation of l_U and r_{hy} in the central (deep) part of the open channel
- calculation of the wetted perimeter <u>of the central (deep) part of the open channel</u> l_U after SELLIN (see figure 1.15):

$$l_{U,HG} = l_{U,S} + h_{G,li} + h_{G,re} \tag{1.29}$$

where: $l_{U,HG}$ – wetted perimeter in the central part of the open channel [m]
$l_{U,S}$ – wetted perimeter of the river bank in the central part of the open channel [m]
$h_{G,li}$ – boundary height on the left border of the central part of the open channel [m]
$h_{G,re}$ – boundary height on the right border of the central part of the open channel [m]

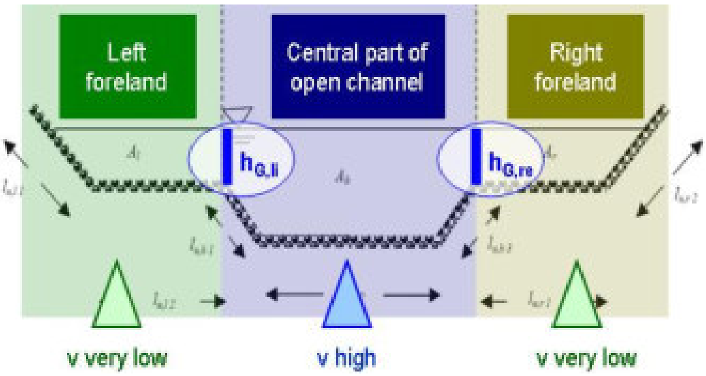


Fig. 1.15:

Illustration for SELLIN weighting method (after LfU, 2002)

- calculation of hydraulic radius r_{hy} of <u>central part of open channel</u> by SELLIN:

$$r_{hy,HG} = \frac{A_{HG}}{l_{U,HG}} \tag{1.30}$$

mit: $r_{hy,HG}$ – hydraulic radius of central part of open channel [m]
A_{HG} – cross-sectional area of central part of open channel [m^2]
$l_{U,HG}$ – wetted perimeter of central part of open channel [m]

- consideration of parts with different k_{St} values <u>within</u> a segment (for example forelands) by application of EINSTEIN / HORTON method

- segmentally application of flow velocities and discharges:
  - → calculation of mean flow velocity v section by section by GMS formula:

$$v_i = k_{St,\,i} * r_{hy,\,i}^{\,2/3} * I_{So,\,i}^{\,1/2} \qquad (1.31)$$

where: v_i – mean flow velocity of section i [m/s]
$r_{hy,\,i}$ – hydraulic radius of section i [m]
$I_{So,\,i}$ – river ground slope of section i [m/m]

→ calculation of discharge Q section by section:

$$Q_i = v_i * A_i \qquad (1.32)$$

where: Q_i – discharge of section i [m³/s]
v_i – mean flow velocity of section i [m/s]
A_i – cross-sectional area of section i [m²]

→ total discharge results in the sum of single discharges: $Q_{ges} = \sum Q_i$ **(1.33)**

1.5.1.2. Un-sectioned flow profiles

Example 1: estimation of maximum discharge capacity of an artificial channel with uniform hydraulic roughness → solution in one step

* **given information** (→ see figure 1.16)**:**

- artificial channel, trapezium profile
- river ground and embankments → asphalt concrete
- width of river ground: 6.0 m

b_{Sp}	width of water level in m
b_{So}	width of river ground in m
1:m	gradient in m/m
h	water depth in m
A	watered cross-sectional area in m²
l_U	wetted perimeter in m

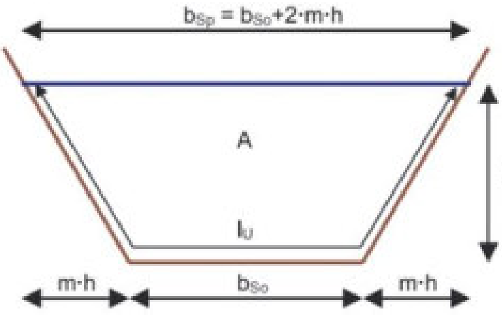


Fig. 1.16:

Un-sectioned flow profile with uniform hydraulic roughness (LfU, 2002)

- embankments gradients right and left: 1:1
- channel depth: 3.5 m:
  - → of that freeboard: 0.5 m
  - → maximum water depth: 3.0 m
- river ground gradient: 0.5 $^0/_{00}$ = 0.0005 m/m

* **calculation steps:**

- watered cross-sectional area:

$$A = b_{So} * h + m * h^2 \qquad (1.34)$$

$$A = 6.0\ \text{m} * 3.0\ \text{m} + 1 * (3.0\ \text{m})^2 = 27\ \text{m}^2$$

Symbol explanation →see figure 1.16

- wetted perimeter:

$$l_U = b_{So} + 2\ h\ (1 + m^2)^{1/2} \qquad (1.35)$$

$$l_U = 6.0\ \text{m} + 2 * 3.0\ \text{m} * (1 + 1^2)^{1/2} = 14.5\ \text{m}$$

Symbol explanation →see figure 1.16

- hydraulic radius by equation 1.16, 2nd term → see page 11:

  $r_{hy} = 27\ \text{m}^2 / 14.5\ \text{m} = 1.86\ \text{m}$

- calculation of flow velocity by GMS formula 1.19:

  → STRICKLER coefficient k_{st} from table 1.3: 72 – 77 $\text{m}^{1/3}/\text{s}$ → selected: 75 $\text{m}^{1/3}/\text{s}$

  → $v = 75\ \text{m}^{1/3}/\text{s} * 1.86\ \text{m}^{2/3} * (0.0005)^{1/2} = 2.5\ \text{m/s}$

- calculation of discharge Q:

  Q = v * A = 2.5 m/s * 27 m^2 = 68.5 m^3/s **≈ 70 m^3/s → maximum discharge capacity**

- verification of flow conditions (streaming or shooting flow) by equation 1.3 → GMS formula is valid for streaming only:

$$\mathbf{Fr} = \frac{1.2 * 2.5\ \text{m/s}}{(9.81\ \text{m/s}^2 * 27\ \text{m}^2 / 12\ \text{m})^{1/2}} = \mathbf{0.64} \quad \rightarrow \textbf{streaming}$$

Example 2: estimation of water level h for a given discharge value Q_{Soll} = 40 m^3/s for the same hydraulic conditions like in example 1 (same channel)

* **methodology:** iterative estimation respective figure 1.17

* **iteration step 1:**

- choice of h_0: $h_0 = 2.0$ m
- calculation of A, l_U and r_{hy}: A = 16.0 m^2, l_U = 11.7 m, r_{hy} = 1.37 m

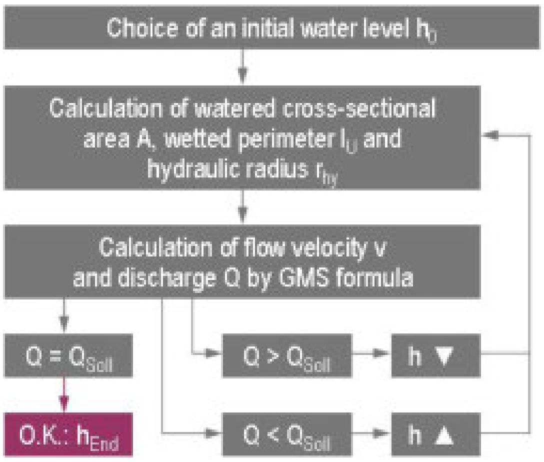


Fig. 1.17:

Iterative estimation of a water level h in case of a given discharge value Q_{Soll}

- calculation of v and Q: v = 2.1 m/s, Q = 33.1 m^3/s
- comparison of Q with Q_{Soll}: $Q < Q_{Soll}$
- conclusion: increase of h → for example from 2.0 to 2.5 m

* **iteration step 2:**

- new h_1: $h_1 = 2.5$ m
- calculation of A, l_U and r_{hy}: A = 21.3 m^2, $l_U = 13.1$ m, $r_{hy} = 1.63$ m
- calculation of v and Q: v = 2.3 m/s, Q = 49.3 m^3/s
- comparison of Q with Q_{Soll}: $Q > Q_{Soll}$
- conclusion: decrease of h → for example from 2.5 to 2.25 m

* **iteration step 3:**

- new h_2: $h_2 = 2.25$ m
- calculation of A, l_U and r_{hy}: A = 18.6 m^2, $l_U = 12.4$ m, $r_{hy} = 1.50$ m
- calculation of v and Q: v = 2.2 m/s, Q = 40.9 m^3/s
- comparison of Q with Q_{Soll}: $Q \approx Q_{Soll}$
- conclusion: **h = final h = h_{End} = 2.25 m**

Example 3: design of a fish way up aid on an existing weir for a not segmented flow profile with non-uniform hydraulic roughness

* **given information:**

- artificial channel, trapezium profile → see figure 1.18
- ecological boundary conditions: fish way up possible for $v \leq 0.5$ m/s → in that case possible?

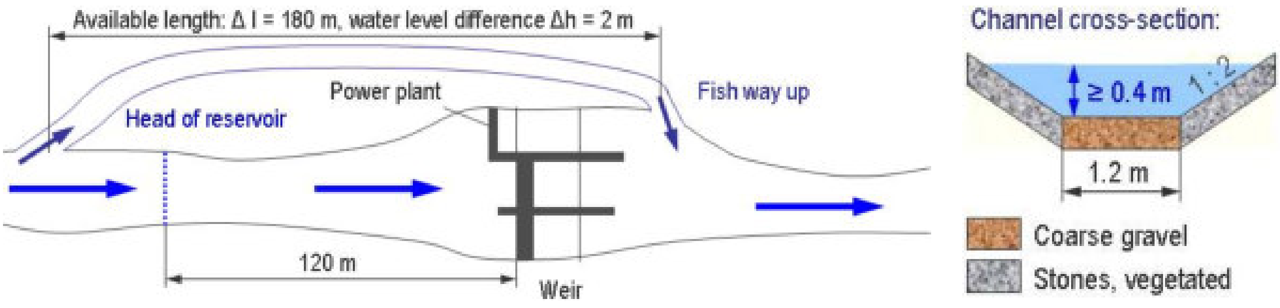


Fig. 1.18: Boundary conditions for example 3 (fish way up), after LfU (2002)

* **calculation steps:**

- needed minimum discharge: $Q = v * A = 0.5\ m/s * 0.8\ m^2 = 0.4\ m^3/s = 400\ l/s$
- wetted perimeters l_U: embankments: $l_{U,\,Bö} = 2 * 0.9\ m = 1.8\ m$, river ground: $l_{U,\,So} = 1.2\ m$
- STRICKLER coefficients: embankments: $k_{St,\,Bö} = 10\ m^{1/3}/s$, river ground: $k_{St,\,So} = 22\ m^{1/3}/s$
- equivalent roughness by EINSTEIN / HORTON (→ see equation 1.28): $12\ m^{1/3}/s$

- basis: GMS flow formula → see equation 1.19
  - → transform to river ground slope I_{So} → maximum river ground slope for $v = 0.5\ m/s$
  - → calculation of maximum river ground slope → $I_{So} = 0.01$ (1 %)
  - → minimum channel length l_{min} for fish way up: $l_{min} = \Delta h / I_{So} = 2\ m / 0.01 = 200\ m$

* **conclusions:**

- available channel length (see figure 1.18): $\Delta l = 180\ m$
- minimum available length $l_{min} = 200\ m$

→ channel with meander and slow-down section

1.5.1.3. Sectioned flow profile with non-uniform hydraulic roughness

* **given information:**

- built out open channel with forelands → see figure 1.19
- river ground material: boulders and mud with irregularities, river ground slope: $I_{So} = 0.0012$
- embankments and forelands with grass vegetation
- estimated flood water level: 201.20 m NN
  - → flooding of forelands
  - → flood water level not yet in the area of the trees

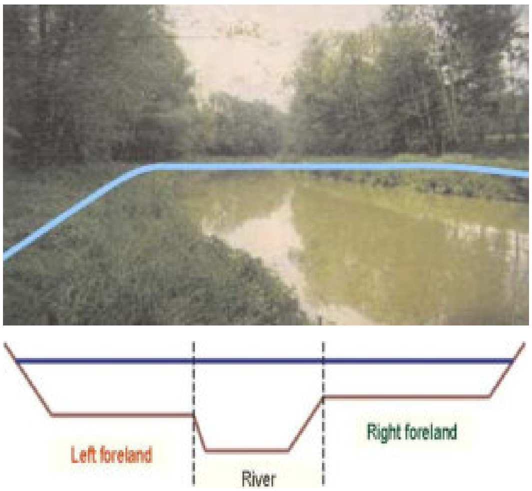


Fig. 1.19:

Sectioned flow profile with non-uniform hydraulic roughness (Photo: LfU, 2002)

* **aim:** estimation of flood peak discharge

* **calculation steps:**

a) calculation of geometric parameters:
  - cross-sectional areas of the right and left forelands and of the river A_{Vl}, A_F, A_{Vr}
  - wetted perimeters of the right and left forelands and of the river $l_{U,\,Vr}$, $l_{U,\,F}$, $l_{U,\,Vr}$
  - hydraulic radii of the right and left forelands and of the river $r_{hy,\,Vl}$, $r_{hy,\,F}$, $r_{hy,\,Vr}$
  - boundary heights on the right and left borders of the central part of the open channel: $h_{G,li}$, $h_{G,re}$

b) estimation of weighted roughnesses k_{St} (EINSTEIN & HORTON) separately for the central part of the open channel and for the forelands (is necessary) → equation 1.28

c) segmented flow velocity and discharge estimation: v_{Vl}, v_F, v_{Vr}, Q_{Vl}, Q_F, Q_{Vr}

d) calculation of total discharge by summation of the segment discharges → equation 1.33

* **solution steps:**

a) calculation of geometric parameters (see figure 1.20):

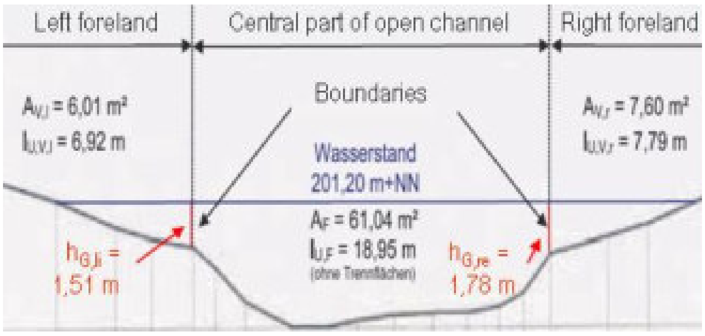


Fig 1.20:

Geometric parameters for the application example (LfU, 2002)

- wetted perimeter of the river by SELLIN (see equation 1.29):
  → $l_{U,\,HG} = 18.95\text{ m} + 1.51\text{ m} + 1.78\text{ m} = 22.24\text{ m}$
- hydraulic radii:
  → forelands: left: $r_{hy,\,Vl} = 0.87$, right: $r_{hy,\,Vr} = 0.98$
  → river (by SELLIN → see equation 1.30): $r_{hy,\,F} = 2.74$
- boundaries between river and left respectively right forelands → see figure 1.20

b) estimation of hydraulic roughnesses k_{St}:
- left foreland: grass (uniform) → $k_{St,\,li} = 25\text{ m}^{1/3}/\text{s}$
- right foreland: grass (uniform) → $k_{St,\,re} = 25\text{ m}^{1/3}/\text{s}$
- river: boulders, mud with irregularities (uniform) → $k_{St,\,F} = 30\text{ m}^{1/3}/\text{s}$

c) segmented flow velocity and discharge estimation:

- flow velocities by GMS formula:
  → left foreland: $v_{Vl} = 25\text{ m}^{1/3}/\text{s} * (0.87\text{ m})^{2/3} * (0.0012)^{1/2} = 0.79\text{ m/s}$
  → right foreland: $v_{Vr} = 25\text{ m}^{1/3}/\text{s} * (0.98\text{ m})^{2/3} * (0.0012)^{1/2} = 0.86\text{ m/s}$
  → river: $v_F = 30\text{ m}^{1/3}/\text{s} * (2.74\text{ m})^{2/3} * (0.0012)^{1/2} = 2.03\text{ m/s}$
- discharge values:
  → left foreland: $Q_{Vl} = v_{Vl} * A_{Vl} = 0.79\text{ m/s} * 6.01\text{ m}^2 = 4.7\text{ m}^3/\text{s}$
  → right foreland: $Q_{Vr} = 6.5\text{ m}^3/\text{s}$
  → river: $Q_F = 124.2\text{ m}^3/\text{s}$

d) calculation of total discharge by summation of the segment discharges → equation 1.33

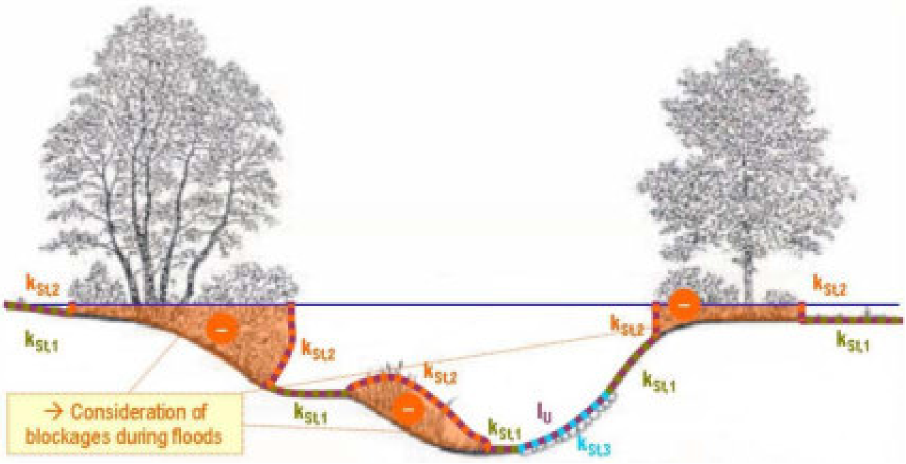
$Q_{ges} = Q_{Vl} + Q_F + Q_{Vr} = (4.7 + 124.2 + 6.5)\text{ m}^3/\text{s} = \underline{135.4\text{ m}^3/\text{s}}$

1.5.1.4. Flow profile with differentiated roughnesses because of heavy vegetation

*** methodology of cross-section area deduction method:**

- application in case of dense vegetation at river ground and/or at embankments → see figure 1.21

  a) trees, bushes and reeds



  b) single trees without bushes

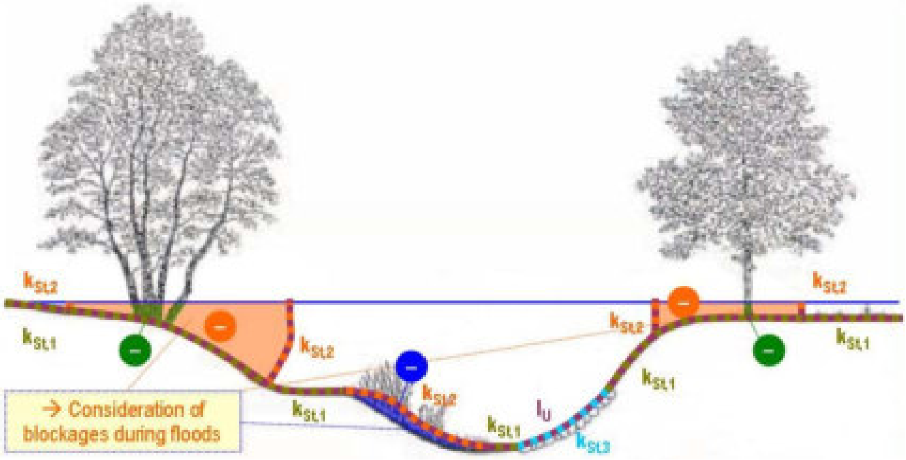


Fig. 1.21:

Cross-section area deduction method (Figure basis: LfU)

- method application independent on kind of channel (not segmented, segmented)
- deduction of cross-sectional areas, where flow velocities are very low, from total cross-sectional area (orange areas in figure 1.21 a)
- estimation of wetted perimeter under consideration of the deducted areas
- assignment of k_{St} values respectively hydraulic roughnesses → in the most cases: estimation of equivalent roughness by k_{St} weighting by EINSTEIN / HORTON (not segmented and segmented channel)
- calculation of v and Q in one step (not segmented channels) respectively segmentally calculation by SELLIN (for segmented channels)

► **distinctions of hardwood and softwood populations:**

  - complete area deduction only in case of dense hardwood vegetation (for example bushes, trees with very small stem distances and trees which stand in flow direction)

- no area deduction in case of grass and young reeds (during floods → lay down in flow direction → reeds k_{St} coefficient analogue grass (20 … 33 $m^{1/3}/s$)
- only 1/3 area deduction in case of full grown reeds and other softwood populations → during floods: lay down in flow direction (blue colour in figure 1.21 b, in comparison to figure 1.21 a called down about 2/3

► **distinctions of single trees** (→ see figure 1.21 b)**:**

- complete area deduction but no consideration in wetted perimeter calculation l_U and no separate k_{St} coefficient → fuselage flows of single trees precise enough considered → no big blockage risks

* **example regarding the application of cross-section area deduction method:**

► **given information:**

- near-natural reconstructed rectangle flow profile → see figure 1.22

Fig. 1.22:

Near-natural reconstructed rectangle flow profile (LfU, 2002)

- maximum expansion water level: 2.0 m
- river ground slope: $I_{So} = 0.005$ m/m
- borders dense vegetated (grass, reeds, willows), river ground materials: boulders and mud

► **aim:** estimation of discharge capacity changes in case of flood events (before and after reconstruction)

► **methodology:**

- estimation of discharge capacity before reconstruction
- estimation of discharge capacity after reconstruction
- discharge capacity comparison

► **estimation of discharge capacity before reconstruction:**

- flow profile before reconstruction → see figure 1.23
- STRICKLER coefficients and wetted perimeter → see also figure 1.23

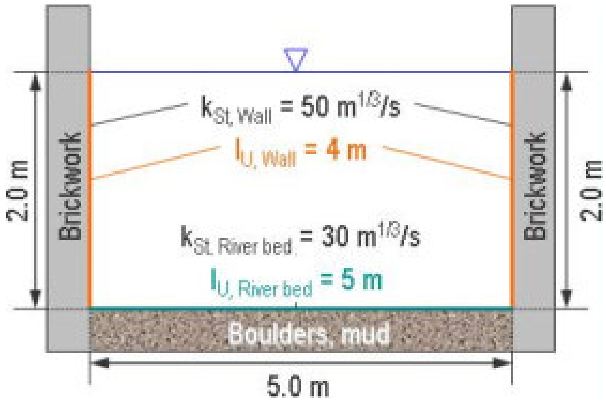


Fig. 1.23:

Flow profile before near-natural reconstruction (after LfU, 2002)

- equivalent roughness by EINSTEIN / HORTON → see equation 1.28:

$$k_{St} = \left(\frac{4\ m + 5\ m}{\dfrac{4\ m}{(50\ m^{1/3}/s)^{3/2}} + \dfrac{5\ m}{(30\ m^{1/3}/s)^{3/2}}} \right)^{2/3} = 36\ m^{1/3}/s$$

- hydraulic radius r_{hy}: $r_{hy} = 10\ m^2 / 9\ m = 1.11\ m$
- discharge Q: $Q = (k_{St} * r_{hy}{}^{2/3} * I_{So}{}^{1/2}) * A = 36\ m^{1/3}/s * (1.1\ m)^{2/3} * (0.005\ m/m)^{1/2} * 10\ m^2$
  $Q = 27.3\ m^3/s$

► **estimation of discharge capacity after reconstruction:**

- flow profile after reconstruction → see figure 1.24
- STRICKLER coefficients and wetted perimeter → see also figure 1.24

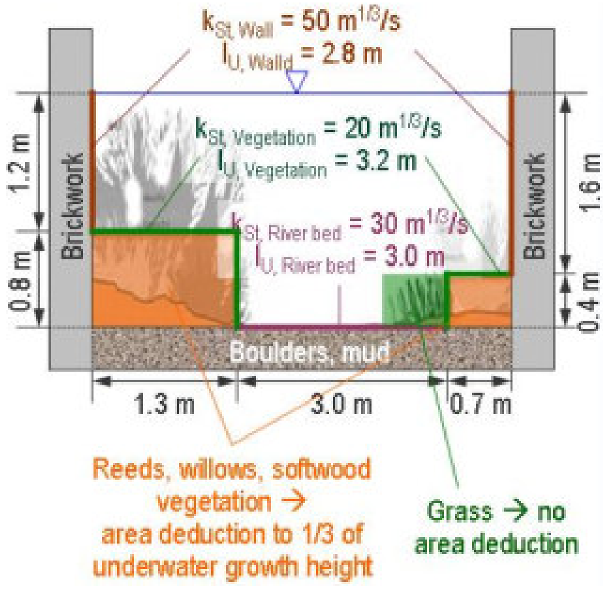


Fig. 1.24:

Flow profile after near-natural reconstruction (after LfU, 2002)

- cross-sectional area: $A = 10\ m^2 - (1.3\ m * 0.8\ m) - (0.7\ m * 0.4\ m) = 8.68\ m^2$

- wetted perimeter l_U and STRICKLER coefficients k_{St}:
  - → brickwork left and right: $k_{St} = 50\ m^{1/3}/s$, $l_U = 2.8\ m$
  - → vegetation: $k_{St} = 20\ m^{1/3}/s$, $l_U = 3.2\ m$
  - → river ground: $k_{St} = 30\ m^{1/3}/s$, $l_U = 3.0\ m$

- equivalent roughness by EINSTEIN / HORTON → see equation 1.28:

$$k_{St} = \left(\frac{2.8\ m + 3.2\ m + 3.0}{\dfrac{2.8\ m}{(50\ m^{1/3}/s)^{3/2}} + \dfrac{3.2\ m}{(20\ m^{1/3}/s)^{3/2}} + \dfrac{3.0}{(30\ m^{1/3}/s)^{3/2}}} \right)^{2/3} = 28\ m^{1/3}/s$$

- hydrailic radius r_{hy}: $r_{hy} = 8.68\ m^2 / 9\ m = 0.96\ m$

- discharge Q: $Q = (k_{St} * r_{hy}^{2/3} * I_{So}^{1/2}) * A = 28\ m^{1/3}/s * (0.96\ m)^{2/3} * (0.005\ m/m)^{1/2} * 8.68\ m^2$

  $Q = 16.8\ m^3/s$

► **discharge capacity comparison:**

- discharge capacity before reconstruction: $27.3\ m^3/s$
- discharge capacity after reconstruction: $16.8\ m^3/s$

→ decrease of discharge capacity about nearly 40 %

- practical application of GAUCKLER-MANNIG-STRICKLER formula → see seminar 13

1.5.2. Example for a hydraulic calculation by means of COLEBROOK-WHITE method

* **main task:** estimation of hydraulic resistance coefficient λ (analogue k_{St} of GMS formula)

* **λ-estimation steps:**

- assignment regarding to channel structure → compacted, segmented (see figure 1.9)
- assignment regarding to cross-sectional figuration → natural open channels in principle relatable to two cross sectional figurations:
  - → rectangle channel (wide rivers)
  - → trapezium channels (all other open channels)
- estimation of river ground and embankment roughnesses → estimation of equivalent sand roghnesses k_S → see figure 1.7 and chapter 1.4.3.2:
  - → uniform ground and embankment roughnesses → constant k_S
  - → inhomogeneous ground and embankment roughnesses → k_S section by section
- estimation of cross-section resistance → for example islands or weedage at the river ground or at the embankments (if existent)
- characterisation of straightforwardness → consideration of flow resistance factors, caused by river straightforwardness: curvature, meander, bays and cut-off meander (if existent)

* **application example: straight and wide rectangle channel → river Donau near Binzwangen**

► **given information:**

- wide rectangle channel
- flow profile: river width b: 38 m, water depth h: 2.2 m → influence of embankments negligible
- river ground slope: 0.001 m/m (0.1 %)
- river ground and embankment material → gravel, grain size distribution curve → see Fig. 1.25

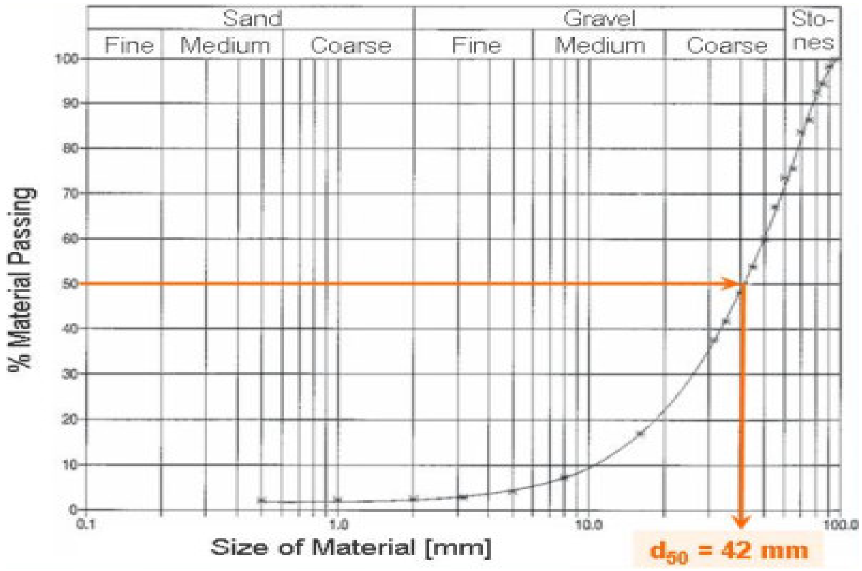


Fig. 1.25:

Sediment grain size distribution curve of river Donau near Binzwangen (after LfU, 2002)

► **aim:** discharge estimation for a water level of 2.2 m

► **solution steps:**

- **channel structure** → compacted, not segmented
- **cross-sectional figuration** → rectangle
- **river ground roughness** → k_S → λ:
  - → k_S calculation methods → see table 1.5, chapter 1.4.3.2
  - → most usually methods in Germany: methods by DVWK and MERTENS:
    k_S by DVWK: 151 mm
    k_S by MERTENS: 105 mm
  - → calculation of λ for a wide rectangle channel:

$$\left[\frac{8}{\lambda}\right]^{0.5} = 2.5 \ln \left[\frac{h}{k_S}\right] + 6.02 \qquad (1.36)$$

where: λ – river ground roughness (resistance coefficient) [ ]
h – water depth [m]
k_S – equivalent sand roughness [m]

$$\left[\frac{8}{\lambda}\right]^{0.5} = 2.5 \ln \left[\frac{2.2\text{ m}}{(0.105 \ldots 0.151\text{ m})}\right] + 6.02 = 12.72 \ldots 13.63$$

- **cross-section resistance** and **straightforwardness** → in that case not necessary (straight pathway)
- **mean flow velocity by Colebrook-White** → see equation 1.20
  - → hydraulic radius $r_{hy} = 1.97$ m
  - → **v** = (12.72 … 13.63) * (9.81 m/s^2 * 1.97 m * 0.001 m/m)$^{0.5}$ = **1.76 … 1.89 m/s**
- **discharge Q: Q** = v * A = (1.76 … 1.89 m/s) * 83.6 m^2 = **147 … 158 m^3/s**
- **check of the range of validity:**
  - → streaming water flow (no shooting flow) → Froude number: Fr = 0.38 … 0.41 < 1 → streaming flow → O.K.
  - → wide channel width in comparison to river depth → b / h = 38 m / 2.2 m = 17.3 > 10 → O.K. (see chapter 1.3.2)
  - → great water depth in comparison to river ground roughness → h [m] > 3 * k_S [m] → 2.2 m > 0.315 … 0.453 m → O.K.

*** further examples in dependence on channel cross-sectional and longitudinal structure:**

- see specialised literature → chapter 1.6
- calculations depending on many factors → high variety of calculation cases → overview see figure 1.26

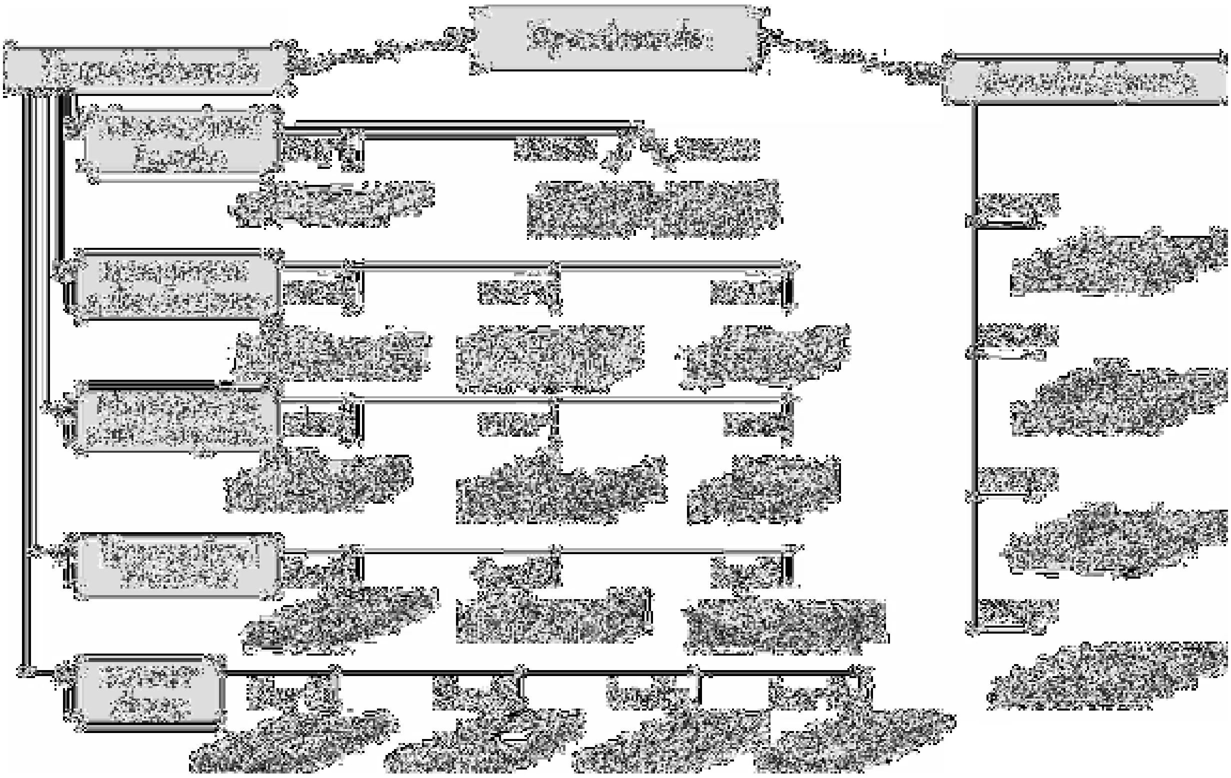


Fig. 1.26: Overview on calculation cases by means of Colebrook-White method (after LfU, 2002)

- practical application of Colebrook-White method → see seminar 14

1.6. Special literature regarding hydraulic investigations

Bollrich, G. (2007):
Technische Hydromechanik 1: Grundlagen. Verlag für Bauwesen.

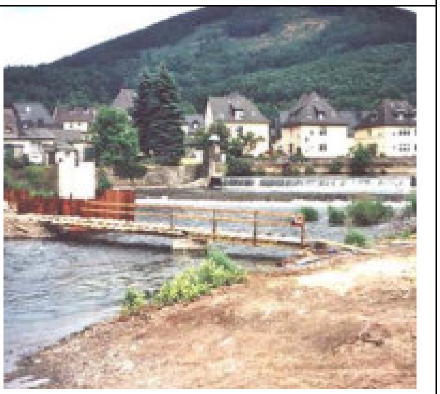
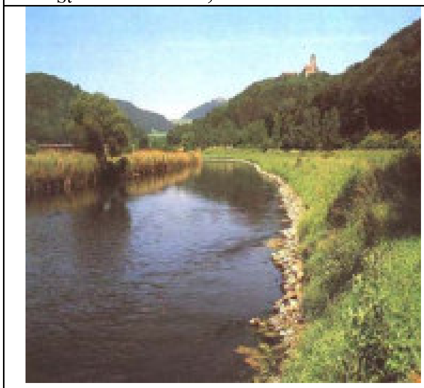
Landesanstalt für Umweltschutz Baden-Württemberg (1999):
Gewässergeometrie. Oberirdische Gewässer, Gewässerökologie 46. Kraft Druck und Verlag GmbH Ettlingen.

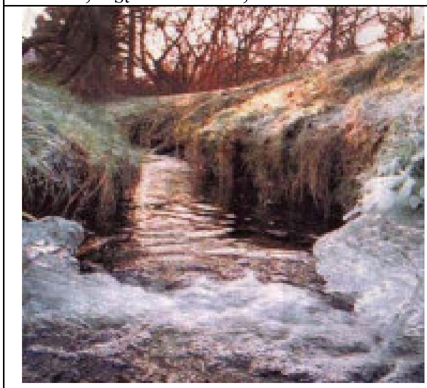
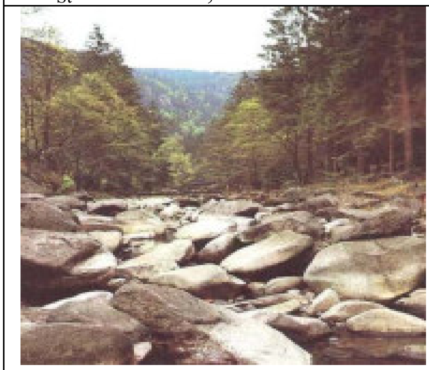
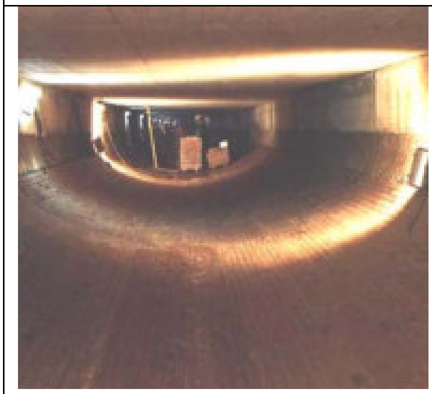
Landesanstalt für Umweltschutz Baden-Württemberg (2002):
Hydraulische Berechnung von Fließgewässern. Arbeitsanleitung Pegel- und Datendienst Baden-Württemberg.

Landesanstalt für Umweltschutz Baden-Württemberg (2002. 2003):
Hydraulik naturnaher Fließgewässer.
Teil 1 – Grundlagen und empirische hydraulische Berechnungsverfahren (2002)
Teil 2 – Neue Berechnungsverfahren für naturnahe Gewässerstrukturen (2002)
Teil 3 – Rauheits- und Widerstandswerte für Fließgewässer in Baden-Württemberg (2003)
Teil 4 – Numerische Modelle zur Strömungssimulation (2003)

Landesanstalt für Umweltschutz Baden-Württemberg (2002):
Vermessungsarbeiten im Pegelwesen. Arbeitsanleitung Pegel- und Datendienst Baden-Württemberg.

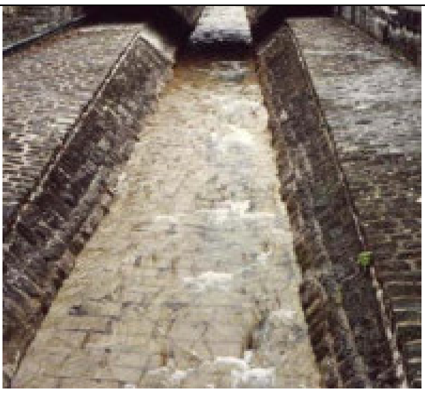
1.7. Illustrations for roughness coefficients

a) natural open channels:		
 River ground: fine sand, mud $k_{St} = 60\ m^{1/3}/s$, $n = 0.017\ s/m^{1/3}$	 River ground: sand, fine gravel $k_{St} = 55\ m^{1/3}/s$, $n = 0.018\ s/m^{1/3}$	 River ground: fine gravel $k_{St} = 50\ m^{1/3}/s$, $n = 0.020\ s/m^{1/3}$
 River: medium gravel $k_{St} = 40\ m^{1/3}/s$, $n = 0.025\ s/m^{1/3}$	 River ground: coarse gravel $k_{St} = 35\ m^{1/3}/s$, $n = 0.029\ s/m^{1/3}$	 Ground: loam, embankments: shrubs, $k_{St}=33\ m^{1/3}/s$, $n=0.030 s/m^{1/3}$

 Ground: loam, embankments: herbs, k_{St}=30m$^{1/3}$/s, n =0.033s/m$^{1/3}$	 Ground gravel, emb.: shrubs/trees k_{St} = 28 m$^{1/3}$/s, n = 0.036 s/m$^{1/3}$	 Ground: stones, emb.: shrubs/trees k_{St} = 25 m$^{1/3}$/s, n = 0.040 s/m$^{1/3}$
 Small irregular soil channels, embankments vegetated k_{St} = 23 m$^{1/3}$/s, n = 0.043 s/m$^{1/3}$	 Mountain streams with rocks and boulders, embankments vegetated k_{St} = 20 m$^{1/3}$/s, n = 0.050 s/m$^{1/3}$	 Mountain torrent with rocks and boulders, embankments vegetated k_{St} = 15 m$^{1/3}$/s, n = 0.067 s/m$^{1/3}$
 Ocean of boulders, emb. vegetated k_{St} = 10 m$^{1/3}$/s, n = 0.100 s/m$^{1/3}$		
b) artificial channels:		
 Steal, smooth grout, tiles k_{St} = 90 m$^{1/3}$/s, n = 0.011 s/m$^{1/3}$	 Smooth concrete k_{St} = 70 m$^{1/3}$/s, n = 0.014 s/m$^{1/3}$	 Brickwork k_{St} = 70 m$^{1/3}$/s, n = 0.014 s/m$^{1/3}$

Rough concrete

$k_{St} = 60\ m^{1/3}/s,\ n = 0.017\ s/m^{1/3}$



Pavement, regular

$k_{St} = 50\ m^{1/3}/s,\ n = 0.020\ s/m^{1/3}$

Concrete with joints, spare vegetated

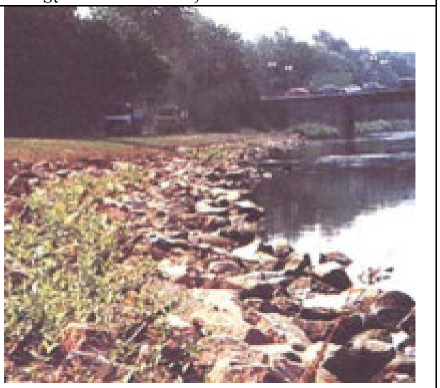
$k_{St} = 45\ m^{1/3}/s,\ n = 0.022\ s/m^{1/3}$

Small channel with sheet pilings

$k_{St} = 35\ m^{1/3}/s,\ n = 0.029\ s/m^{1/3}$

Grass pavers, gravel, stones

$k_{St} = 33\ m^{1/3}/s,\ n = 0.030\ s/m^{1/3}$



Stone embankment, coarse

$k_{St} = 25\ m^{1/3}/s,\ n = 0.040\ s/m^{1/3}$

Rough bottom slide (rock ramp)

$k_{St} = 15\ m^{1/3}/s,\ n = 0.067\ s/m^{1/3}$

c) Forelands, flooded in case of floods:

Short grass

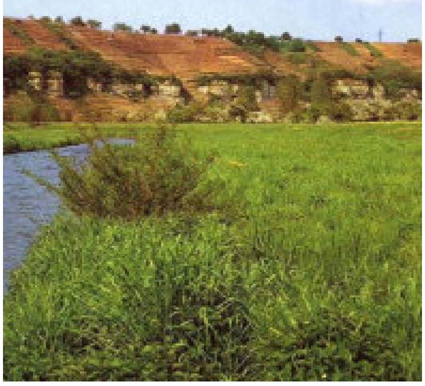
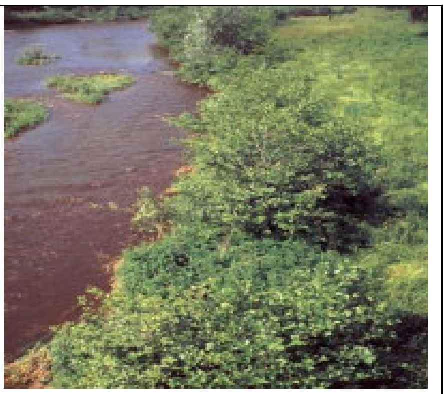
$k_{St} = 40\ m^{1/3}/s,\ n = 0.025\ s/m^{1/3}$

Dense grass vegetation

$k_{St} = 30\ m^{1/3}/s,\ n = 0.033\ s/m^{1/3}$

Forest floor

$k_{St} = 28\ m^{1/3}/s,\ n = 0.036\ s/m^{1/3}$

 Meadow, single trees $k_{St} = 25\ m^{1/3}/s$, $n = 0.040\ s/m^{1/3}$	 Grass, herbs and shrubs $k_{St} = 22\ m^{1/3}/s$, $n = 0.045\ s/m^{1/3}$	 Irregular foreland $k_{St} = 15\ m^{1/3}/s$, $n = 0.067\ s/m^{1/3}$
 Foreland with control structures $k_{St} = 12\ m^{1/3}/s$, $n = 0.083\ s/m^{1/3}$		

Primary picture sources: Anhäuser (1981), LfU (1996) und Schröder (2000)

Further picture sources: Darmstädter Echo, Döring, Keller, Knauf, Manuel, Stadtentwässerung München

2. Sediment transport in running waters

2.1. Definition and components of sediment transport

* **Sediment transport definition:**

- transport of particles in water, whose density is higher than waters density
- part of the sediment balance
- link between particle denudation (erosion) and deposition (sedimentation)

* **knowledge of sediment transport mechanism necessary for:**

- in cases where reservoir and lake silting up is possible
- knowledge of possible river bed changes (for example after renaturation)
- progressive river ground siltation → groundwater input reduction
- planning of engineering constructions (for example for flood protection)
- risk identification → for example danger of collapses of bridges or embankment breakups risks
- identification of risks regarding to sediment deposition in case of flood events
- wilful sediment transport initiation in connection with ecological orientated building operations at or in open channels:
  - → development of an own dynamics in renaturated channels
  - → formation of different flow conditions in open channels (rapids, death zones, ...)
  - → high ecologic functionality of open channel → high species diversity
- identification and quantification of sedimentation and erosion processes in open channels:
  - → for example in connection with river incision in case of absence of natural bed load because of impoundments (dams, barrages, weirs)
  - → resulting from extreme discharges (floods or low water)

* **sediment balance of open channels:**

- sediment balance processes in a catchment → see figure 2.1

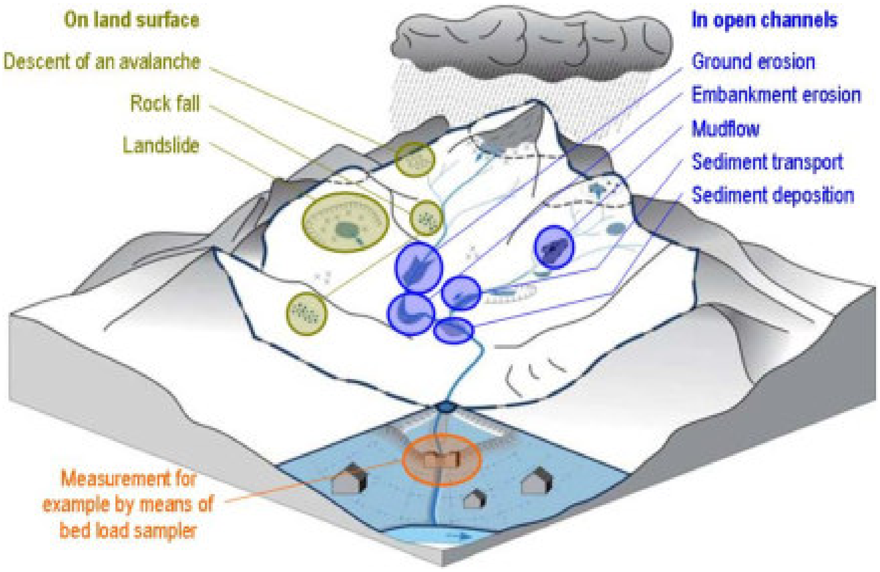


Fig. 2.1:

Sediment balance processes (after GRASSE ET AL., 2010)

- sediment transport components → see figure 2.2

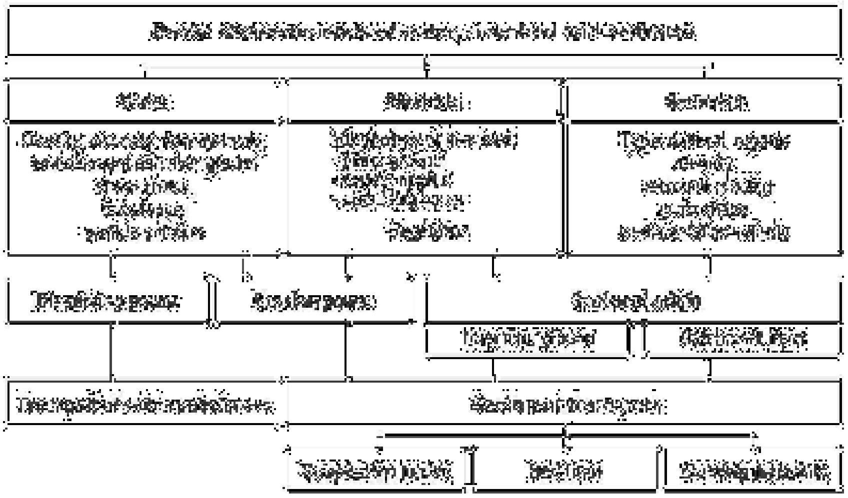


Fig. 2.2:

Main components of sediment transport (after LEHMANN ET AL., 2005, modified)

- comments to figure 2.2:
  - → Ice is no floating material.
  - → total sediment transport = suspended matter + bed load + floating materials (sometimes in narrower sense suspended matter and bed load only, in some literature solute substances included in total sediment transport)
  - → suspended matter in most cases the main part of sediment transport in lowlands
  - → bed load in most cases the main part of sediment transport in the mountains
- Definition suspended matter: solids, which are in static or dynamic equilibrium with water → floating by turbulence
- Definition bed load: solids, which are moved at river ground → sliding, rolling, in case of high ground slope skipping
- boundary between suspended matter and bed load:
  - → depending on grain size and flow velocity → see figure 2.3

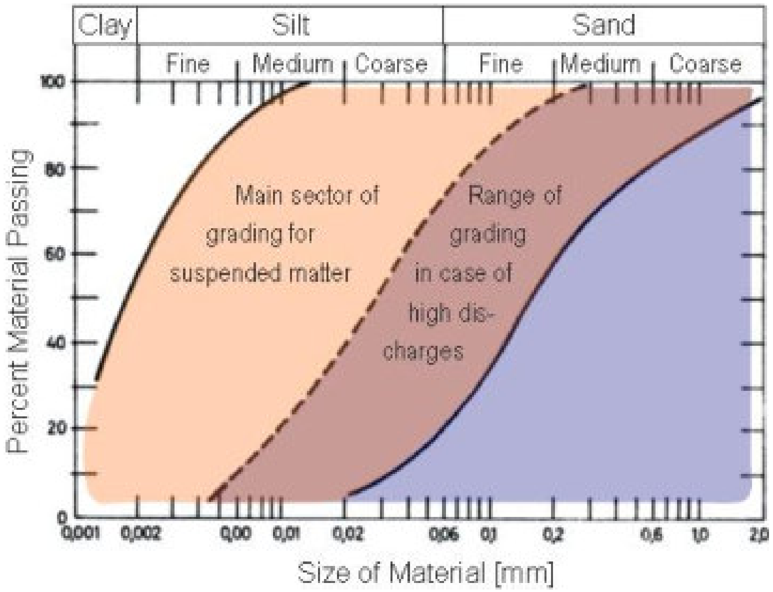


Fig. 2.3:

Dependence of the boundary between suspended matter and bed load (after MANIAK, 2005)

→ calculation of boundary between suspended matter and bed load by FROUDE number:

$$Fr^2 = \frac{v_{gr}^{\;2}}{g * d_{gr}} = 360 \;\rightarrow d_{gr} = 2.832 * 10^{-4}\, v_{gr}^{\;2} \tag{2.1}$$

where: Fr – FROUDE number [ ]
v_{gr} – boundary flow velocity [m/s]
g – gravitation constant [m/s^2]
d_{gr} – boundary grain diameter [m]

calculation example: $v_{FLOW} = 1.5$ m/s → $d_{gr} = 6.4 * 10^{-4}$ m = 0.64 mm

- percentage bed load / suspended matter dynamic in time and space:
  - → depending on hydrologic situation (low water, mean flow conditions, flood)
  - → tendential decrease of bed load amount at river longitudinal section → see figure 2.4

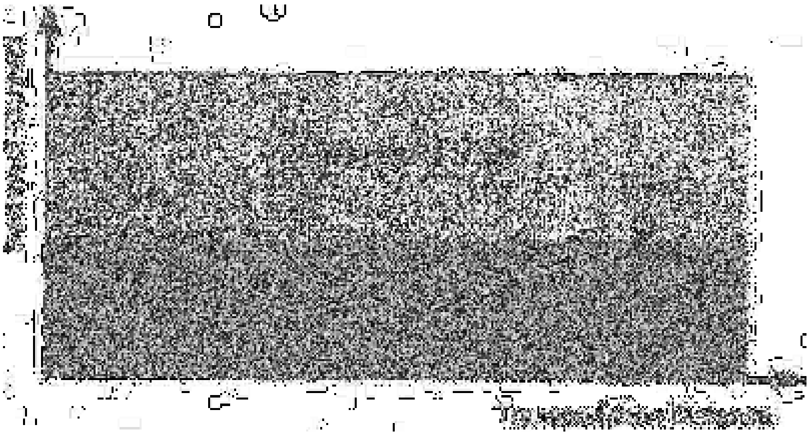


Fig. 2.4:

Amounts of suspended matter and bed load at river longitudinal section (after BEZZOLA, 2004)

2.2. Sediment transport measuring methods

2.2.1. Measuring of suspended matter

* **overview on methods:** → see figure 2.5

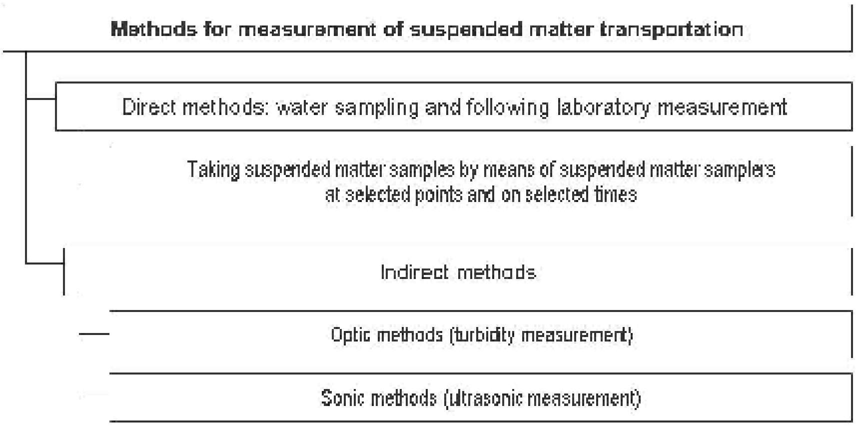


Fig. 2.5:

Overview on methods for measuring suspended matter

- advantages and disadvantages of direct measurement methods:
  → advantage: high accuracy
  → disadvantage: in most cases sophisticated, discontinuous measurement
- advantages and disadvantages of indirect measurement methods:
  → advantage: continuous measurement possible
  → disadvantage: many failure modes

→ conclusion: combined measurements (simultaneous application of direct and indirect methods)

* **direct measurement methods:**

- ***problem:*** irregular distribution of suspended matter concentrations in open channels in space and time → analogue to flow velocity in an open channel
- ***problem solution:*** suspended matter measurements at representative cross-sectional points analogue to flow velocity by current meter
- ***measurements at representative cross-sectional points (see figure 2.6):***

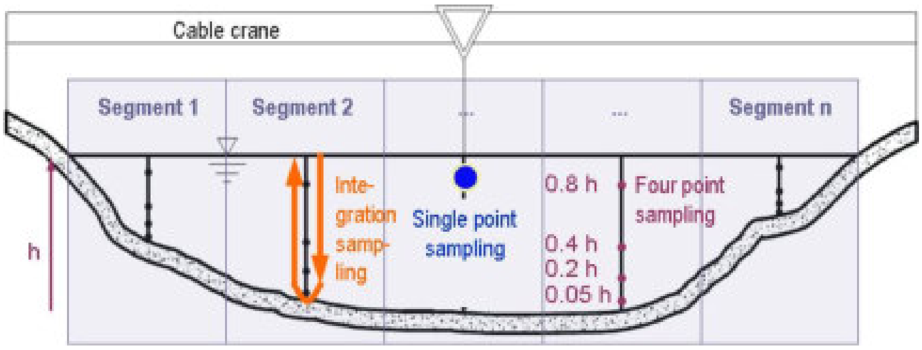


Fig. 2.6:

Suspended matter sampling at representative cross-sectional points

Single point sampling:
→ sample taking from midstream near water surface
→ simple, quickly realizable, but no high accuracy

Multipoint sampling:
→ subdivision of cross-section into several segments → measurement at the centre of each segment in several depth → methodology analogue to flow velocity measurement by means of current meter (see lecture notes “Hydrology I”, chapter 5.2.3)
→ mostly applied: four point sampling in 5 %, 20 %, 40 % and 80 % of water depth (percentage from river ground to the top)
→ parallel to it: measurements of flow velocity and discharge
→ very high accuracy but time-consuming

Integration sampling:
→ continuously sink of a sampler and following elevation (in case of bottle sampling the bottles should be nearly complete filled)
→ acceptable accuracy, not as time-consuming as multipoint sampling, but large number of failure modes → requires experience of the sampler
→ opportunities regarding to segment subdivisions: regarding to identical widths or identical discharges

- ***measurements at representative moments:***

→ depending on kind of open channel (lowland, mountains)
→ depending on hydrologic situation (low water, mean flow conditions, flood)
→ in case of mean flow conditions: suspended matter sampling at suspended matter measuring points without continuous data logging at least one time per day, in case of continuous data logging at least one or two times per week
→ during floods: higher frequency of sampling activities
→ in case of low water: frequency reduction of sampling to two or three samples per week

- ***samplers:***

Suspended matter bottles:
→ bucket: extraction of big samples for grain size analysis, possibly influence of flow and therefore suspended matter exsolution
→ sampler bottles: in most cases usage of devices with inflow and vent pipes (see figure 2.7) → precise measurement, integrated sampling possible

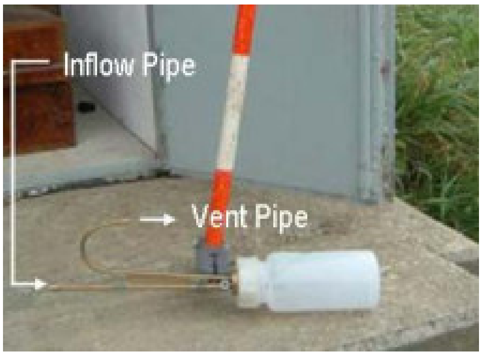


Fig. 2.7:

Sampler bottles with inflow and vent pipes (Photo: BOKU, 2008)

suspended matter samplers:
→ devices for sampling suspended matter point by point or integrative
→ sampling suspended matter point by point by means of valve timing → controlled water inflow into the sampler
→ filling time in case of integrated sampling can be determined before final measurement by try and error → main aim: avoidance of suspended matter overconcentration in the sample
→ at present most precise sampler: US-P61A see figure 2.8) → worldwide accepted and used, reference device after standard ISO 4363

a) NIELSEN-Sampler (producer: SEBA, Germany) b) US-P61 Suspended-Sediment-Sampler (USA)

Fig. 2.8: Examples for suspended mater samplers ***(Photos: Stimpf, Seitz)***

pump removal:

→ sampling by water pumping

→ temporal or integrated sampling possible

→ please note: limited suction head and no frost resistance!

method comparison → see table 2.1

Table 2.1: Comparison of methods for suspended matter sampling

	Bottles	Sampler	Pump removal
Time need for sampling	Low	High	Low
Maintenance effort	Low	Medium	Low
Usage in case of high flow velocities	Yes	US-P61: yes, else no	Yes
Sampling point by point	Not possible	Yes	Yes
Integrated sampling	In dependence on type	Yes	Yes
Isokinetic sampling*)	Not possible	US-P61: yes, else mostly no	In a small flow velocity range only

*) isokinetic sampling: inflow velocity into the sampler v_E = flow velocity v

- if $v_E > v$ → lower suspended matter concentration in the sample in comparison to real river concentration
- if $v_E < v$ → measurement of a suspended matter concentration which is too high

- ***laboratory investigations of suspended matter samples received by direct methods:***

methodology: filtration

→ pressure filtration (pressure of water through membrane filter)

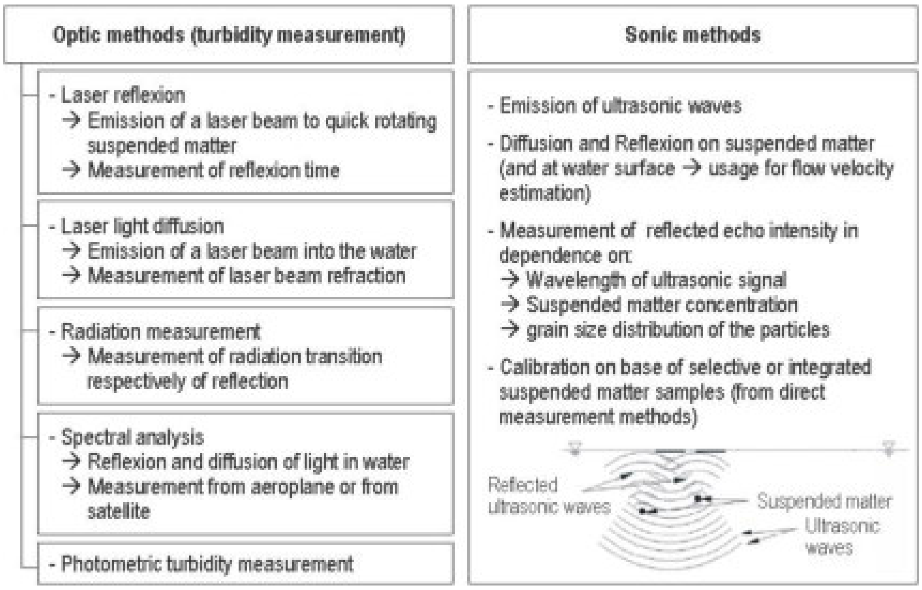
→ vacuum filtration (suction of water through membrane filter)

→ membrane filter drying to mass constancy

→ suspended matter concentration = dry weight of filtered substances / sample water volume

* **indirect measuring methods:**

- **overview on methods:** → see figure 2.9



Optic methods (turbidity measurement)

- Laser reflexion
  → Emission of a laser beam to quick rotating suspended matter
  → Measurement of reflexion time
- Laser light diffusion
  → Emission of a laser beam into the water
  → Measurement of laser beam refraction
- Radiation measurement
  → Measurement of radiation transition respectively of reflection
- Spectral analysis
  → Reflexion and diffusion of light in water
  → Measurement from aeroplane or from satellite
- Photometric turbidity measurement

Sonic methods

- Emission of ultrasonic waves
- Diffusion and Reflexion on suspended matter (and at water surface → usage for flow velocity estimation)
- Measurement of reflected echo intensity in dependence on:
  → Wavelength of ultrasonic signal
  → Suspended matter concentration
  → grain size distribution of the particles
- Calibration on base of selective or integrated suspended matter samples (from direct measurement methods)

Fig. 2.9:

Indirect methods for suspended matter estimation – overview on methods

- at present mostly used: photometric turbidity measurements

  → Measurement option 1: emission of visible or infrared light from one side of the river (emitter) and measurement of transition on the opposite river side (receiver) → the higher the suspended matter concentration the higher light dispersion (lower value of light transition at receiver)

  → Measurement option 2 (at present primary used): emission of visible or infrared light into the water from the water surface and measurement of the in the emitters direction reflected light → concentric positioning of photodiodes around the emitter, measurement of reflected light intensity as an analogue quantity of suspended matter concentration in the water

  → regularly calibration necessary because:
    - lower accuracy in comparison to direct measurement methods
    - in case of stationary devices: light translucence decrease at turbidity measuring probe because of „biofouling“ (algae growth at sensor)

2.2.2. Measuring of bed load transport

* **overview on measurement methods:** → see figure 2.10

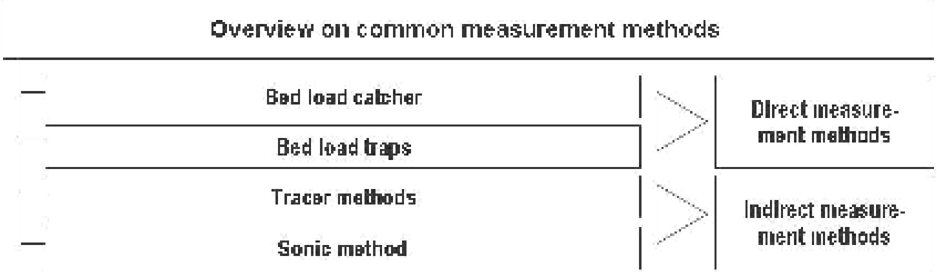


Fig 2.10:

Overview on measurement methods of bed load transport

* **bed load catcher:**

- construction → see figure 2.11
- catcher open in water flow direction
- let down by cable windlass into the water body
- usage in case of sandy or gravelly river ground
- catcher mouth flexible adjustable → precise adaption to river ground → adjustment control by underwater camera
- because of specific construction → minimization of flow disturbances → nevertheless: increase of inflow velocity because of funnel-shaped widening

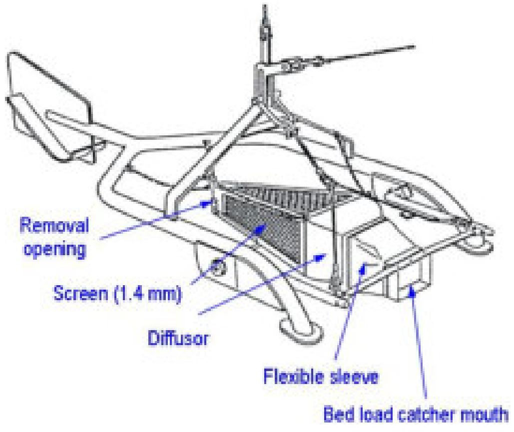


Fig. 2.11:

Bed load catcher of the German Federal Institute of Hydrology (after DVWK, 1992)

* **bed load traps:**

- installation of bed load traps in the river ground preferably over the hole cross-section
- mostly permanent installations (concrete), sometimes mobile installations (high-grade steel)
- measurement of bed load weight or bed load volume by regular emptying respectively load cells
- in case of load cells → emptying of collecting container, before load cell is complete full

* **tracer methods:**

- usage of coloured stones (different colours for different stone sizes, coloured stone distribution analogue to river bed stone size distribution)
- usage of natural external bed load of the same size and density that colour differs
- preparation of stones with iron → measuring by induction coils
- preparation of stones with micro-transmitters → transponder
- cross-sectional colour coding

* **sonic method:**

- usage of underwater microphones (hydrophones)
- two measurement methods:
  - → sound recording caused by bed load movements
  - → sound recording which occur in case of bed load collision on a metal plate
- calibration by means of bed load catcher or bed load traps necessary

* **further indirect measurement methods:**

- analysis of excavated bed load volumes from a artificial reservoir → long-time measurement data necessary
- delta measurements in river mouths (in case of large rivers):
  - → estimation of sediment volume
  - → estimation of grain size distribution by drilling, sediment taking and laboratory methods

* **comparison of methods:**

- advantages and disadvantages of the methods → see table 2.2

Table 2.2: Comparison of methods for bed load registration

	Direct measurement methods	Indirect measurement methods
Advantages	- Precise estimation of bed load quantity - Option of analysing bed load samples (for example grain size distribution)	- Especially suitable for investigations at begin of bed load movement - Sonic method suitable also for flow velocities > 2 m/s
Disadvantages	- Bed load traps: high expense concerning construction, maintenance (emptying) and measuring - Bed load catcher: only partly or not suitable for grain diameter > 32 mm and for flow velocities > 2 m/s	- Bed load values estimation only after calibration (by means of direct methods) possible

- conclusion: An universal method at present does not exist!

2.3. Sediment transport calculation methods

2.3.1. Problems in connection with sediment transport calculation

* **linkage of hydraulics and sediment transport:** → see figure 2.12

Fig. 2.12:

Dependence of sediment transport on river hydraulics

* **parameter which are describing sediment transport in open channels:**

- **appearance of river ground:** plane or lumpy (ripple, dunes, sandbanks, potholes)
- **physical properties of river ground:** sediment density, grading, grain shape, settling velocity
- **physical properties of water:** density, kinematic viscosity
- **longitudinal and cross sections:** curvatures, cross-sectional and slope changes
- **river hydraulics:** bottom shear stress, special flow velocity distribution, cross-sectional turbulence distribution

→ large number of parameter, partly difficult determinable

* **assumptions and simplifications for practically orientated calculations:**

- **flow:** approximately steady-state, uniform, streaming (not shooting)
- **cross-section:** compacted (not segmented) channel, occasional changes in cross-sectional figuration
- **river ground:** bed material more or less uniform and not compacted (without cohesiveness), no vegetation
- **meandering:** straight pathway without heavy curvature
- **sediment:** no organic matter, not compacted (without cohesiveness), no consideration of river inputs by erosion
- **local effects:** unconsidered → for example secondary flow at piers, in potholes, ...

2.3.2. Estimation of sediment transport beginning

* **theoretical basics:**

- sediment transport beginning = f (grain size) → grain size dependent critical state
- critical state estimation → one of the biggest problems in sediment hydraulics
- characteristics of critical state:
  → begin of sediment movement at river ground
  → begin of sediment swirls

- parameter for quantification of transport beginning:
  - → critical flow velocity
  - → critical bottom shear stress
  - → critical water depth
  - → critical slope
  - → critical discharge

* **transport beginning at critical flow velocity:**

- estimates of critical flow velocities for a rough assessment → see table 2.3

Table 2.3: Estimates of critical flow velocities after German Industry Standard DIN 19661/2

Texture of river ground		$v_{m,kr}$ [m/s]
River bed material not colloidal (single grain structure)	Fine sand, grain size 0.063 – 0.2 mm	0.20 ... 0.35
	Medium sand, grain size 0.2 – 0.63 mm	0.35 ... 0.45
	Coarse sand, grain size 0.63 – 2.0 mm	0.45 ... 0.60
	Fine gravel, grain size 2.0 – 6.3 mm	0.60 ... 0.80
	Medium gravel, grain size 6.3 – 20.0 mm	0.80 ... 1.25
	Coarse gravel, grain size 20.0 – 63.0 mm	1.25 ... 1.60
	Stones, grain size 63.0 – 100.0 mm	1.60 ... 2.00
River bed material colloidal	Silt, low bulk density	0.10 ... 0.15
	Loam, low bulk density	0.15 ... 0.20
	Sandy loam, high bulk density	0.40 ... 0.60
	Loam, high bulk density	0.70 ... 1.00
	Loam, very high bulk density	0.90 ... 1.30
Grass overgrown	Grass, enduring overflowed	1.50
	Grass, temporary overflowed	2.00

- quantification of relationship between critical flow velocity and grain diameter → diagram after HJULSRÖM (1935) → see figure 2.13 (estimated empirically)
- not valid for cohesive sediments (sand, gravel) with grain diameter > 0.1 mm
- valid for near plane river bed only

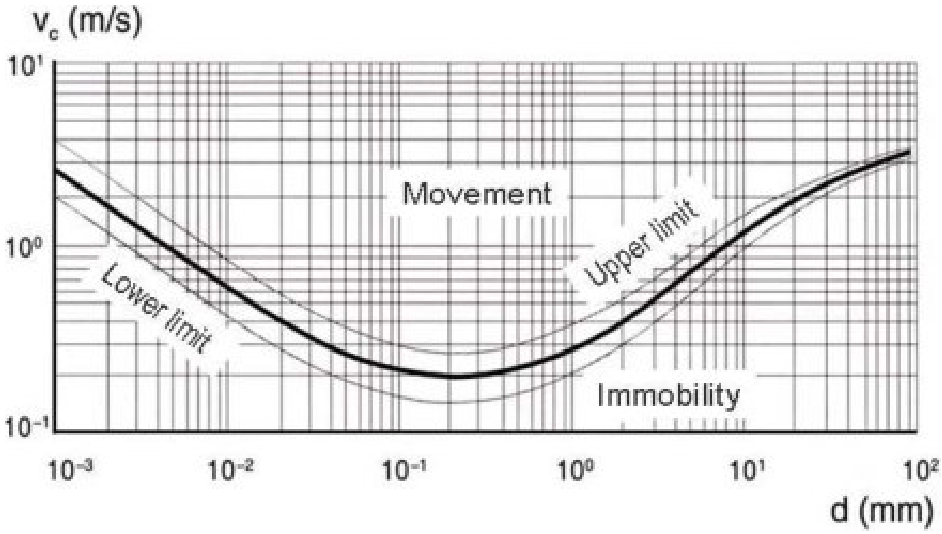


Fig. 2.13:

HJULSRÖM diagram (after DVWK, 1992)

- further quantifications → empirical equations

  For example: estimation after ZANKE (1982):

$$v_{m,kr} = 2.8 \sqrt{\rho' g d + 14.7 \frac{\upsilon}{d} c_a} \quad \text{with } \rho' = \frac{\rho_S - \rho}{\rho} \tag{2.2}$$

where: $v_{m,kr}$ - critical mean flow velocity [m/s]
ρ' - relative sediment density [–]
g - gravitation constant [m/s^2]
d - mean grain diameter of river bed material [m]
υ - viscosity of water [m^2/s]
c_a - adhesion (for natural sands $c_a = 1$)
ρ_S - sediment density [kg/m^3], in most cases 2 600 – 2 700 kg/m^3
ρ - water density of open channel [kg/m^3], mostly 1 000 kg/m^3

→ equation 2.2 is valid for water depth between 0.7 and 2 m (mean app. 1.4 m)

→ for greater depth → multiplication by factor f:

$$f = \left[\frac{h}{1.4} \right]^{1/6} \tag{2.3}$$

where: h - water depth [m]

→ increasing critical flow velocity for increasing water depth

* **transport beginning at critical bottom shear stress:**

- flow deceleration because of friction at river boundaries (river ground, embankments)
- value of friction force per unit of area is called bottom shear stress → for illustration → see figure 2.14

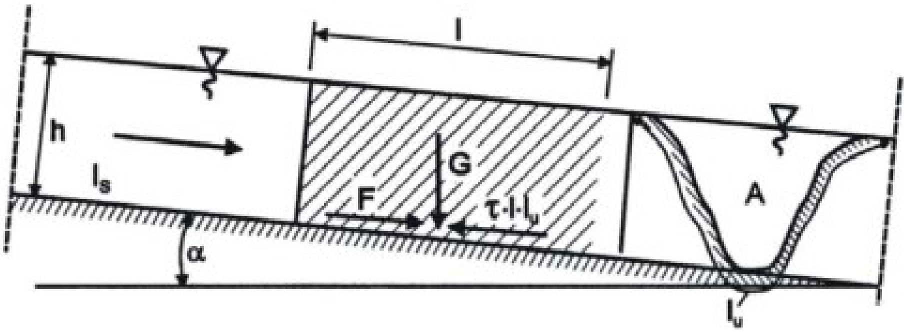


Fig. 2.14:

Definition of bottom shear stress (PATT, 2009)

- further usual expression: actual shear stress
- further usual expression: actual shear stress → see table 2.4

Table 2.4: Estimates of critical bottom shear stresses after German Industry Standard DIN 19661/2

Texture of river ground		τ_{Kr} [N/m²]
River bed material not colloidal (single grain structure)	Fine sand, grain size 0.063 – 0.2 mm	1.0
	Medium sand, grain size 0.2 – 0.63 mm	2.0
	Coarse sand, grain size 0.63 – 2.0 mm	3.0
	Fine gravel, grain size 2.0 – 6.3 mm	3.0 – 6.0
	Gravel-sand-mixture, enduring overflowed	9.0
	Gravel-sand-mixture, temporary overflowed	12.0
	Medium gravel, grain size 6.3 – 20.0 mm	15.0
	Coarse, grain size 20.0 – 63.0 mm	45.0
	Oblately bed load, 1 – 2 cm high, 4 – 6 cm long	50.0
River bed material little colloidal	Sand, low bulk density, loamy sediment, mud	2.0 – 2.5
	Loamy gravel, enduring / temporary overflowed	15.0 / 20.0
River bed material hardly colloidal	Loam, low bulk density	3.5
	Loam, high bulk density respectively mud, clay	12.0
Grass overgrown	Grass, enduring overflowed	15.0
	Grass, temporary overflowed	30.0

- methodology: calculation of real bottom shear stress and comparison with critical bottom shear stress:
  → immobility (no sediment transport): real bottom shear stress τ < critical bottom shear stress τ_{kr}
  → begin of sediment transport: $\tau \approx \tau_{kr}$
  → sediment transport: $\tau > \tau_{kr}$

- basis for real bottom shear stress calculation τ

$$F = G \cdot \sin\alpha \approx G \cdot I_S = l \cdot A \cdot \rho \cdot g \cdot I_S \qquad (2.4)$$

where: F - weight force in flow direction [N]
G - weight force of water body [N]
α - river ground slope [°] (for $\alpha \rightarrow 0$: $\alpha \approx I_S$)
ρ - water density [kg/m³]
g - gravitation constant [m/s²]
I_S - river ground slope [m/m]
A - cross-sectional area [m²]
l - length of water body [m]

Because of equilibrium condition $F = \tau \cdot l \cdot l_u$ (see figure 2.14) results (formula after DU BOYS, 1879):

$$\tau = \rho \cdot g \cdot r_{hy} \cdot I_S \qquad (2.5)$$

where: τ - real bottom shear stress [N/m²]
ρ - water density [kg/m³]
g - gravitation constant [m/s²]
r_{hy} - hydraulic radius [m] = A / l_u (definition → see chapter 1.3.1
I_S - river ground slope [m/m]

Improvement of Du Boys formula by Meyer-Peter and Müller (1949) → reduction to transport efficient bottom shear stress (transport efficient: consideration of the fact that in case of small rivers a big part of shear stress is caused by river embankments and not by river ground):

$$\tau = \rho \cdot g \cdot r_{hy} \cdot I \left(\frac{k_{St}}{k_r} \right)^{3/2} \tag{2.6}$$

where: k_{St} - Strickler coefficient [$m^{1/3}/s$]
k_r - grain roughness [m]
(all other symbols → see equation 2.5)

at what:

$$k_r = \frac{26}{d_m^{\ 1/6}}$$

where: k_r - grain roughness [m]
d_m - mean grain diameter [m]

standard values: $k_{St}/k_r = 1.0$ for plane river bed
$k_{St}/k_r = 0.5$ for high bed waves (for example dunes)

- methods for critical bottom shear stress estimation τ_{kr}:

a) tabular estimation by means of German Industry Standard Din 19661/2 (values → see table 2.4) → for rough estimation

b) graphical estimation by means of German Industry Standard Din 19661/2 (see figure 2.15) → for cohesive river bed materials

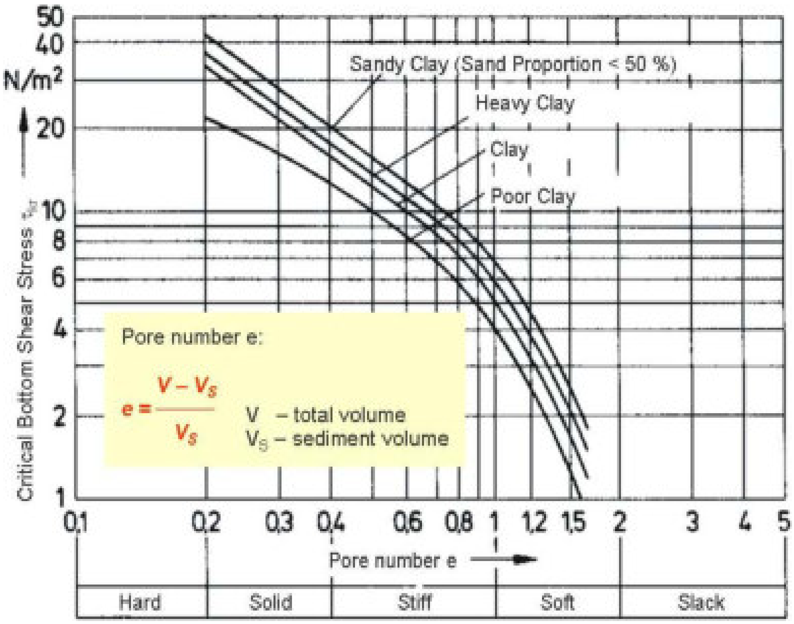


Fig. 2.15:

Critical bottom shear stress in dependence on pore number after German Industry Standard Din 19661/2

c) by precise calculation (modelling) → high number of models in dependence on boundary conditions

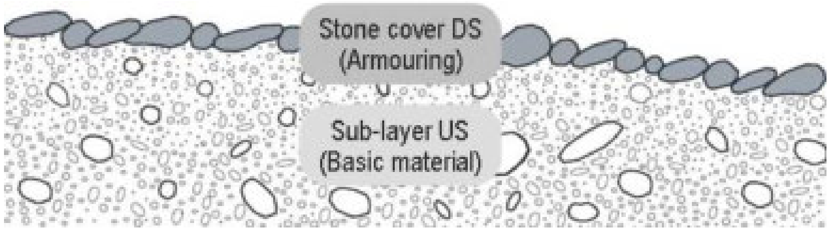
- calculation methods for critical bottom shear stress τ_{kr}:

  calculation method after SCHIELDS (1936):

$$\tau_{kr} = 0.06\,(\rho_S - \rho) \cdot g \cdot d_{50} \qquad (2.7)$$

where: τ_{kr} - critical bottom shear stress [N/m^2]
ρ_S - sediment density [kg/m^3] (in most cases 2 600 – 2 700 kg/m^3)
ρ - water density [kg/m^3]
g - gravitation constant [m/s^2]
d_{50} - grain size at 50 % material passing [m]

→ is valid for non-cohesive river ground sediments and uniform materials
→ is valid for river grounds without stone cover (for illustration → see figure 2.16)



$d_{m\,DS} \approx d_{90\,US}$

Fig. 2.16:

Stone cover and sub-layer at river ground (after BEZZOLA, 2004)

calculation method after MEYER-PETER and MÜLLER (1949) for non-cohesive and non-uniform river sediments without stone cover:

$$\tau_{kr} = 0.047\,(\rho_S - \rho) \cdot g \cdot d_m \qquad (2.8)$$

where: d_m - mean grain diameter [m]
(all other symbols → see equation 2.7)

calculation method after MEYER-PETER and MÜLLER (1949) for non-cohesive and non-uniform river sediments with stone cover:

$$\tau_{kr} = 0.045\,(\rho_S - \rho) \cdot g \cdot d_{90} \qquad (2.9)$$

where: d_{90} - grain diameter at 90 % material passing [m]
(all other symbols → see equation 2.7)

calculation method after GRAF, SUSZKA (1991) for steep river bed slope:

$$\tau_{kr} = 0.042 \cdot 10^{2.2I}\,(\rho_S - \rho) \cdot g \cdot d$$

where: I - river ground slope [m/m]
(all other symbols → see equation 2.7)

- comparison of the calculated real bottom shear stress τ with the critical bottom shear stress τ_{kr} → statement, if sediment transport occurs or not

2.3.3. Calculation of sediment transport amounts

- **overview on methods:**
  - application of empirical transport formulae respectively of bed load transport models
  - general dependences of sediment transport amounts:
    - → proportionality to flow velocity
    - → proportionality to bottom shear stress
    - → proportionality to river discharge

- **empirical transport formulae:**
  - sediment transport formula after MEYER-PETER and MÜLLER (1949):

related to mass:

$$m_T = \frac{8}{\rho^{1/2}\,(1-\rho/\rho_S)\cdot g}\,(\tau - \tau_{kr})^{3/2} \tag{2.10}$$

where: m_T - sediment transport per river bed metre (at cross-section) [kg/(m s)]
τ - real bottom shear stress [N/m^2]
τ_{kr} - critical bottom shear stress [N/m^2]
ρ_S - sediment density [kg/m^3] (in most cases 2 600 – 2 700 kg/m^3)
ρ - water density [kg/m^3]
g - gravitation constant [m/s^2]

related to volume:

$$m_T = \frac{8}{(\rho_S-\rho)\cdot g}\,(\tau - \tau_{kr})^{3/2} \tag{2.11}$$

where: m_T - sediment transport per river bed metre (at cross-section) [m^3/(m s)]
((all other symbols → see equation 2.10)

→ applicable for sand and gravel river beds

→ sediment transport amount m_G [kg/s or m^3/s] is calculable by integration of sediment transport per river bed metre m_T over river bed width b_s:

$$m_G = \int_0^{b_S} m_T\,db \tag{2.12}$$

where: m_G - sediment transport amount [kg/s or m^3/s]
m_T - sediment transport per river bed metre (at cross-section) [kg/(m s) or m^3/(m s)]
b_S - river bed width [m]

- sediment transport formula after SMART and JÄGGI (1983):

$$m_G = \frac{4\,\rho_S}{[(\rho_S/\rho)-1]} \left[\frac{d_{90}}{d_{30}}\right]^{0.2} \cdot I^{1.6} \cdot r_{hy} \cdot v_m \cdot b \left[1 - \frac{\tau_{kr}}{\tau}\right] \qquad (2.13)$$

mit:
m_G - sediment transport amount [kg/s]
τ - real bottom shear stress [N/m^2]
τ_{kr} - critical bottom shear stress [N/m^2]
ρ_S - sediment density [kg/m^3] (in most cases 2 600 – 2 700 kg/m^3)
ρ - water density [kg/m^3]
d_{90} - grain size at 90 % material passing [m] (d_{30} analogue)
I - river ground slope [m/m]
B - river width [m]
r_{hy} - hydraulic radius [m$^{1/3}$/s]
v_m - mean flow velocity [m/s]

→ application for mountain creeks with steep slope (I = 0.5 – 20 %)
→ requires the estimation of flow velocity and discharge

* **sediment transport models:**

- at present mostly applied models: models with empirical equations but physically determined parameters
- examples:
  → Mike 21C from Danish Hydraulic Institute (DHI)
  → Delft2D from WL Delft Hydraulics (principal model structure → see figure 2.17)
  → Delft3D from WL Delft Hydraulics
- modelled processes: hydraulics, sediment transport, river geometry

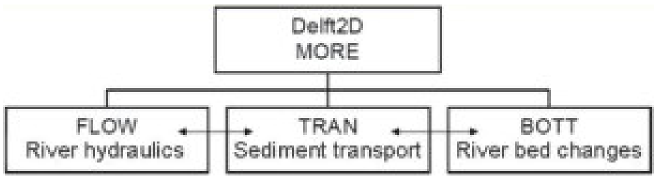


Fig. 2.17:

Delf2D principal model structure (after MOHN, 2001)

* **major assumptions of calculation methods** (concerns empirical transport formulae and sediment transport models equally)**:**

- validity for non-cohesive and relatively uniform sediments → gravel, sand
- streaming flow (no shooting)
- near steady-state uniform flow conditions
- compacted channel cross-section (in case of segmented cross-sections → subdivision into compacted sub-cross-sections → separate consideration of sub-cross-sections
- no aquatic plants, no biofilm at river bed
- no river bed consolidation by chemical processes
- *essential conclusions:*
  → application fields of calculation methods in comparison to river variety limited
  → additional measurement of sediment amounts over a long period

2.3.4. Estimation of sediment freights and sediment balances

* **definitions:**

- **sediment freights** = within a given period (month, year, flood event) transported sediment amount → description of sediment transport ability of a longitudinal river segment under consideration of discharge behaviour
- **sediment balances** = difference of sediment transport and sediment removal related to observation period

* **methodology related to sediment transport freights:**

- continuous estimation of sediment transport amounts in dependence on discharge by means of empirical formulae or sediment transport models, if applicable by measurements) → sediment transport hydrograph (for example: see figure 2.18)

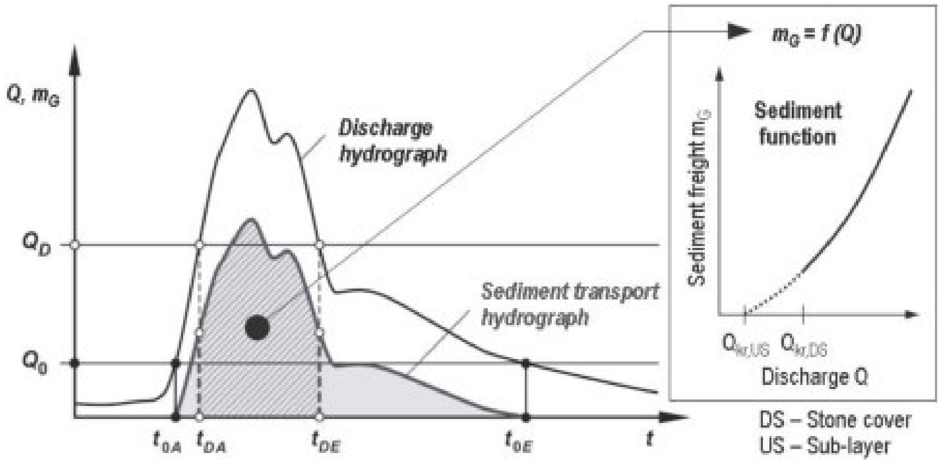


Fig. 2.18:

Sediment transport hydrograph in dependence on sediment function (expanded after Bezzola, 2004)

- estimation of sediment frights by discharge and sediment distributions over a given period (for example → see figure 2.19)

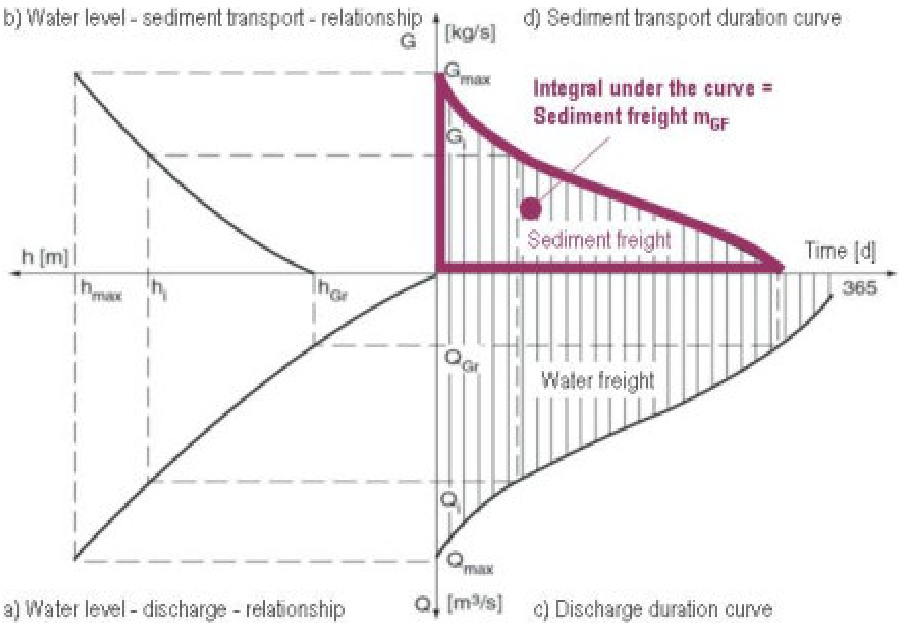


Fig. 2.19:

Estimation of sediment frights by discharge and sediment distributions (after Patt, Gonsowski,, 2011)

- **sediment freights:**
  - conclusions regarding to river bed erosion or sedimentation at considered river segment → see figure 2.20
  - conclusions regarding changes in longitudinal section and cross-section → see figure 2.21

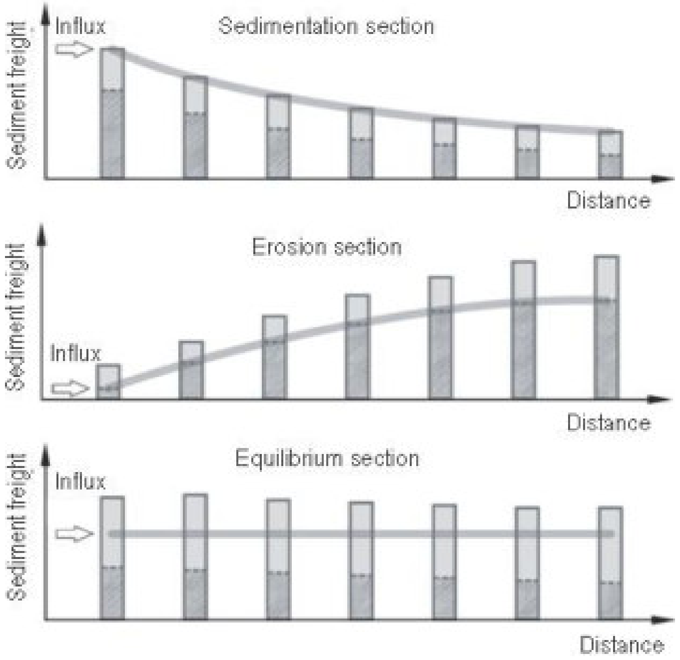


Fig. 2.20:

Sediment freights in case of sedimentation and erosion processes (after B*EZZOLA, 2004)*

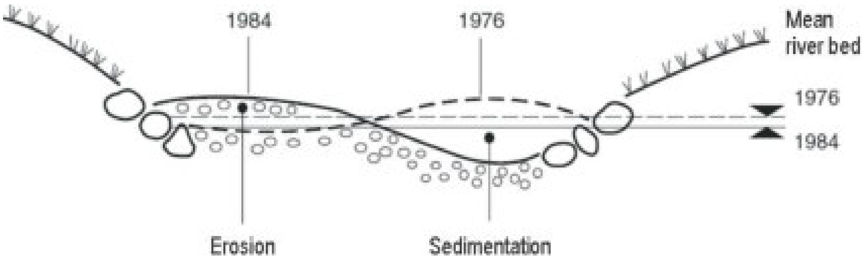


Fig. 2.21:

Cross-sectional changes in result of sedimentation and erosion processes (after P*ATT,* G*ONSOWSKI,, 2011)*

2.4. Typology of running waters

- **definition and characteristics:**
  - **stream typology** = typecast of channels regarding water way structures, which are depending on sediment freights
  - stream typology characteristics → see figure 2.22

- **characterisation of abiotic factors;**
  - longitudinal section, sediment balance and meander degree → see figures 2.23 – 2.25
  - further morphologic parameters → see lecture notes “Hydrology I”, chapter 5.6.1

- **characterisation of biotic features:**
  - see course “Limnology”

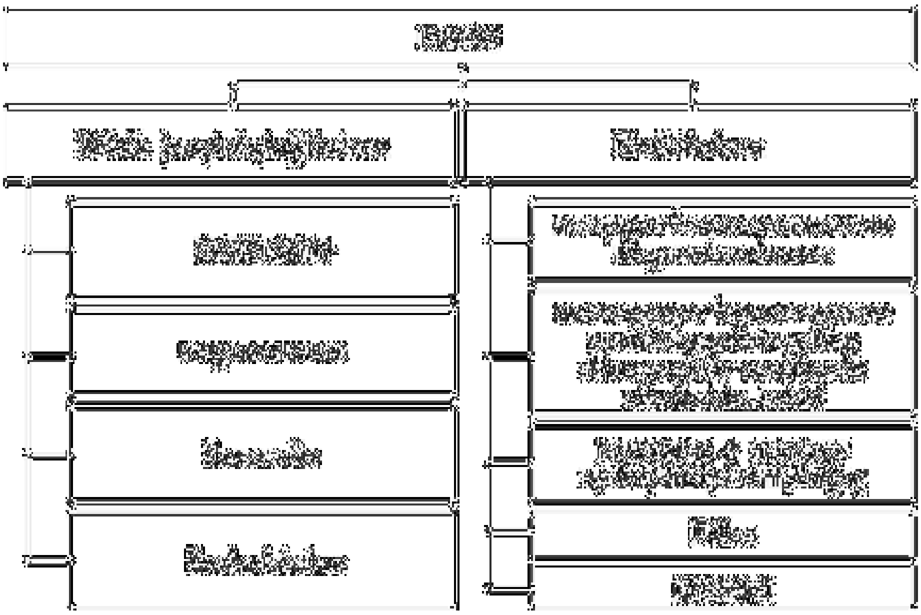


Fig. 2.22:

Factors of stream typology

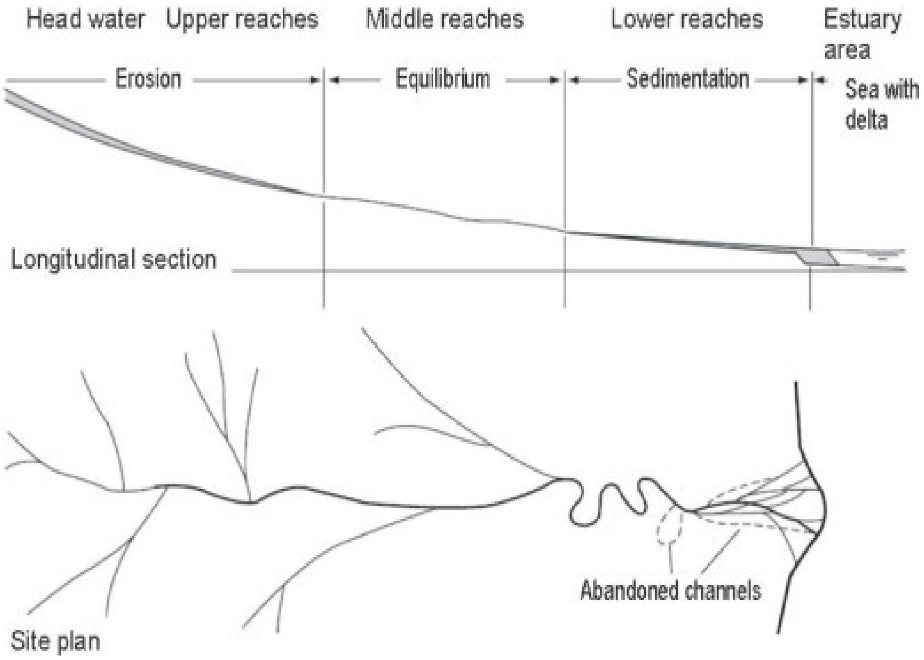


Fig. 2.23:

Sedimentation and erosion along river longitudinal section (after P*ATT*, G*ONSOWSKI*, *2011)*

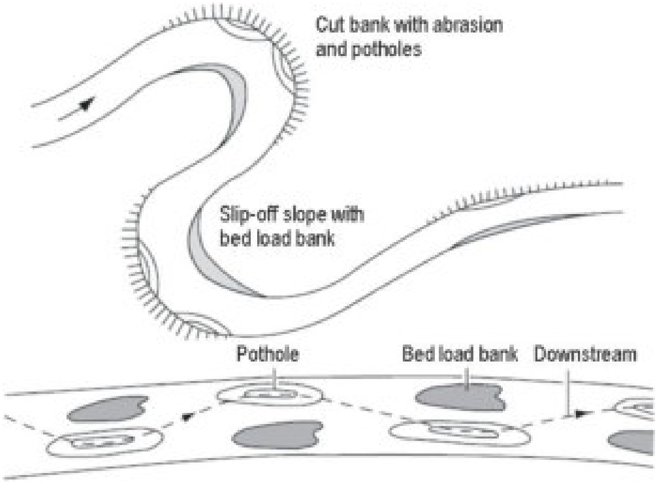


Fig. 2.24:

Sediment caused processes in correlation with meandering (after P*ATT*, G*ONSOWSKI*, *2011)*

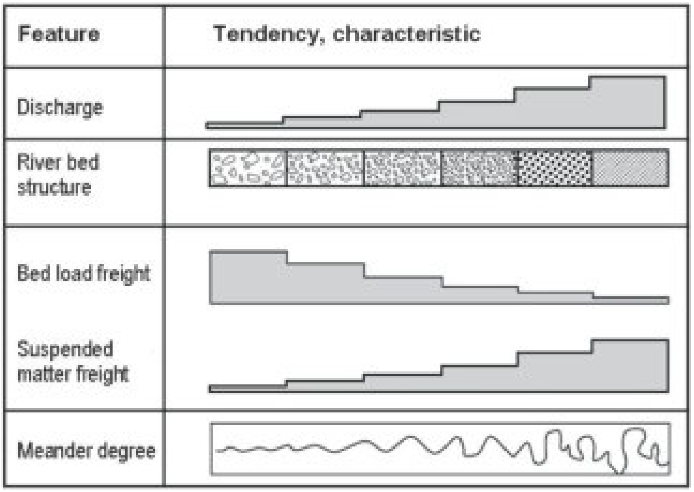


Fig. 2.25:

Changes of abiotic factors within longitudinal section (after P*ATT,* G*ONSOWSKI, 2011)*

2.5. Special literature for sediment transport

Kraus, W., H. Patt und H. Jürging (2011):
Naturnaher Wasserbau. Entwicklung und Gestaltung von Fließgewässern. 4. aktualisierte Auflage, Springer-Verlag Berlin, Heidelberg.

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Lehmann, B., H.-H. Bernhart und F. Nestmann (2005):
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ALPRESERV - Sustainable Sediment Management in Alpine Reservoirs considering ecological and economical aspects. Sedimentquellen und Transportprozesse. Print: Universität der Bundeswehr München (Germany).

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Gewässercharakter, Flussmorphologie und Geschiebetransport. Nachdiplomkurs in den angewandten Erdwissenschaften. Versuchsanstalt für Wasserbau, Hydrologie und Glaziologie, ETH Zürich.

Habersack, H., M. Haimann, W. Kerschbaumsteiner und P.Lalk (2008):
Schwebstoffe im Fließgewässer - Leitfaden zur Erfassung des Schwebstofftransportes. Österreichisches Bundesministerium für Land- und Forstwirtschaft, Umwelt und Wasserwirtschaft. Druck: AV+Astoria Druckzentrum GmbH Wien.

3. Ice formation in streams

3.1. Necessity of ice formation investigations in flowing waters and process description

* **hazards because of ice formation and ice breakup:**

- hazard of water management buildings (dikes, weirs, lockage, bridges, ...)
  - → pressure
  - → mechanic power → because of water level variations in case of closed icecap
    → because of ice floes, which are transported with flux
- narrowing of river cross-section
  - → reduced discharge capacity
  - → increase of flow velocity residual flow profile
- other matters:
  - → prevention of ship traffic
  - → hazards in case of entering ice covers

→ necessity of ice investigations

* **process description of ice formation in open channels:**

- ice types and influencing factors on ice formation on open channels → see figure 3.1

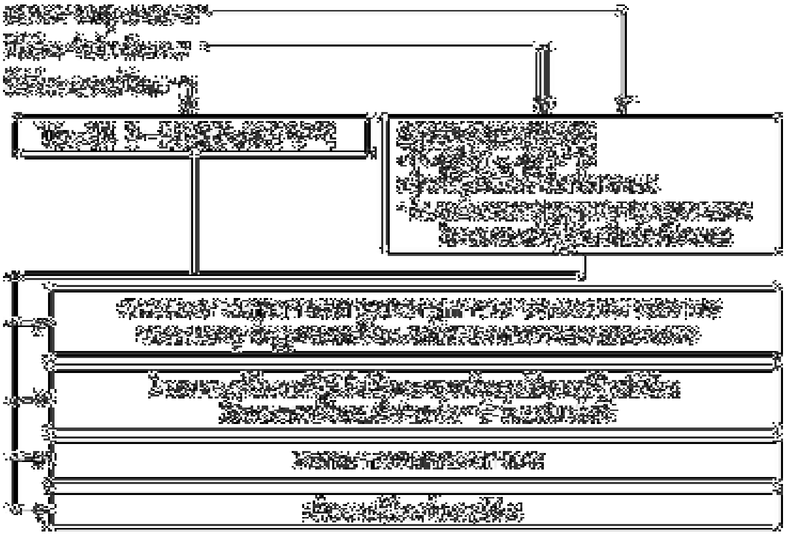


Fig. 3.1:

Ice types and influencing factors on ice formation on open channels

- processes of icecap growth in open channels → see figure 3.2

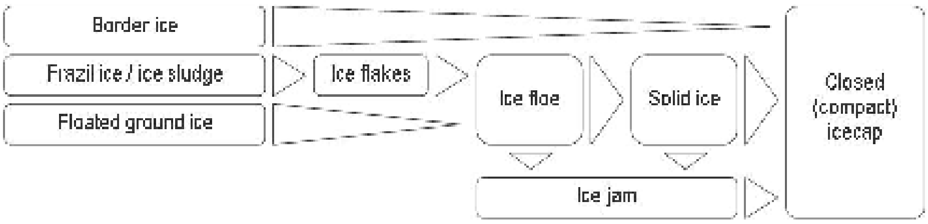
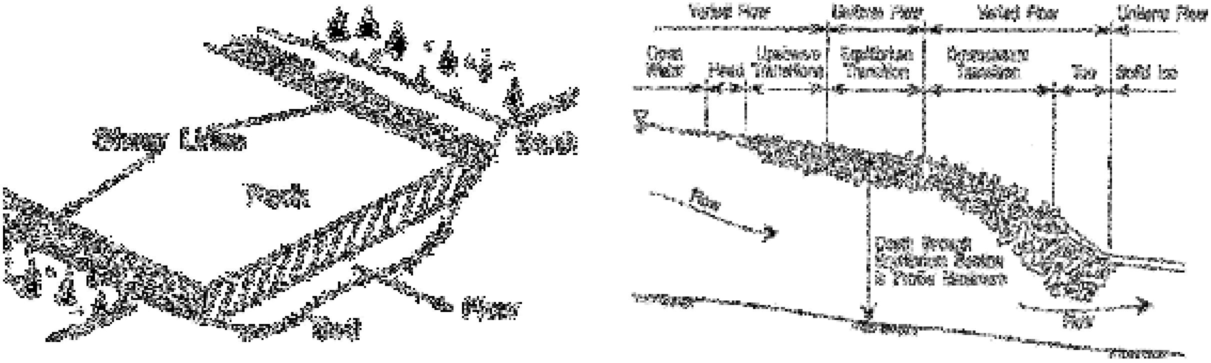


Fig. 3.2:

Processes of icecap growth in open channels

- dangerous conditions: ice floe, ice jam and in some cases compact icecap

- consequences of compact ice cover formation:
  - → reduction of discharge cross-section → see figure 3.3
  - → reduction of discharge capacity → consequences of solid ice on discharge exemplary for gauge Neu Darchau river Elbe) in January 1979 → see figure 3.4
  - → considering continuity equation → higher flow velocities or higher water levels (in reality in most cases lower discharges but distinct higher water levels)



a) Growth of a closed ice cover b) Flow condition in case of ice jam

Fig. 3.3: ***Options for discharge profile reduction (MAIDMENT, 1993)***

Fig 3.4:

Influences of a compact icecap on water level and discharge during an ice period at gauge Neu Darchau in January 1979 (after WSV)

- frequency of river icing occurrence in Germany:
  - → ice formation in all great rivers in Germany possible
  - → tendency of more frequent icing in the eastern parts (higher continentality → see lecture notes “Hydrology II”, chapter 1) → concerns especially Oder and Elbe rivers
  - → example for the frequency of solid ice formation → see table 3.1

Table 3.1: Solid ice formation frequency on Elbe river (from 1960)

Winter	Solid ice reach from Geesthacht weir upstream
1962 / 63	Elbe kilometre 21,3 (CR, few km downstream of Vrchlabi)
1971 / 72	Elbe kilometre 422 (Havelberg)
1978 / 79	Elbe kilometre 528 (Tießau)
1983 / 84	Elbe kilometre 553 (Heisterbusch)
1984 / 85	Elbe kilometre 328,5 (Magdeburg)
1986 / 87	Elbe kilometre 392 (Tangermünde)
1996 / 97	Elbe kilometre 291,5 (Barby)
2005 / 06	Elbe kilometre 555 (Radegast)
2008 / 09	Elbe kilometre 548 (Bleckede)

* **ice breakup in open channels:**
  - temperature > 0 °C → reduction of ice cover thickness → ice breakup → ice floe → perhaps ice jam possible
  - main difference of ice flow in case of ice formation: in most cases higher discharges in case of ice breakup (see figure 3.4) → higher flow velocities → higher mechanic power during ice breakup

3.2. Ice formation and ice breakup quantification methods

* **interesting things in connection with ice formation and ice breakup:**
  - ice formation:
    → position of 0 °C conditions on rivers
    → ice cover thickness
    → water level increase because of icing
  - ice breakup:
    → reduction of ice cover thickness
    → moment and position of ice breakup
    → transport velocity of ice floes

* **general calculation basement:** heat balance equation for open channels:

$$Q = Q_{KW} + Q_{LW} + Q_K + Q_L + Q_P + Q_{GW} + Q_{GB} + Q_F \tag{3.1}$$

where: Q - heat transformation respectively heat enthalpy of water body
Q_{KW} - short-wave radiation
Q_{LW} - long-wave radiation
Q_K - convective heat flux of the air
Q_L - latent heat flux because of evaporation
Q_P - heat flux of precipitation
Q_{GW} - heat flux of groundwater
Q_{GB} - combined geothermal and soil sediment heat flux
Q_F - fluid friction heat (positive value means heat flux to the water body)

[all parameters in J/(cm2/d)]

- main parameter heat supply: short-wave radiation
- main parameters of heat loss:
  → long-wave radiation
  → convective heat flux of the air
  → latent heat flux because of evaporation
- heat flux of precipitation and heat flux of groundwater may be positive or negative
- combined geothermal and soil sediment heat flux important in case of compact ice cover only
- fluid friction heat especially important in case of high flow velocities

- most important parameters for the process of ice formation: long-wave radiation, convective heat flux of the air, latent heat flux because of evaporation and heat flux of precipitation
- snowfall into the water body causes great heat loss:
  - → temperature < 0 °C
  - → release of latent heat because of snow melting
- big relation of surface area to river volume → fast heat exchange stream water / atmosphere
- **problem:** estimation of heat balance equation parameters sophisticated
- **problem solution:** usage of approximate formulae

* approximate formulae for ice formation quantification:

► *estimation of position of 0 °C conditions on rivers (regarding the meaning of the parameters → see figure 3.5):*

$$L_0 - L_X = -\frac{\rho_W * C_W * v * h}{C_0} \ln \frac{-T_L}{T_X - T_L} \tag{3.2}$$

where: L_0 - downstream position [m] of 0 °C condition
L_X - upstream position [m] where temperature T_X = X °C
ρ_W - water density [kg/m³]
C_W - specific heat capacity of water [J/(kg * K)]
v - mean flow velocity [m/s]
h - depth of the complete unfrozen water body [m]
T_L - air temperature [°C]
C_0 - heat exchange coefficient [W/(m^2 * K)]

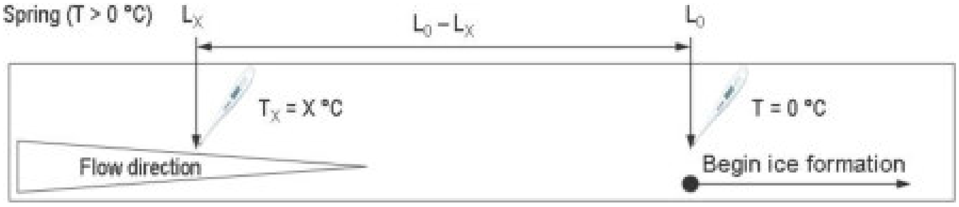


Fig. 3.5:

Parameter for estimation of 0 °C position

- equation 3.2 is valid for open channels which are cool down downstream → is normally the case because temperature of springs > 0 °C

► ***calculation of ice cover thickness:***

$$h(t) = 2 * \lambda * t \frac{T_W - T_L}{\rho * Q} + \frac{\lambda^2}{\alpha^2} - \frac{\lambda}{\alpha} \tag{3.3}$$

(symbol explanation → s. continuation page)

where: $h(t)$ - ice cover thickness [m]
λ - ice heat conduction coefficient [$W/(m^2 * K)$] → good isolator, value approximately λ = 2,22 $W/(m * K)$
t - time [s]
T_W - water temperature [°C]
T_L - air temperature [°C]
ρ - ice density [kg/m^3] → 918 kg/m^3
Q - amount of energy for phase transition water / ice → 333 J/kg
α - heat conductivity coefficient [$W/(m^2 * K)$] → parameter characterises the heat transition from ice to atmosphere → hard depending on wind speed (the higher wind speed the higher heat transport)→ mean value: approximately 6 $W/(m^2 * K)$

- boundary conditions for equation validity (see heat balance equation 3.1):
  - → valid for low flow velocity only gilt → negligence of fluid friction
  - → valid for big river thickness → negligence of des soil sediment heat flux
  - → valid for conditions where heat inputs or heat losses by precipitation and groundwater are negligible

► ***calculation of water level increase of a river because of icing:***

$$\frac{h_i}{h_0} = \left\{ 1 + \left[\frac{n_i}{n_b} \right]^{3/2} \right\}^{2/5} + 0,92 \, \frac{t_i}{h_0} \tag{3.4}$$

where: h_i - water level because of icing [m]
h_0 - water level without icing [m]
n_i - hydraulic roughness of ice cover (MANNING coefficient) [$s/m^{1/3}$]
n_b - hydraulic roughness of river bed (MANNING coefficient) [$s/m^{1/3}$]
t_i - thickness of ice cover [m]

*** approximate formulae for ice breakup quantification:**

► *calculation of ice cover thickness reduction in the phase of mechanical strength reduction:*

$$t_M = \beta * \sum D \tag{3.5}$$

where: t_M - ice thickness reduction [m]
β - empiric coefficient ($\beta \approx 0,004 \ldots 0,01$ m/(°C)
$\sum D$ - sum of daily mean of air temperature > 0 °C, related to the melt season

► *calculation of the moment and the position of ice breakup:* precise calculation not possible because of an great number of influences

→ first on rapids, weirs and water falls
→ usage of information from previous breakup periods

► *estimation of ice floes transport velocity:*

- → transport velocity ≈ river flow velocity
- → methods for estimation of flow velocity (measurement, calculation) → see lecture notes “Hydrology I” and lecture notes “Hydrology III”, chapter 1

3.3. Deepening literature

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Handbook of Hydrology. Chapter 7: Snow and floating ice. McGraw-Hill

Maniak, U. (2005):
Hydrologie und Wasserwirtschaft. Eine Einführung für Ingenieure. Springer-Verlag Berlin, Heidelberg, 5. Auflage

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Eisgefahren - Informationen – Maßnahmen – Zuständigkeiten. Druckfabrik Dresden, Februar 2007

4. Flood analysis

4.1. Flood formation

* **definition:**

flood = temporary discharge increase above base discharge

- temporal behaviour (rapidity of discharge increase and discharge recession) → in dependence on catchment characteristics → see lecture notes “Hydrology I”
- base discharge → part of base flow (groundwater component) which drains off over ground
- in practical work mostly: discharge exceeds bank-full discharge (or bank-full stage), sometimes: discharge (in some cases water level) exceeds defined discharge threshold value
- bank-full stage approximately calculable:

$$HWU = MHW - 1/3\,(MHW - MW) \tag{4.1}$$

where: HWU - bank-full stage
MHW - mean flood level
MW - mean water level

* **flood reasons, flood formation:**

- overview → see figure 4.1

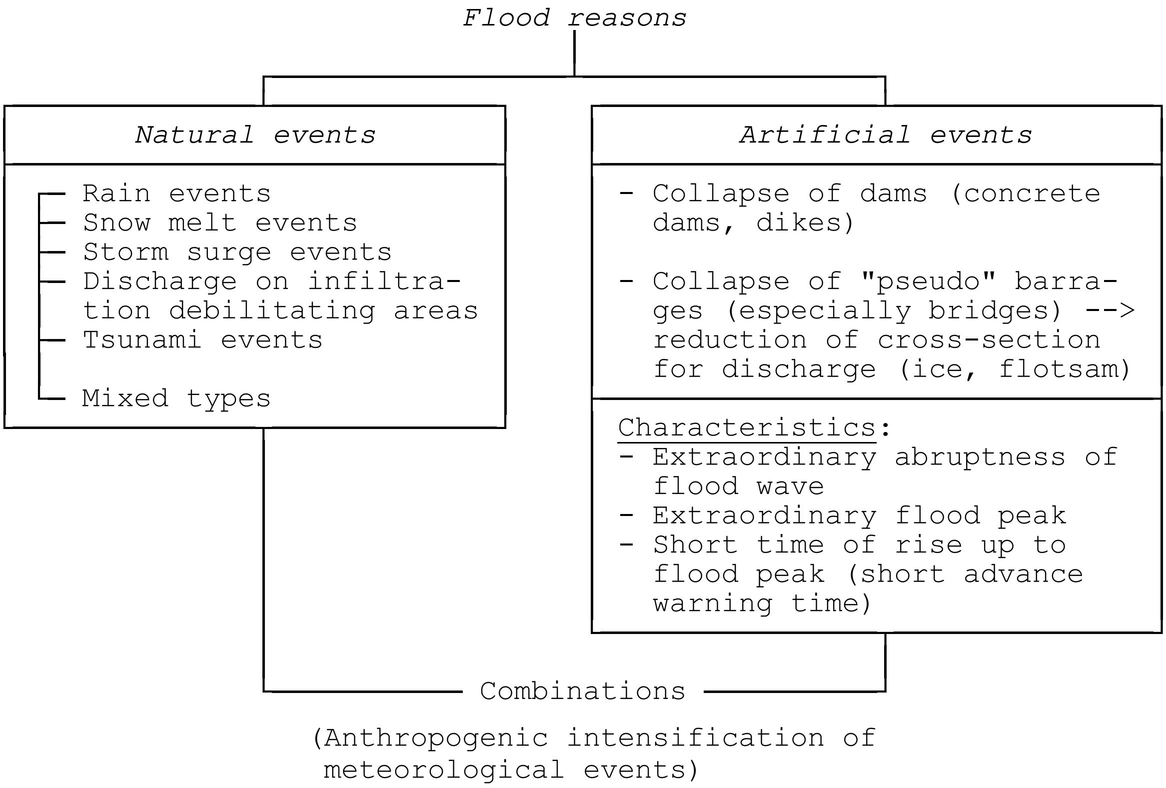


Fig. 4.1: Schematic overview on flood formation events

* Characteristics of meteorological inducted floods:

a) floods because of rain events:

▹ storm rain events (shower, thunderstorm):
- relevant for small catchments (approximately < 50 km^2)
- rainfall intensity in a range of some mm/min
- rainfall duration < 1 h

▹ continuous rain:
- relevant for large catchments
- long continuing, spacious, intensive rainfall events (frontal systems, windward precipitation)
- rainfall intensity ca. 5 ... 20 mm/h (maximum)
- duration: some days
- typical weather conditions for continuous rain events in south and east Germany: V_b weather conditions

b) snow melt floods:

- quickly ablation of a thick snow pack in the mountains
- snow melt on water saturated or frozen soil in the lowlands

c) storm surge floods:

- mostly caused by triple effects: strong landward winds + flood caused by rain or snow melt + high tide (for example storm surge disaster Hamburg 1962)

* formation or intensification of floods because of anthropogenic activities:

▹ *dike breaks:*

- causes:
  - → dike overflow → retrogressive erosion (air flow starting and progressing)
  - → undercurrent → underground saturation by undercurrent and sliding surface formation → construction movement because of upstream side pressure
  - → hydraulic base failure → dike undercurrent in case of an impermeable layer near the surface → effect of buoyancy
  - → perfusion → in cases where impermeable substrata used for dike construction (embankment failure, inner erosion)
  - → for details → see courses “Engineering Geology” and “Dam Construction”
- remedy: optimized dike construction from hydraulic point of view → see figure 4.2

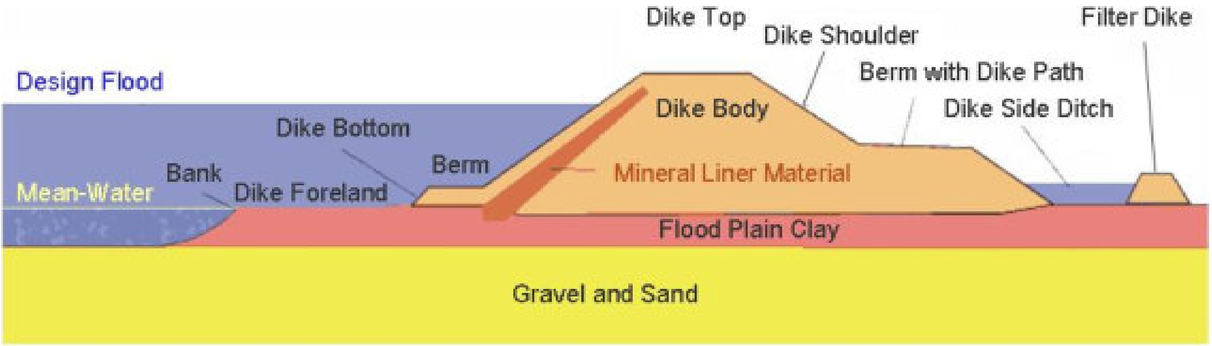


Fig. 4.2: Cross-section of a modern river dike (after O. Roehling)

- ***collapses of barrages:***
  - causes:
    - → great number of causes possible (separate or in combination)
    - → construction fault, material defects, subsoil problems, heavy rainfalls, storms, landslides, earthquakes, military war activities
  - examples for dam failures:

    <u>example 1: Teton dam failure (USA)</u>:
    - → zonal heaped up dam
    - → storage volume: 356 Mio. m^3 (for comparison: Bleiloch reservoir: 215 Mio. m^3)
    - → construction time: 1972 – 75, first complete filling in spring 1976
    - → dam zones with different compactions (upper part → slight compaction, lower part → 5 times higher compacted) → after complete reservoir filling: higher subsidence in the part with lower compaction → tensions, cracks, formation of hollows (also reinforced by the fact that dike materials were installed below the optimum water content)
    - → geological situation: in the valley rhyolites, overlain by alluvial layers, at the valley sides partially basalt → strong fissured → high water transmissivity
    - → technical activities for ground improvement: injection of concrete by means bore holes in the large fissures → with limited success only → dam undercurrent → lateral influx into the dam → disaster on 05.06.1976
    - → maximum discharge after dam collapse: > 20 000 m^3/s during the 5-hour emptying time (for comparison: river Elbe, gauge Dresden: mean discharge = 325 m^3/s, flood discharge in 2002: 4 860 m^3/s)
    - → damage balance: 11 deaths, flooding length: 120 km, total damage: about $ 1 billion
    - → conclusions: review of all dams immediately after the disaster, safety programs mandatory for new buildings dam, periodic monitoring of all dams in the United States legally after 1976

    <u>Example 2: Vajont landslide (Italy)</u>:
    - → location: Italian Alps, approximately 90 km north of Venice
    - → type: Concrete arch dam, construction period: 1956 – 1961, height: 261 m (Cologne Cathedral: 157 m), till then the world's highest dam, the purpose of the system: power generation, flood control, storage capacity: 150 million m^3
    - → landslide hazard area → limestone overlay by clay layer → slip plane
    - → October 1960 (stowage level: > 170 m): landslide with a slip velocity of 3.5 cm/d and a crack length of about 2 km
    - → 04.11.1960 (stowage level: 180 m): landslide (700 000 m^3), duration: app. 10 min (slow)
    - → after that drawdown the water level to 135 m → decrease of slip velocity to 1 mm/d
    - → deducted false conclusion: slip velocity = f (water level) → continued operation of the construction (water level and slide controlled)
    - → 09.10.1963: suddenly rapid increase of slip velocity to about 20 cm/d → rapid discharge of water out of the reservoir → reduction of hydraulic counterpressure and buoyancy reduction of the rock mass → reduced water levels = reduction of the pore water pressure of the clay layer → reduction of internal friction → landslide
    - → clod of earth of about 2 000 m, 1500 m wide and 100 thickness moves down
    - → about 30 million cubic meters of mud and water spill over the dam like a tsunami
    - → dam does not collapse despite of 8-fold overload
    - → flood wave: approximately 140 – 250 m high, speed nearly 100 km/h (jet effect)
    - → 5 villages hardly destroyed (strongest Longarone), about 2 000 death

Example 3: dam failure Malpasset (France):

→ construction period: 1952 - 1954
→ reinforced concrete arch dam, height: 59 m, crest length 222 m, storage space: 48 million m³
→ purpose of the dam for water supply, irrigation
→ sudden collapse on 02.12.1959
→ causes: pressure of the building closed fractures in the subsurface (gneiss) → fissure water pressure pushes he abutment of the dam to above, additionally high storage water level
→ Effects: 40-meter high tidal wave, v = 70 km / h, 421 deaths, total damage: $ 68 million

Example 4: Banqiao dam failure and 61 others (China):

→ largest flood disaster worldwide → 62 dam failures due to domino effect (08.08.1975)
→ meteorological reason: typhoon "Nina" (P_{MAX} = 160 mm/h or 1 630 mm/d, duration: several days, T ≈ 2 000 a) → Q_{MAX} ≈ 78 000 m³/s
→ 86 000 direct flood victims, 145 000 victims of epidemics, approximately 6 million buildings destroyed, about 11 million homeless
→ accident causes: botch in connection with planning and construction (partly usage of inappropriate building materials)

Example 5: dam breaks in Germany:

→ wars → e.g. World War II: Möhne dam (1 600 death), Eder dam (nearly 100 death)
→ associated with exceptional floods (dam failures Königheim-Gissigheim (Baden-Württemberg) on 21.06.1984 and Glashütte (Saxony) on 12.08.2002

▹ ***collapse of pseudo barrages:***

- Pseudo barrages = buildings that are not measured as storage structures, especially bridges
- Processes: Offset of floating debris or ice → reduction of discharge cross-sectional area → water impoundment → pressure load, possibly overflowing → overload, possibly undermining the abutment structure → fracture

* Examples of flood-prone areas: Gottleuba and Müglitz catchments (East Erzgebirge):

▹ ***catchment characteristics:***

- location of the catchment areas→ see figure 4.3

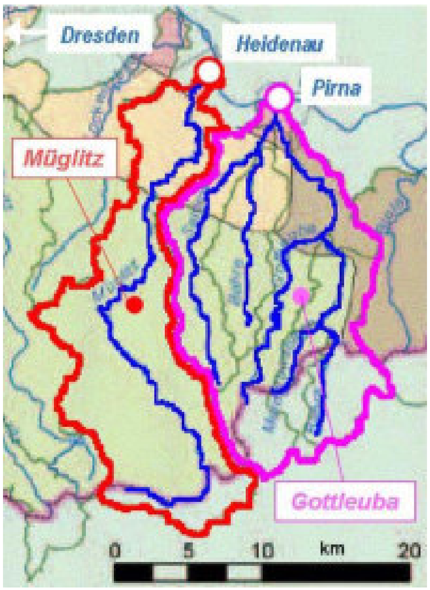


Fig. 4.3:

General map of the catchment areas of rivers Müglitz and Gottleuba

- <u>regular recurrence of catastrophic flood events</u>:
    - 13 December 1480: no information about damages
    - 1552: about 100 dead, despite low population density
    - 30 July 1897: probably more than 8 dead, 7.5 million Gold Marks total damage
    - 8 / 9 July 1927: 152 dead, 70 million Reichmark total damage
    - 22 July 1957: about 100 million GDR Marks total damage
    - 1958: about 50 million GDR Marks total damage
    - 12 / 13 Aug. 2002: 12 killed in the Erzgebirge, approximately EUR 290 million total damage

- <u>Causes of flood formation</u>:
    - well known since the turn of the 19th /20th century
    - Deforestation and drainage of forest land in the Osterzgebirge (Eastern Ore Mountains)
      → beginning with the settlement in the 11th century (tin ore mining)
      → deforestation in large scale with the 3rd wave of settlement in 1480 (in connection with the expansion of mining)
      → decrease in woodlot to about 30 % in the flood forming regions
    - flood-forming morphology
      → enlarged plateaus at the catchment area boundaries:
        → gneiss plateau, flat wavy, large headwater area, high river density, streams meander
      → deep valleys with steep slopes in the middle and lower reaches of the streams:
        → middle reaches: Tourmaline granite, v-shaped valley, low river density, nearly no meandering of the river
        → lower reaches: Cretaceous layers, U-shaped valley, low river density, nearly no meandering of the river
      → flood favouring discharge concentration conditions
    - flood favouring from meteorological point of view:
      → occurrence of distinctive V_b weather conditions each approximately 10 years
      → frequent occurrence of south-eastern weather conditions → showers and thunderstorms

- <u>causes of the high flood damages</u>:
    - high population density in the valleys
    - cultivation (housing) and land use (agriculture, wood industry, mills) into the flood control areas (directly to the streams)
    - until 1957 no flood protection measures

▹ *characteristics of the flood event of 1897:*

- Long-lasting rains due V_b weather condition at the end of July
- maximum: 30. July 1897
  → days rainfall: 110 mm/d, water-saturated soils → high runoff formation
  → swelling the discharge of river Gottleuba to about 100 m^3/s, the river Müglitz to about 240 m^3/s (for comparison: MQ of river Elbe at Dresden gauge: 325 m^3/s)
- damages:
  → villages destroyed and damages to bridges, railways, telephone network in the valleys of rivers Müglitz and Gottleuba
  → at least 8 dead
  → most affected site: Weesenstein

▹ *characteristics of the flood event of 1927:*

<u>meteorological situation</u>:

- low over central Europe (V_b cyclone track, rather precipitation characteristics of South-Eastern situation accordingly)
- rainfall amount: ca. 200 mm in some hours only, peak: 113 mm in 25 min (localized)
- nearly stationary air mass boundary (cold/warm) to the Erzgebirge → weather map of 8 July 1927 → see figure 4.4

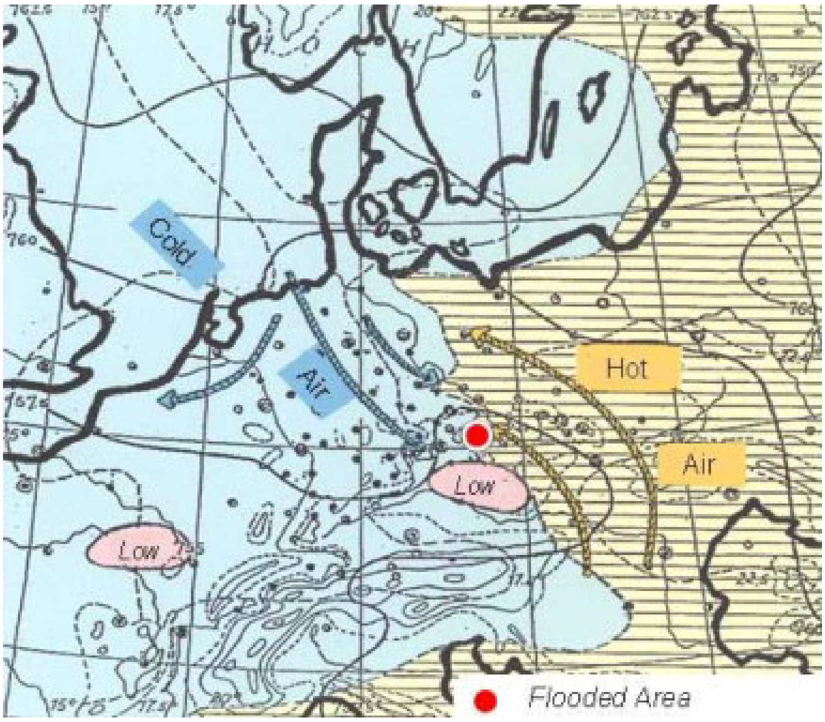


Fig. 4.4:

Weather map of 8.7.1927 (after W*EICKMANN, 1927)*

- weather and flood history:
  - ● 15:45 – 19:00 clock:
    - several storms → flood without significant damages
    - discharge decrease after 19:00 clock
  - ● about 21:00 clock: anew storm
  - ● 22:30 clock: storm peak
    - strong erosion processes → transport of uprooted trees, flotsam … in the water
    - storage of trees at several bridges above villages Gottleuba and Berggießhübel
    - "Pseudo" dams break → exceptionally steep flood wave amazed the population
    - maximum discharges: river Gottleuba (gauge Neundorf): 433 m^3/s, river Müglitz (gauge Dohna): 330 m^3/s
    - damage balance: 152 dead, 110 destroyed and 160 damaged bridges, 188 buildings destroyed, 70 million RM damage
    - first aid comes after 4 days

▹ *characteristics of the flood event of 1957:*

- long-lasting rains due V_b weather condition, showery intensifies
- flood peak discharges: comparable with those of 1897
- compared to 1927 slower swelling of the discharge → no human sacrifice

- *characteristics of the flood event of 2002:*
  - high soil moisture of the area by high rainfall in late July / early August
  - 11. – 13.08.2002: classical V_b weather situation with long-lasting rainfall
    → area rainfall: Gottleuba catchment: 225 mm, Müglitz catchment: 296 mm (T >> 100 a)
  - runoff coefficients: 50 – 90 %
  - maximum discharge rates: river Gottleuba: 135 m^3/s (effective flood protection)
    river Müglitz: 400 m^3/s = HHQ (comparatively low flood protection)
  - return periods: river Gottleuba: app. 100 years, river Müglitz: > 200 years
  - damage: 12 people killed in the Eastern Ore Mountains, approximately EUR 290 million total damage in both catchments

- *chronicle of flood protection:*
  - *before to the event in 1897*:
    - project proposal by architect Otto Horn (1890):
      → stone wall, 24 m high, 230 m long, reservoir area: 23 ha
      → storage capacity: 2.3 million m^3 = 2.3 hm^3
      → costs: 0.9 million Gold Marks
  - *between 1897 and 1927*:
    - calling for the enforcement of the Horn project proposal by the mayor of Gottleuba in provincial and state authorities
    - construction of dams in the valleys of the rivers Red and Wilde Weißeritz only → higher industrialisation → priority with respect to flood protection
    - no flood protection measures in the East Ore Mountains
  - *between 1927 and 1957*:
    - taking up again the Horn project, project cost now totals about 10 million Reichmark
    - required storage volume > 4 million m^3
    - example for a storage volume calculation based on flood event of 1927 → see figure 4.5
    - end of the 1920s: planning work of three dams (2 in the Müglitz valley, 1 in Gottleuba valley) for a total of 30 million Reichmark, planned buildings construction: in the period 1930 – 39
    - at the end of the 1920s recognized and required: no flood protection to technical measures only
    - but: no implementation of flood protection measures due to unemployment costs, war reparations, armament, war and postwar years

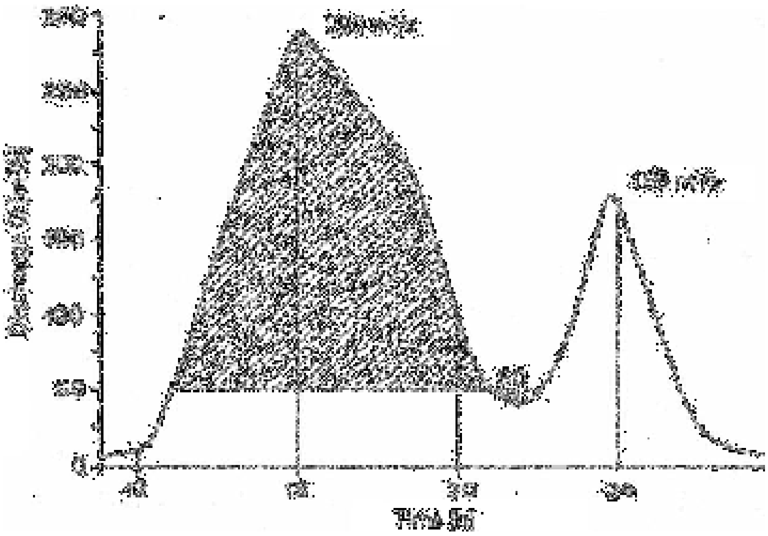


Fig. 4.5:

Reservoir storage calculation, based on a modified discharge hydrograph of the flood event on 8^{th} and /9^{th} July 1927 (after DREHER, 1927)

- *<u>after the event in 1957</u>:*
  - 1958: start of construction of the flood protection system in East Ore Mountains
  - in the mid-70s, completion of the flood control system with completion of the dam Oelsengrund (Gottleuba valley):
    → storage capacity: 13.2 million m^3 (multi-purpose storage), storage area: max. 66 acres
    → cost: app. 180 million GDR Marks
  - today: prioritization of natural retention capacity in combination with technical measures

- *<u>Balance of costs (flood protection / flood damage)</u>* → see table 4.1

Table 4.1: Costs for flood protection in comparison to the total flood damage

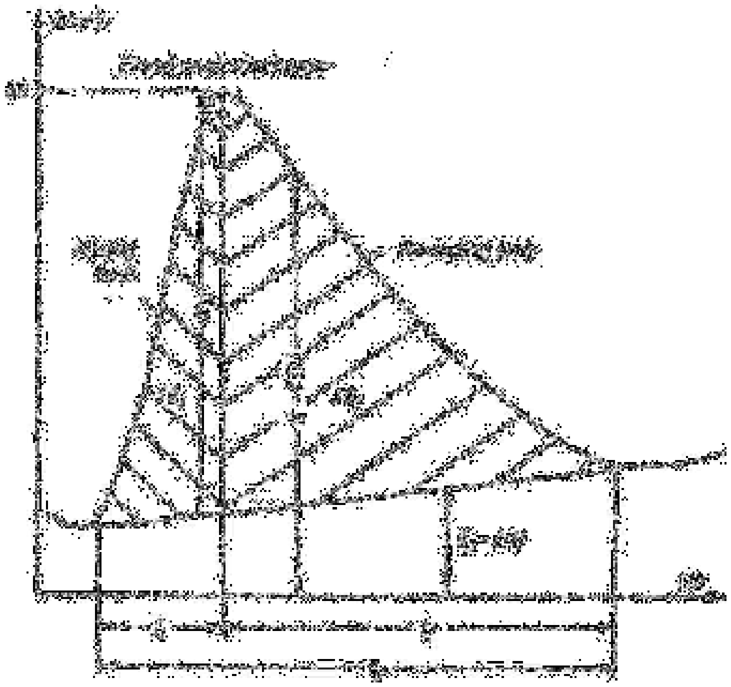
Perid	Project costs or construction costs	Total loss
Before 1897	About0.9 million Gold Marks	
Flood event of 1897		Approximately 7.5 million Gold Marks
After 1897	? (Inflation)	
Flood event of 1927		Approximately 30 million Reichmark
After 1927	About 10 – 30 Mio. Reichmark	
Flood event of 1957		Approximately 67 million GDR Marks
Flood event of 2002		Approximately EUR 290 million

4.2. Flood hydrograph characteristics

* **Flood peak discharge HQ** (→ see figure 4.6)**:**

▹ ***important for:***

- river control (waterway construction: dike heights, flood plains)
- design of flood spillways in case of small dams
- design of further flood discharge constructions (for example side-channel spillways)



Q - Discharge
Q_B - Base flow discharge
HQ - Flood peak discharge
Q_S - Flood peak discharge after base flow discharge separation
t_S - Flood rise time
t_f - Recession time
t_g - Flood duration
SQ_S - Discharge sum of flood rising
SQ_f - Discharge sum of flood recession
SQ_g - Total discharge sum

Fig. 4.6:

Characteristic values of a flood wave (after Dyck, et al., 1976)

* **total duration of the flood event t_g:**

- ▹ *key parameter for:* stability of dikes (overflow, undercurrent, perfusion, hydraulic base failure → s. chapter 4.1)

* **Flood rise time t_S:**

- ▹ *key parameter for warning:*
  - a few minutes (for example on dam failures)
  - a few hours (in small catchment areas)
  - several days ... weeks (in large catchment areas)

* **flood discharge sum SQ:**

- ▹ *key parameter for:*
  - design (respectively volume) of reservoirs (dams and retention basins)
  - design of spillways of larger dams

  → flood discharge sum results from discharge hydrograph

  → Flood discharge sum of one single flood event can account up to 20 % of the annual discharge amount!

* **further characteristics:**

- return period T (a measure of the frequency of occurrence)
- characteristic values of the time-spatial progress (see lecture notes “Hydrology I”, chapter 5.7, especially figure 5.46):

  → time delay of flood wave downstream

  → decrease of the peak discharge (reason: retention)

  → increase in flood discharge sum (source: tributaries)

4.3. Flood calculation methods

4.3.1. Aims of flood calculations

* **general objectives:**

- investigation of the regularities of the flood formation
- flood forecast
- preparation of hydrological information design of equipment in or near rivers
- river management regarding discharge
- control of water management constructions
- deduction of arrangements for flood protection

* **specific aims related to the course “Hydrology III”:**

- answer the questions: What return period has a flood event?
- or the reverse: What value is a flood with a certain return period T concerning flood peak level, flood peak discharge respectively flood discharge sum?
- It will not answer the question, when a flood with a certain return period occurs. → is a essential task for flood forecasting

4.3.2. Overview on calculation concepts

- calculation concept depends on the availability of long term discharge data
- in the case of the presence of high-resolution discharge rates (at least daily data) over a long (at least 20 years) period → extreme statistic investigations → stochastic conception
- in the absence of such data: flood calculation based on catchment area information → deterministic conception
- overview of contents of the two concepts → see figure 4.7

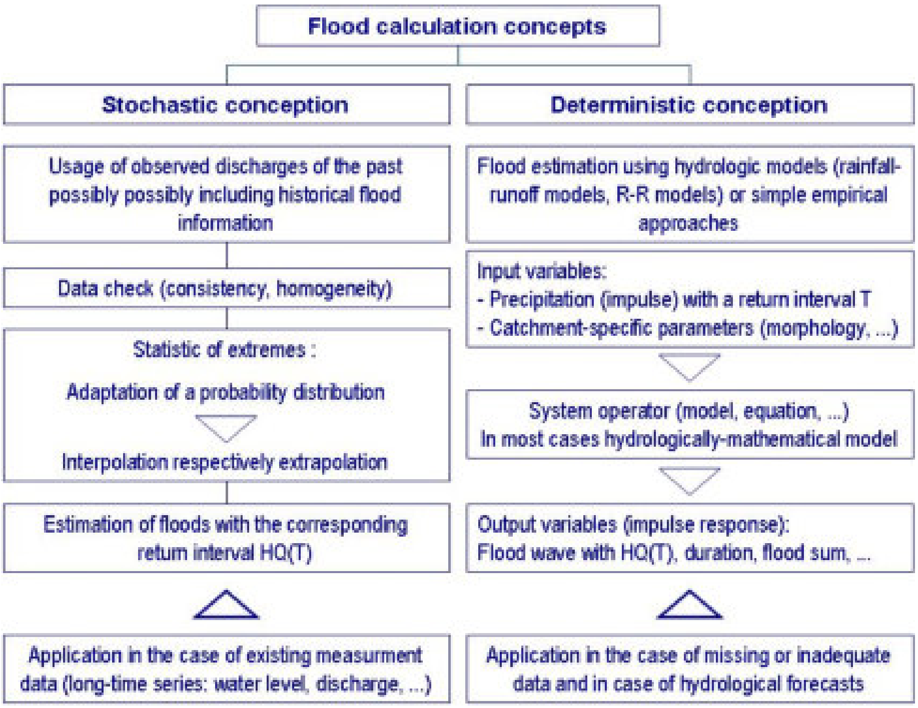


Fig. 4.7:

Flood calculation concepts

4.3.3. Stochastic conception

* **methodology explanation on example of a flood data series of river Flöha, gauge Borstendorf:**

a) selection of the observation period:

- length of series of observations: > 20 ... 30 a
- source: “Gewässerkundliche Jahrbücher” (German Hydrological Yearbooks)
- Observation period for this example: 1931 – 1967 (→ see table 4.2)

b) selection of the largest annual flood peak discharges HQ(a):

- source: “Gewässerkundliche Jahrbücher” (German Hydrological Yearbooks)

- advantages in case of usage of the annual maximum floods (each year one flood peak discharge):
→ criterion of independence of the flood peak discharges (mutual non-interference) fulfilled
→ extrapolation simplified return period: years

- Other selection criteria (for example all flood peak discharges above a threshold value) have not such advantage.

- values for the example → see table 4.2
- data verification for consistency and homogeneity → see lecture notes “Hydrology I”, chapter 5.5.1

Table 4.2: Annual maximum flood peak discharges HQ(a) of gauge Borstendorf, river Flöha, 1931 – 1967

Year	HQ(a) $[m^3/s]$	Year	HQ(a) $[m^3/s]$	Year	HQ(a) $[m^3/s]$	Year	HQ(a) $[m^3/s]$
1931	36.6	1941	82.8	1951	28.4	1961	50.0
1932	235	1942	89.9	1952	50.0	1962	39.7
1933	77.3	1943	24.4	1953	89.2	1963	22.8
1934	44.6	1944	122	1954	147	1964	32.5
1935	81.2	1945	74.3	1955	104	1965	143
1936	52.9	1946	137	1956	54.4	1966	75.0
1937	192	1947	93.9	1957	158	1967	73.8
1938	62.0	1948	112	1958	208		
1939	99.8	1949	89.2	1959	46.5		
1940	106	1950	33.0	1960	80.9		

c) rank of peak discharges HQ(a):

- starting with the smallest HQ(a) → see table 4.3
- ordinal number (serial number) m = 1 ... n (here: n = 37), 1 – smallest value, n – largest value

Table 4.3: Flood calculation table for parameter determination for gauge Borstendorf, river Flöha, flood data series: 1931 – 1967

m	HQ(a) $[m^3/s]$	P_U [%]	m	HQ(a) $[m^3/s]$	P_U [%]
1	22.8	2.63	20	80.9	52.63
2	24.4	5.26	21	81.2	55.26
3	28.4	7.90	22	89.2	57.90
4	32.5	10.53	23	89.2	60.53
5	33.0	13.16	24	92.8	63.16
6	36.6	15.79	25	93.9	65.79
7	39.7	18.42	26	99.8	68.42
8	44.6	21.05	27	104	71.05
9	46.5	23.68	28	106	73.68
10	50.0	26.32	29	112	76.32
11	50.0	28.95	30	122	78.95
12	52.9	31.58	31	137	81.58
13	54.4	34.21	32	143	84.21
14	62.0	36.84	33	147	86.84
15	73.8	39.47	34	158	89.47
16	74.3	42.11	35	192	92.11
17	75.0	44.74	36	208	94.74
18	77.8	47.37	37	235	97.37
19	79.9	50.00			
$\overline{HQ} = 87.80\ m^3/s$ $s(HQ) = 51.75\ m^3/s$					

m – rank HQ(a) – annual maximum flood peak discharge P_U – nonexceedance probability

$\overline{HQ}$ – mean value of annual maximum flood peak discharges
s(HQ) – standard variation of annual maximum flood peak discharges

d) calculation of nonexceedance probabilities P_u for each HQ(a):

$$P_u = m / (n + 1) * 100\ \% \qquad (4.2)$$

where: P_u - nonexceedance probability [%]
m - rank
n - total number HQ(a)

- usage of n + 1 (and not n values), because it can be assumed that the HQ of the observed series is not the maximum HQ in all, but rather there exists at least one greater HQ

- list of nonexceedance probabilities P_u in table 4.3

- direct connection of nonexceedance probability P_u and return period T (the so-called annuality) of a flood event:

$$T = 1 / [1 - (P_u / 100\ \%)] \qquad (4.3)$$

where: T - return period [a]
P_u - nonexceedance probability [%]

e) choice of the distribution function which is to be adapted to the HQ (a) values:

- relevant distribution functions in hydrology related to flood problems:

┌──> - extreme value distribution type I (EI)
│ - PEARSON distribution type III
│ - Log-PEARSON distribution type III
│ - Log-Normal distribution
└── in the flood statistics in most cases successful and therefore used

- For each distribution function exists a specific distribution function diagram (for EI → figure 4.8).

- If a distribution function can be adapted to the observation data series, then the values stretch in the distribution function diagram to a straight line.

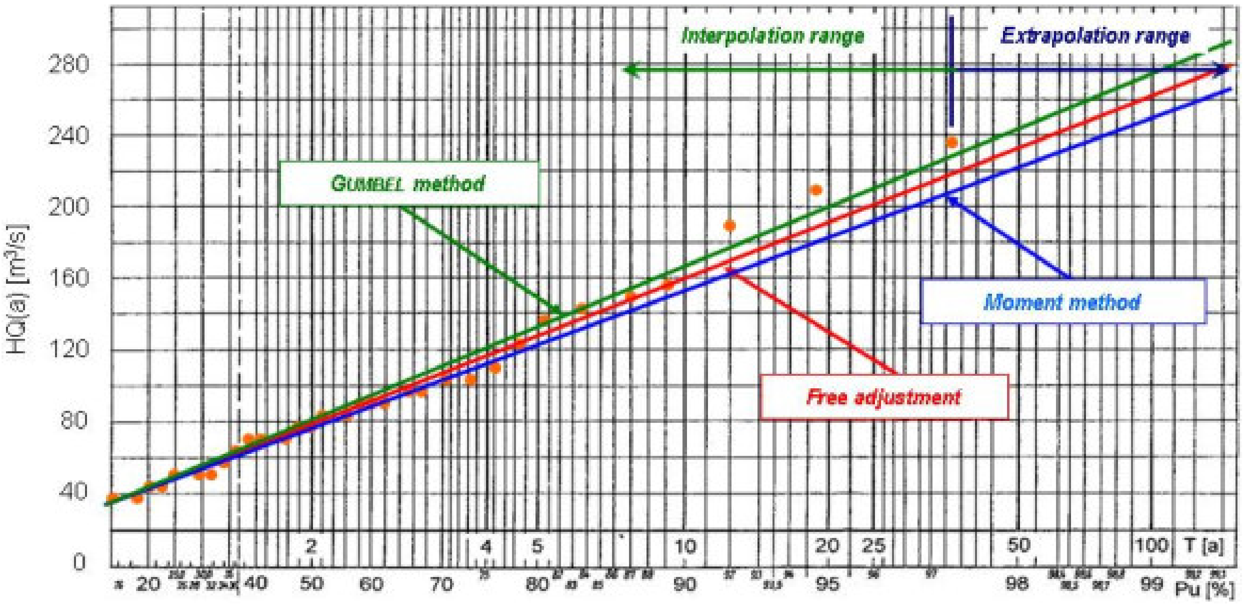


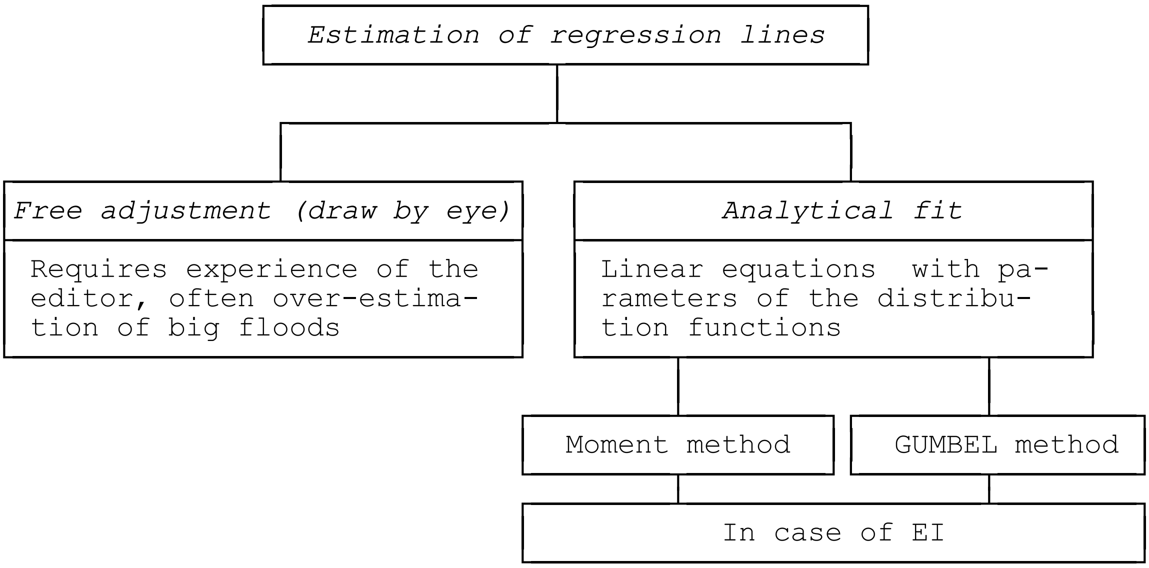
Fig. 4.8: Probabilities of the annual maximum flood discharges HQ of gauge Borstendorf, river Flöha, series 1931 to 1967 and adjusted probability distributions in EI distribution function diagram (in this case under consideration of historical floods), after DYCK, ET AL. (1976)

f) transferring the HQ with the corresponding P_U to EI distribution function diagram:

- y-axis: observed HQ(a) (from table 4.3)
- x-axis: nonexceedance probability P_u (from table 4.3) with corresponding return periods T [y]

g) regression line determination:

- overview on methods for the estimation of regression lines → see figure 4.9



Requires experience of the editor, often over-estimation of big floods

Linear equations with parameters of the distribution functions

Fig. 4.9: Methods for estimation of regression lines

h) estimation of parameter which are describing the regression line:

▹***Moment method:***

- linear equation of the moment method:

$$HQ(T) = \overline{HQ} + s(HQ) * k(T) \tag{4.4}$$

where HQ(T) - peak discharge [m^3/s] of a flood event with a return period T [a]
$\overline{HQ}$ - mean value of all HQ(a) [m^3/s]
s(HQ) - standard deviation of all HQ(a) [m^3/s]
k(T) - k(T) relationship

- determination of k(T) relationship by equation 4.5:

$$k(T) = -\frac{6^{0.5}}{\pi}\left[\gamma + \ln \ln \frac{T}{T-1}\right] \tag{4.5}$$

where k(T) - k(T) relationship
T - return period [a]
γ - EULER constant ($\gamma = 0.5772$)

- for the example is obtained:

$\overline{HQ} = 87.8\ m^3/s$

$s(HQ) = 51.75\ m^3/s$

$k(T) = -0.7797\ \{0.5772 + \ln\ln[T/(T-1)]\}$

▹***Gumbel method:***

- Linear equation of the GUMBEL method:

$$\boldsymbol{HQ(T) = mod\,(HQ) + 1/a * y(T)} \qquad (4.6)$$

where $$\boldsymbol{mod\,(HQ) = \overline{HQ} - \overline{y}_n * s(HQ) / \sigma_n}$$
(4.7)

and $$\boldsymbol{y\,(T) = -\ln\ln\,(T/(T-1))} \qquad (4.8)$$

and $$\boldsymbol{1/a = s\,(HQ) / \sigma_n} \qquad (4.9)$$

where $HQ(T)$	-	peak discharge [m^3/s] of a flood event with the return period T [a]
mod HQ	-	modal value of all HQ(a) [m^3/s]
$\overline{HQ}$	-	average of all HQ(a) [m^3/s]
$s(HQ)$	-	standard deviation of all HQ(a) [m^3/s]
$y(T)$	-	y(T) relationship
T	-	return period [a]
$\overline{y}_n$	-	arithmetic mean of the reduced order characteristic→ statistical tables, only dependent on the total number of values (→ see table 4.4)
σ_n	-	standard deviation of the reduced order characteristic→ statistical tables, only dependent on the total number of values (→ see table 4.4)

Table 4.4: Arithmetic mean $\overline{y}_n$ and standard deviation σ_n of reduced order characteristics in dependence on total numbers of values n

n	$\overline{y}_n$	σ_n	n	$\overline{y}_n$	σ_n
15	0.5128	1.0206	44	0.5458	1.1499
20	0.5236	1.0628	46	0.5468	1.1538
22	0.5268	1.0755	48	0.5477	1.1574
24	0.5296	1.0865	50	0.5485	1.1607
26	0.5320	1.0961	55	0.5504	1.1681
28	0.5343	1.1047	60	0.5521	1.1747
30	0.5362	1.1124	70	0.5548	1.1854
32	0.5380	1.1193	80	0.5569	1.1938
34	0.5396	1.1255	90	0.5586	1.2007
36	0.5410	1.1313	100	0.5600	1.2065
38	0.5424	1.1363	500	0.5724	1.2588
40	0.5436	1.1413	1000	0.5745	1.2685
42	0.5448	1.1458			

- for the example is obtained:

mod HQ = $87.8 - 0.5416\ (51.75 / 1.1338) = 61.3\ m^3/s$

1/a $= 51.75 / 1.1338 = 45.64\ m^3/s$

i) creation of linear equation:

- for the example is obtained:

Moment method: ***$HQ(T) = 87.8 + 51.75\,k(T)$*** ***(4.10)***

GUMBEL method: ***$HQ(T) = 61.3 + 45.64\,y(T)$*** ***(4.11)***

j) straight line plot to EI distribution function diagram:

- inserting of at least two different return periods T in the linear equation for moment method (equation 4.10) respectively GUMBEL method (equation 4.11)
- for the example is chosen: T = 50 y, 100 y and 500 y (→ see table 4.5)
- plot of the calculated straight lines to EI distribution function diagram (→ see figure 4.8)

Table 4.5: Flood peak discharges of gauge Borstendorf, river Flöha for different return periods T after different estimation methods

Estimation method		T = 50 y	T = 100 y	T = 500 y
Moment method	k(T)	2.59	3.14	4.38
	HQ(T) [m³/s]	222	250	315
GUMBEL Method	y(T)	3.90	4.60	6.21
	HQ(T) [m³/s]	241	273	346
Free adjustment	HQ(T) [m³/s]	232	262	330

k) extrapolation und interpretation:

- basics: figure 4.8 and table 4.5
- In the example the three largest flood peak discharges are lying over the lines of best fits.

possible causes:

a) used distribution function is “untrue” → teat another distribution function

b) flood peak discharges HQ have very likely higher return period T:
  - → risk of overvaluation of floods in the extrapolation range (especially in case of free adjustment)
  - → in the decision-making process (for example in relation to flood protection → see chapter 4.4) is indeed on the "safe side" - while not optimal (→ high costs)

- extrapolation range for n = 37 (37 years series of observations):

→ extrapolation without knowing the "true" distribution function, strictly speaking, not allowed (only interpolation)

→ error increases with extrapolation rage (data check!)

→ rule of thumb for determining the extrapolation range:

- T < 3 n (if no additional information available)
- T = x n (if no additional information available, for example historical flood marks) with x = f (type of additional information, number of additional information)
- in the example: extrapolation without additional information (n = 37 y) to about 100 years possible

* **information extension (usage of additional information):**

▹ ***overview on possible information extensions:***

▹ ***causal information extension:***

- rainfall-storage-runoff-statistics for a catchment including information on precipitation, soil moisture, storage capacity (storage ability of the catchment), …
- internal classification of floods into genetically homogeneous data collective (for example to summer floods, winter floods, snow smelt floods ...), disadvantage: data series reduction

▹ ***temporal information extension:***

- usage of historical sources:
  - → written sources: handwritten and printed paper (old chronicles, books, magazines, …) → caution: people have a tendency to exaggerate! critical information control! → quantitatively often not reliable
  - → pictorial, graphical representations: maps, pictures, photographs, drawings, old construction plans, maps of flood plains →in most cases quantitative respectively semi-quantitative reliable
  - → objective sources: flood marks, inscriptions, sediment deposits, localisation of things which are flushed away, … → in most cases quantitative respectively semi-quantitative reliable
- in case of great rivers mostly historical sources available, in case of small rivers or streams in most cases not
- problem: in most cases water level information deductable but not flood discharge information
- methodology for estimation of historical flood peak discharges by means of historical water marks:
  - → reconstruction of historical river cross-section (in most cases quite different from today)
  - → estimation of the historical longitudinal river slope
  - → assumptions about the hydraulic roughness on the river banks of the river bottom
  - → application of hydraulic models for flood peak discharge calculation
  - → requires experience of the hydrologist
- threshold statistics: involvement of HQ above a threshold (often flood peak discharge HQ ≥ 3 – 5 mean discharge MQ) → note independence of the HQ values!

▹ ***spatial information extensions:***

- usage of regional analogies → data transfer of an observed catchment (for example catchment with gauges) to an unobserved catchment or to sub-catchments
- requirement: similar hydrological characteristics of the catchments → transfer function = f (morphology, pedology, usage) → method rarely used without difficulties

- ***artificially created flood series:***
  - simulation of long-time discharge series (by means of rainfall-runoff-models)
  - filtering of the flood discharges
  - application of extreme value statistics to the so-generated discharge series
  - combination of deterministic and stochastic conceptions

- ***application for the example (→ see figure 4.8):***
  - extrapolation range in case of presence of historical flood information, which includes flood data from the last 200 years → to approximately 600 years expandable (3 * 200 y)
  - nevertheless: tendency of flood peak discharge overestimation for big return periods still exists

- application of stochastic conception → seminar 15

4.3.4. Deterministic conception

4.3.4.1. Overview

* **Characteristic of deterministic conception** (→ see figure 4.7):

- modelling of flood hydrograph (flood peak discharge, duration of the flood event, discharge sum ...) in case of data absence or insufficient data series
- often used in small catchments especially for design problems
- transformation of precipitation into runoff by means of system operator (usually mathematical models) considering catchment parameters → rainfall-runoff-models
- **practical determination of pedological parameters → see seminars 16 and 17**
- **heavy rainfall determination → see lecture notes “Hydrology I” and seminar 20**

* **possible approaches**:

- application of regionalisation methods (similarity considerations)
- usage of hydrological models (rainfall-runoff-transformation , taking into account the characteristics of the catchment area)

4.3.4.2. Regionalisation methods

* **aim:** transformation of flood values (flood peak level, flood peak discharge, discharge sum) from observed catchments to unobserved areas

* **methodology:**

- usage of long-term discharge measurements of an observed catchment
- derivation of statistical or empirical parameters
- transfer of these characteristics and parameters to the unobserved area (similar considerations)
- Calculation of flood values for the unobserved region → empirical method

* **envelope curve after WUNDT:**

▹ ***basis:*** empirical flood formulae considering the catchment area

▹ ***methodology:***

- plot of the world's highest measured flow values HHQ and related catchment areas in the double-logarithmic paper → see figure 4.10

- derivation of the relationship: $HHQ = a * A_E^{\,b}$ (4.12)

  where: HHQ – probable maximum discharge [m^3/s]
  A_E – catchment area [km^2]
  a and b – empirical coefficients

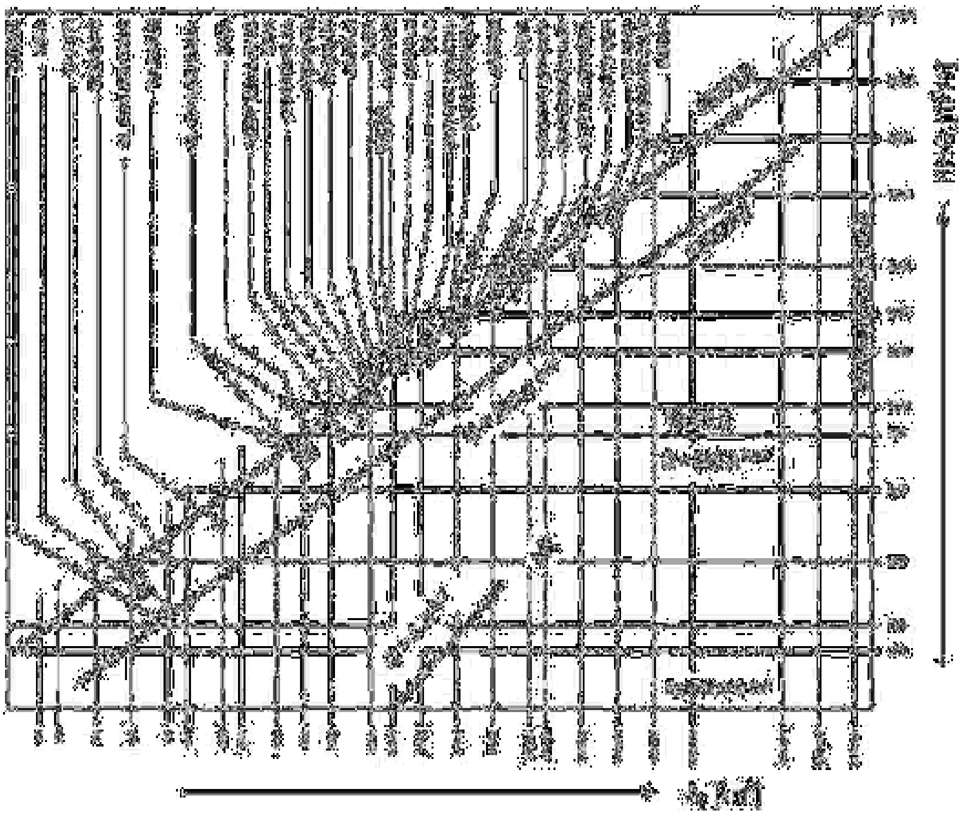


Fig. 4.10:

*Envelope curves of observed maximum discharges HHQ worldwide, in Europe and in Great Britain (after **HERSCHY, 2002)***

- note: very different conditions (especially regarding rainfall) worldwide → no transfer to any regions (for example to catchments in Germany) possible
- in addition to the envelope curve for the highest discharges → second envelope curve, which is not including the upper tenth of the peak values → 90% of peak discharges are below the curve → 90 % curve → occasionally equal to HQ(100):

$$HQ_{90\%} \approx HQ(100) = 13.8 * A_E^{\,0.6} \qquad (4.13)$$

(all symbols → see equation 4.12)

- note: often the tendency to over-estimation of HQ (100) resulting from the use of formula 4.13

▹ ***examples of the use of the 90% envelope curve after WUNDT for the estimation of HQ (100) values:***

- example 1: observed catchment up to gauge Borstendorf (river Flöha):
  - → catchment area: 641 km^2
  - → for comparison: HQ(100a) = 250 – 273 m^3/s (according to stochastic conception) → see figure 4.8., according to newer investigations (after the flood in 2002): HQ(100a) = 372 m^3/s
  - → $HQ_{90\%} \approx HQ(100) = 13.8 * A_E^{0.6} = 13.8 * (641\ km^2)^{0.6} = 667\ m^3/s$
  - → strong overestimation of HQ(100)

- example 2: unobserved Münzbach catchment (river outlet near Halsbrücke):
  - → catchment area: 150 km^2
  - → $HQ_{90\%} \approx HQ(100) = 13.8 * AE^{0.6} = 13.8 * (150\ km^2)^{0.6} = 279\ m^3/s$
  - → because there is no gauge → no assessment possible

▹ ***advantages of the envelope curves after WUNDT:***

- fast, uncomplicated application
- for calculation only one parameter (catchment area) necessary

▹ ***disadvantages of the envelope curves after WUNDT:***

- neglect of many influencing factors regarding flood peak
- in nearly all cases overestimation of HQ values → only a rough estimate of HQ values in the case of hydrologic preliminary studies
- caution especially in case of application for catchments < 100 km^2

* „Rational Formula“ – rational method:

▹ ***starting point:*** empirical formulas taking into account flood influencing parameters (mostly rainfall, runoff coefficient and catchment area)

▹ ***„original formula“:*** developed in 1889 in the USA (valid for very small catchments < 0.8 km^2):

$$HQ = 0.278\ \Psi * PI * A_E \qquad (4.14)$$

where: HQ – flood peak discharge [m^3/s]
Ψ – runoff coefficient [ ]
PI – rainfall intensity [mm/h]
A_E – catchment area [km^2]

- originally applied to the calculation of peak runoff in urban areas for sewer network dimensioning → because of the mostly close-meshed sewer network valid for very small catchments (< 0.8 km^2)
- various modifications worldwide → also for the calculation of peak runoff in small (possibly medium-sized) catchments

- ***for Bavaria modified rational formula:***
  - also applicable for catchments > 0.8 km^2

$$HQ(T) = \frac{\Psi * P * A_E}{0.5\ (t_{An} + t_{Ab}) * 0.06} \qquad (4.15)$$

where: HQ(T) – flood peak discharge with a return period of T years [m^3/s]
Ψ – runoff coefficient [ ] → for areas south of the Danube: 0.4 – 0.6. north: 0.5 – 0.7
P – volume of rainfall for the design rainfall duration (rainfall duration which is producing the maximum flood peak) [mm] → same return period as HQ
A_E – catchment area [km^2]
t_{An} – flood rise time to ,the flood peak [min]
t_{Ab} – recession time of the flood wave [min]

- calculation of the design rainfall duration (a assumption for the determination of precipitation amount in equation 4.15):

$$P_D = t_{An} = 227 \left(\frac{L^3}{\Delta h} \right)^{0.385} \qquad (4.16)$$

where: P_D – design rainfall duration [min]
L – length of flow from the watershed to the calculation point [km]
Δh – height difference [m]

- calculation of the recession time of the flood wave (assumptions for the equations 4.14 and 4.15) → depends on the retention properties:

$$t_{Ab} = F * t_{An} \qquad (4.17)$$

where: F – factor, depending on the retention capacity, values → see table 4.6

Table 4.6: Retention factor F as a function of land use

Land use	Hydraulic characteristics	F [ ]
Settlement area	Mostly paved, hydraulically smooth surfaces	1
Scattered buildings with gardens, agricultural land	Only partially paved areas	1.25
Mixture of forest, agricultural and meadow areas, building subordinate	Normal case	1.5
High proportions of forest, moorland, often peaty soils	High retention, rough conditions	2

- ***application of the for Bavaria modified rational formula for the estimation of HQ(100) values of river Flöha and stream Münzbach:***
  - example 1: observed Flöha catchment up to gauge Borstendorf (characteristics → see above):
    → runoff coefficient $\Psi = 0.55$ (derived from field information)
    → L = 47 km, Δh = 450 m

→ $P_D = t_{An} = 227\ (47^3 / 450)^{0.385} = 1\,844$ min = 31 h
→ P for P_D = 31 h using KOSTRA-Atlas of German Weather Survey DWD: P = 140 mm
→ $t_{Ab} = 1.5\ t_{An} = 1.5 * 1\,844$ min = 2 766 min (factor F = 1.5 from table 4.6)
→ HQ(100) = 357 m^3/s
→ good comparability with stochastically determined values (see above)

- example 2: unobserved Münzbach catchment (river outlet near Halsbrücke, characteristics → see above):
  → runoff coefficient $\Psi = 0.55$ (derived from field information)
  → L = 13 km, Δh = 240 m
  → $P_D = t_{An} = 227\ (24^3 / 240)^{0.385} = 1\,081$ min = 18 h
  → P for P_D = 18 h using KOSTRA-Atlas of German Weather Survey DWD: P = 113 mm
  → $t_{Ab} = 1.5\ t_{An} = 1.5 * 1\,081$ min = 1 622 min (factor F = 1.5 from table 4.6)
  → HQ(100) = 115 m^3/s
  → because there is no gauge → no assessment possible

* **index flood method:**

▹ ***aim:*** transfer of information of a observed (mostly large) catchment on unobserved (mostly small) areas by correlation (premise: flood-homogeneous region)

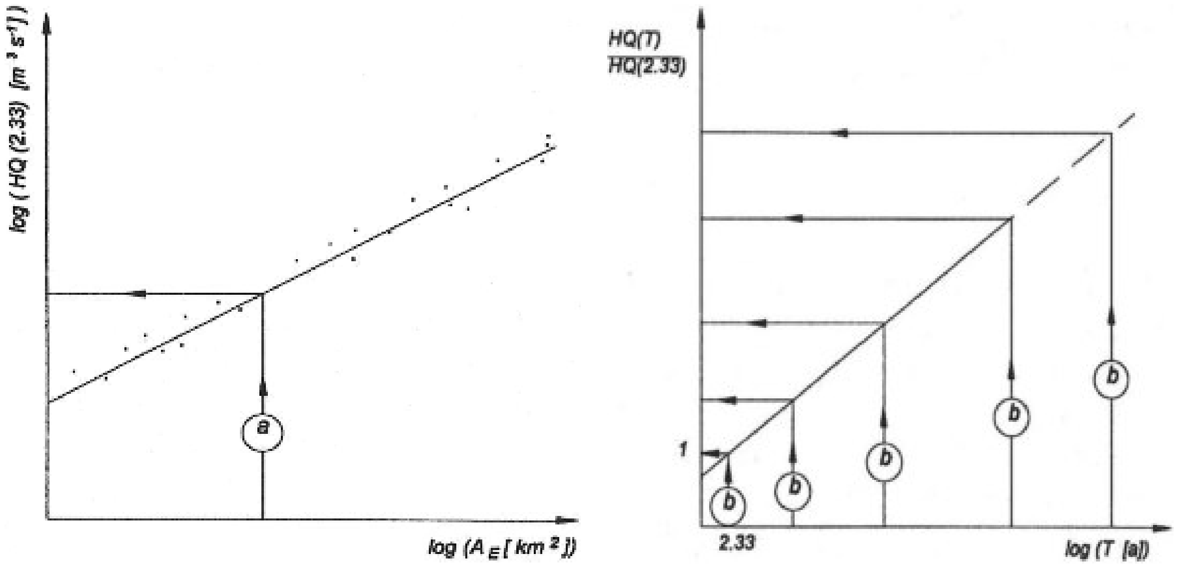
▹ ***methodology:***

- initial consideration: HW probability distributions of sub-areas have in a flood-homogeneous region a similar form
- use of features of the HW-probability distribution for the transmission
- example: type I extreme value distribution (EI):
  → crucial feature: flood peak discharge HQ with a return period T = 2.33 y
  → HQ(2.33) corresponds to the expectancy value $E(Y) = \gamma = 0.577$ → EULER constant
  → for HQ(2.33) is true in many cases: $HQ(2.33) = f\,(A_E)$
  → HQ(2.33) → reference value
  → conversion of HQ (2.33) value to different HQ (T)

▹ ***practical transformation of the methodology:***

a) estimation of reference value HQ(2.33):
  - research as many gauges as possible with long-time series of measurements
  - application of stochastic conception
  - estimation of the flood value HQ(2.33) for each gauge (EI distribution function diagram)
  - plot of HQ(2.33) as a function of catchment area A_E in log-log paper → see figure 4.11 a

b) conversion of HQ(2.33) to HQ(T):
  - Calculation of the quotient HQ (T) / HQ (2.33) for all measured gauges
  - Graphical representation → see figure 4.11 b
  - basis for the estimation of HQ(T) in unobserved sub-catchments



a) Dependence of HQ (2.33) value of the catchment area A_E

b) Conversion of HQ (2.33) values for other return intervals

Fig. 4.11: Estimation methodology of flood peak values with different return periods by means of flood index method (DYCK, PESCHKE, 1995)

- ***application of flood index method for the determination of HQ(100) values of river Flöha and stream Münzbach:***
  - example 1: observed Flöha catchment up to gauge Borstendorf (characteristics → see above):
    - → estimation of HQ(2.33) → see figure 4.12
    - → HQ(2.33) ≈ 85 m^3/s

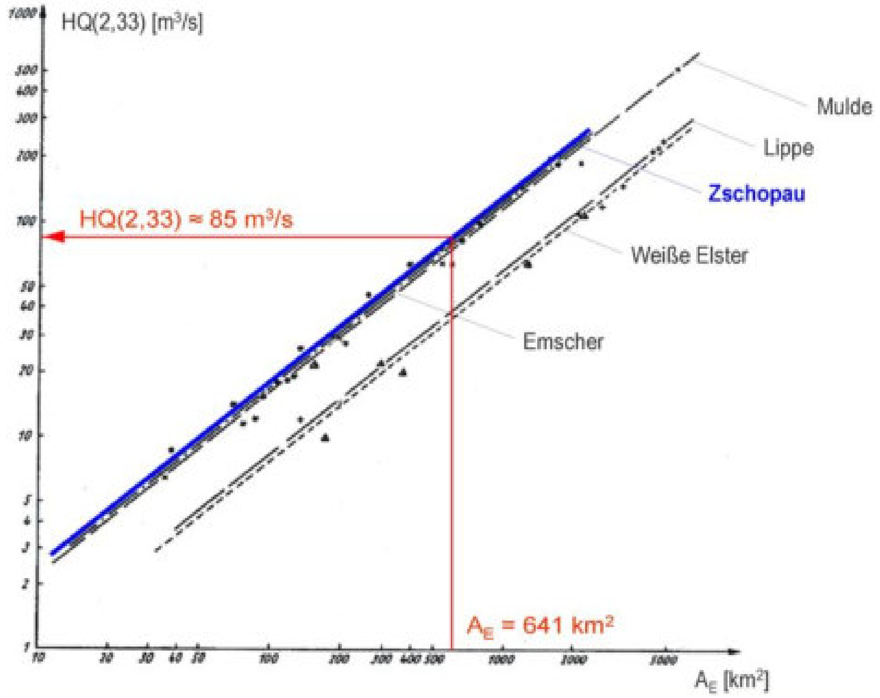


Fig. 4.12:

Estimation of the reference value HQ(2.33) for Flöha catchment up to gauge Borstendorf (subcatchment of river Zschopau), image basis: DYCK, PESCHKE (1995)

→ determination of HQ (100) value (2.33) on the basis of the reference value HQ (2.33) → see figure 4.13

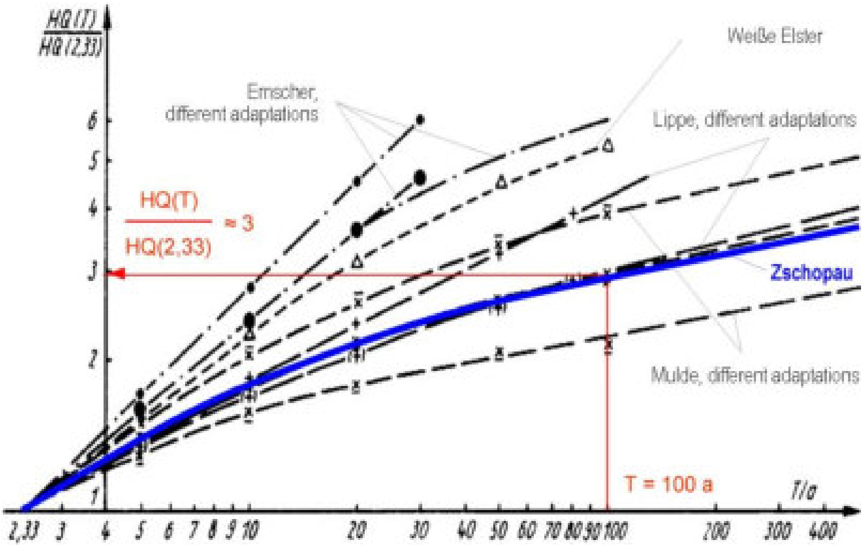


Fig. 4.13:

Determination of the ratio HQ (T) / HQ (2.33) for the catchment area up to the gauge Borstendorf (sub-catchment of the river Zschopau), image basis: DYCK, PESCHKE (1995)

→ $HQ(100) \approx 3\ HQ(2.33) \approx 3 * 85\ m^3/s \approx 255\ m^3/s$
→ good comparability with stochastically determined values (see above)

- example 2: unobserved Münzbach catchment (river outlet near Halsbrücke, characteristics → see above):
  → $A_E = 150\ km^2$ → $HQ(2.33) \approx 22\ m^3/s$ (after figure 4.12)
  → $HQ(T) / HQ(2.33) \approx 3$ (for T = 100 a after figure 4.13)
  → $HQ(100) \approx 3\ HQ(2.33) \approx 3 * 22\ m^3/s \approx 66\ m^3/s$
  → because there is no gauge → no assessment possible

* **extended flood index method after LAUTERBACH/GLOS:**

▹ ***starting point:***

- measured monthly flood peak discharges HQ(m) at 179 gauges
- in addition to the catchment area A_E are used more parameters for HQ estimation
- use of monthly flood peak discharges HQ(m) and not of annual flood peak discharges HQ(a), because for many of the 179 gauges, no long-term discharge measurements were available (sometimes only 7 years)) → increase the amount of data by a factor of 12 in the case of using monthly HQ values
- Relationship between HQ (m) and HQ(a):

$$T\,(a) \approx \frac{1}{12}\,T\,(m) \qquad (4.18)$$

where: T (a) – Wiederkehrsintervall [a]
T (m) – Wiederkehrsintervall [mon]

▹ ***steps:***

- adaptation of a probability distribution function → here: not EI but lognormal distribution (monthly HQ values stretch in the log-NV-distribution diagram to a straight line) → see figure 4.14

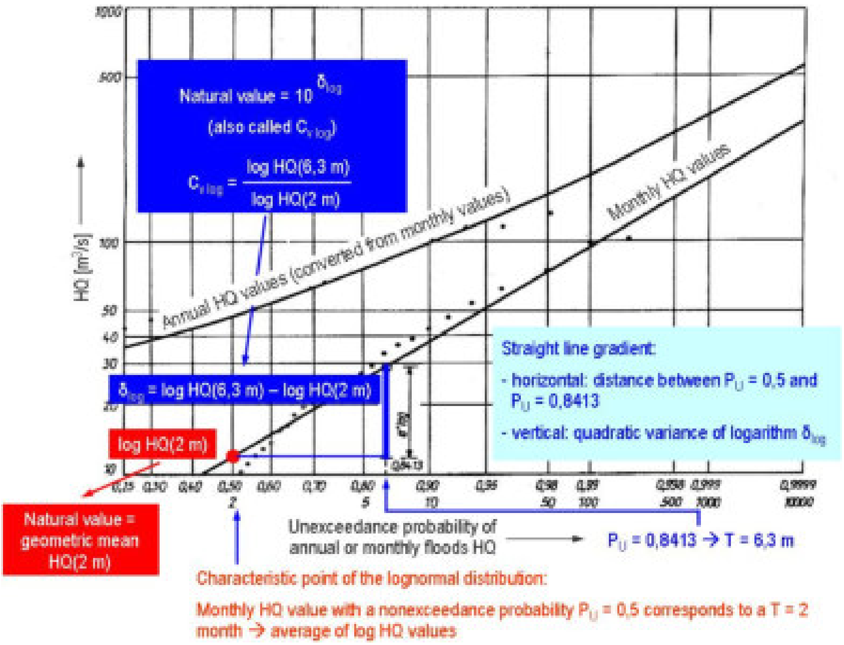


Fig. 4.14:

Monthly HQ values in log-normal distribution diagram (after LAUTERBACH, GLOS*, 1965)*

- parameter determination of lognormal distribution → here: point HQ(2m) and slope of the line ($C_{v\,log}$) in lognormal distribution diagram → see figure 4.14
- estimation of any HQ(m) or HQ(a)
- result transmission to unobserved areas

▹ ***methodology:***

- substitution of HQ(2m) and $C_{v\,log}$ by easily identifiable catchment parameters → application of empirical equations for HQ(2 m) and $C_{v\,log}$
- empirical equationto calculate the geometric mean of the monthly HQ values HQ(2m):

$$HQ(2m) = \frac{10^{\alpha} * I^{0.20} * A_E{}^{0.86}}{(L^2 / A_E)^{0.10} * 10^{\Delta C3} * 10^3} \qquad (4.19)$$

where: HQ(2m) - geometric mean of the monthly HQ values [m³/s]
α - local factor → see equation 4.21
I - catchment gradient [m/m]
A_E - catchment area [km²]
L - length of river [km]
ΔC_3 - correction term for retention areas → see equation 4.20

where:

$$\Delta C_3 = 0.31 * \frac{A_R}{A_E} - 0.031 \qquad (4.20)$$

where: A_R - retention influenced areas → Catchments above lakes, reservoirs and ponds [km²]
(all other variables → see equation 4.19)

- local factor α → consideration of the geographic region (lowland … uplands) and the pedological and geological features in the catchment area
- identification of α on the basis of gauge measurements:

$$\alpha = \log HQ(2m) + 0.14 \log A_E - 0.2 \log I + 0.1 \log (L^2/A_E) + \Delta C_3 \qquad (4.21)$$

(all variables → see equation 4.19)

→ local factor α = f (HQ(2m), morphometric parameter

- empirical equation to calculate the slope of the straight line in lognormal distribution diagram:

$$C_{v\,log} = \beta - 5.9\,\alpha - 0.22 \log A_E \qquad (4.22)$$

where: $C_{v\,log}$ - slope of the straight line in lognormal distribution diagram
α - local factor
β - local factor
A_E - catchment area [km^2]

- identification of β analogous to local factor α on the basis of gauge measurements:

$$\beta = HQ(2m) / HQ(6.3m) + 0.22 \log A_E + 5.9\,\alpha \qquad (4.23)$$

(all variables → see equation 4.19)

→ local factor β = f (HQ(2m, 6.3m), morphometric parameter)

- mapping of local factors α and β → isolines of α and β → for East Germany regionwide available, example → see figure 4.15
- estimation of any HQ(m) or HQ(a):

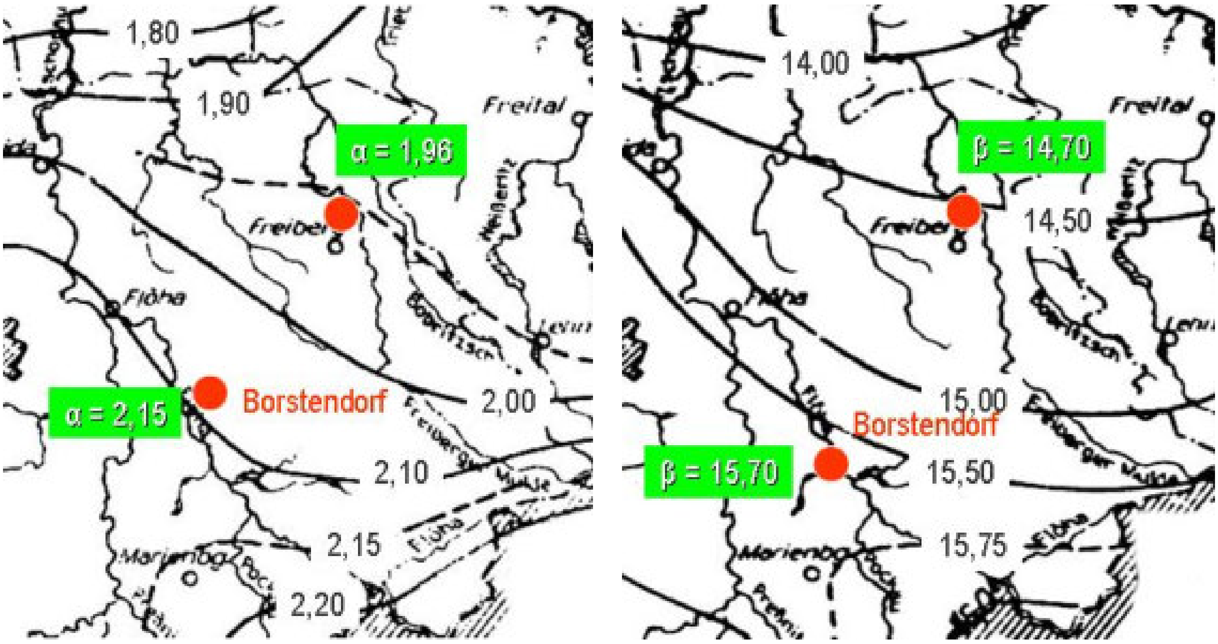
$$HQ(m) \text{ or } HQ(a) = HQ(2m) * (C_{v\,log})^u \qquad (4.24)$$

where: HQ(m) - flood peak discharge with a return period in month [m^3/s]
HQ(a) - flood peak discharge with a return period in years [m^3/s]
HQ(2m) - geometric mean of monthly HQ values [m^3/s]
$C_{v\,log}$ - slope of the straight line in lognormal distribution diagram
u - argument of GAUSSIAN error integral → special tables for different return periods → u different for the calculation of HQ(m) and HQ(a) → example for annual return periods see Table 4.7

Table 4.7: Argument of GAUSSIAN error integral u for different return periods T [y]

Return period T [y]	2	5	10	20	25	50	100
Argument of GAUSSIAN error integral [ ]	1.732	2.128	2.394	2.638	2.713	2.935	3.144

- ***application of Lauerbach-Glos method for the estimation of HQ(100) values of river Flöha and stream Münzbach:***
  - example 1: observed Flöha catchment up to gauge Borstendorf (characteristics → see above):
    - → catchment area $A_E = 641$ km^2, from it retention influenced: $A_R = 109$ km^2
    - → river length $L = 46$ km
    - → catchment slope $I = 0.12$ m/m
    - → correction term for retention areas $\Delta C_3 = 0.022$ (from equation 4.20)
    - → local factor $\alpha = 2.15$ (from equation 4.21 and figure 4.15 a)
    - → geometric mean of monthly HQ values HQ(2m) = 20.2 m^3/s (from equation 4.19)
    - → local factor $\beta = 15.70$ (from equation 4.23 and figure 4.15 b)
    - → straight line slope in lognormal distribution diagram: $C_{v\,\log} = 2.40$ (from equation 4.22)
    - → HQ(100) from equation 4.24: HQ(100) = 316 m^3/s
    - → good comparability with stochastically determined values (see above)
  - example 2: unobserved Münzbach catchment (characteristics → see above):
    - → catchment area $A_E = 150$ km^2, from it retention influenced: $A_R = 40$ km^2
    - → stream length $L = 22$ km
    - → catchment slope $I = 0.14$ m/m
    - → correction term for retention areas $\Delta C_3 = 0.052$ (from equation 4.20)
    - → local factor $\alpha = 1.96$ (from figure 4.15 a)
    - → geometric mean of monthly HQ values HQ(2m) = 3.8 m^3/s (from equation 4.19)
    - → local factor $\beta = 14.55$ (from equation 4.23)
    - → straight line slope in lognormal distribution diagram: $C_{v\,\log} = 2.51$ (from equation 4.22)
    - → HQ(100) from equation 4.24: HQ(100) = 68 m^3/s
    - → because there is no gauge → no assessment possible



a) Local factor α b) Local factor β

Fig. 4.15: Local factors of Lauterbach-Glos method for both catchments (image basis: Lauterbach, Glos, 1965)

- practical application of the regionalization methods → see seminars 18 and 19

4.3.4.3. Rainfall-runoff models

* **aims:**

- detection / quantification of the hydrological processes runoff formation, runoff concentration and discharge routing by means of mathematical approaches
- Main purpose: transformation of the main input variable rainfall into the output variables runoff and discharge → description of these models therefore: rainfall-runoff models
- principle process indicated diagram → see figure 4.16

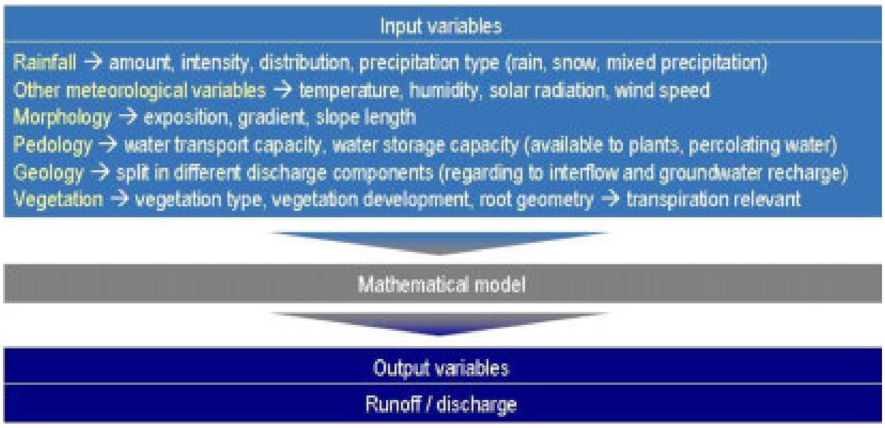


Fig. 4.16:

Significant input and output parameters of a rainfall-runoff model

* **methodology:**

- at present two ways of rainfall-runoff transformation (see also Figure 4.17):
  - → continuous modeling of the rainfall-runoff process over a long (usually several months or several years) period → event differentiated and spatially distributed rainfall-runoff model
  - → modelling of single storm events (event-related) for example in case of design work → conceptual rainfall-runoff model

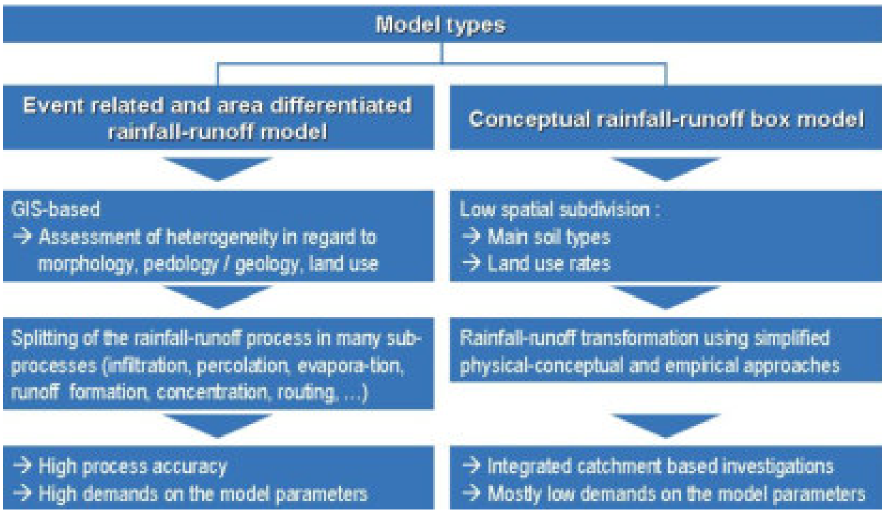


Fig. 4.17:

Types of rainfall-runoff models

→ Depending on the task and the available data, both approaches have their justification.

- strategy to the transformation of rainfall into runoff / discharge:
  - → consideration of the hydrological processes which are necessary for runoff estimation (precipitation, evaporation, storage change)
  - → modelling of runoff formation (effective precipitation, direct runoff)
  - → simulation of flood hydrograph (temporal behaviour of a flood event)
  - → superposition of flood waves from different sub-catchments (significant especially in a large catchment areas and in widely branched river networks) → so-called routing

* **example of an event related and area differentiated rainfall-runoff model – model WaSiM-ETH:**

▹ ***model contents:***

- developed at the ETH Zurich,
- in Germany, Austria and Switzerland often used
- suitable for studies on the discharge behavior in the past, in connection with planning procects and for the quantification of possible impacts of land use and climate changes in the future
- structure of the model → see figure 4.18
- model input and output variables → see table 4.8

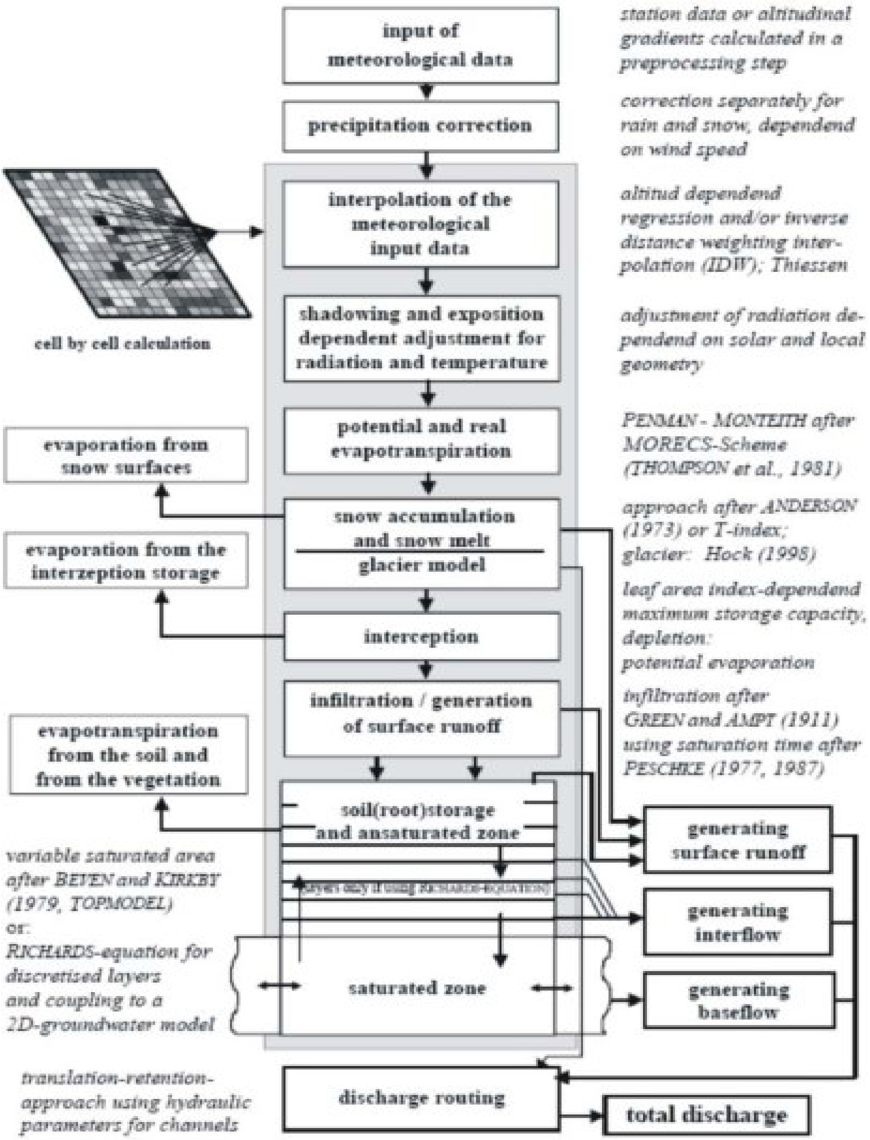


Fig. 4.18:

*Model structure of the rainfall-runoff model WaSiM-ETH (**SCHULLA, 2007)***

Table 4.8: Input and output variables of rainfall-runoff model WaSiM-ETH

Model input variables		
Morphologic-morphometric parameter	*Land use and soil parameter*	*Meteorological data*
- Catchment area - River routing - Catchment slope - Expositions → from DGM	- Land use type (meadow, agriculture, forest, town, …) - Soil parameters for water transport (k_f-values) and for water storage (saturation, field capacity, wilting point)→ from digital soil and land use maps	Time discretization: 1 min - 1 d - Precipitation Time discretization: 1 d - Air temperature - Humidity - Solar radiation - Sunshine duration - Wind speed
Model output values		
- Water balance (precipitation, evaporation, runoff, ...) - Discharge hydrograph: → peak discharge during flood events → flood rise time → discharge sum		

- ***example of model application - studies in the catchment of the river Wilde Weißeritz (Saxony):***

  main aim:

  - quantification of effects of land use changes and climate changes on the runoff behavior in the study area
  - location of the study area → see figure 4.19

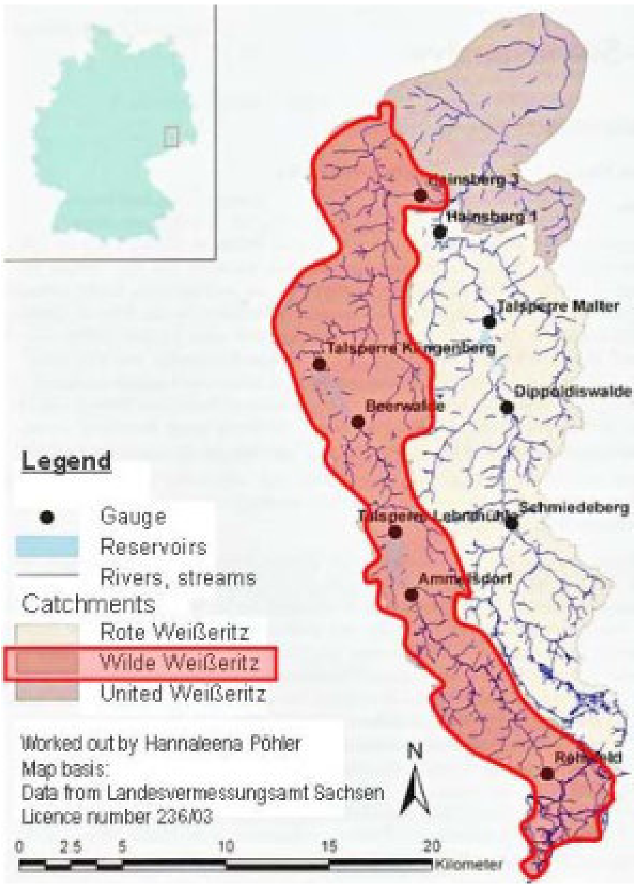


Fig. 4.19:

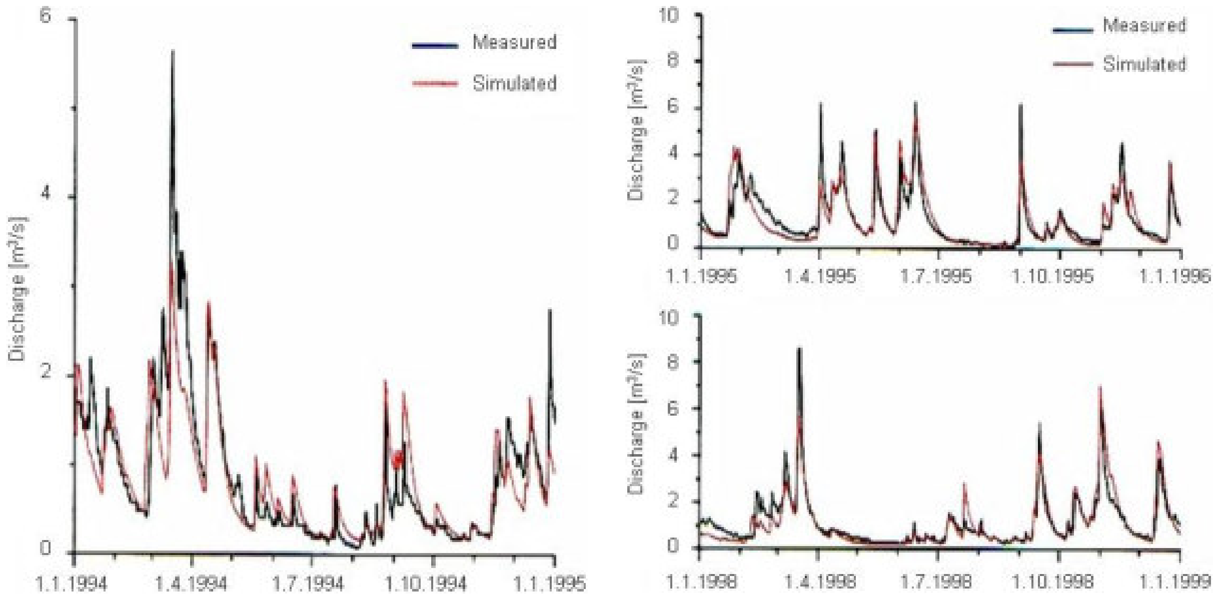
Catchment of the river Wilde Weißeritz (after PÖHLER, 2006)

Model parametrization (pre-processing):

- preparation of information on morphology, pedology, land use and meteorology:
  - → digital elevation model DEM
  - → land use map
  - → soil map
  - → location of climate stations
- climate data (long-period data series of the past and climate scenarios for the future)

Model calibration present state:

- exemplary on gauge Ammelsdorf (location → see figure 4.19)
- selection of a calibration period
- selection criteria:
  - → periods with different hydrological conditions, for example snow melt periods, dry periods, heavy rain events, …
  - → here: selection of the year 1994
- calibration model run with initial parameter set
- testing of the model results (in this case discharges) on the basis of measured discharges
- modification of uncertain model parameters in plausible limits till the differences between simulated and measured discharges → minimum
- calibration results for gauge Ammelsdorf (year 1994) → see figure 4.20 a
- coefficient of determination $R^2 = 0.71$
- overall, a successful model calibration (only snow melt period in late March / early April is not optimal simulated by the model)



a) Model calibration b) Model validation

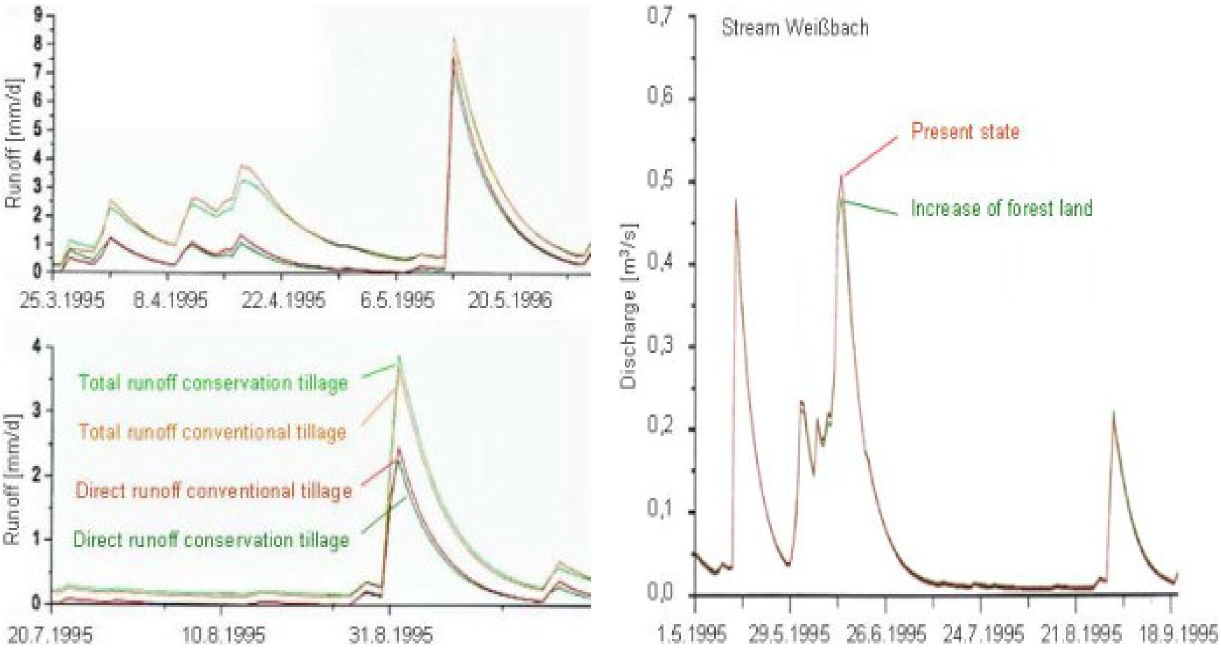
Fig. 4.20: Results of model calibration and validation (model Wasim-ETH) for the gauge Ammelsdorf (river Wilde Weißeritz), after POEHLER *(2006)*

Model validation present state:

- validation = check of the truth content → here: What is the truth of the calibrated model for 1994, outside the calibration period, without changing other parameters?
- selection of the validation period → here period 1991 – 2000 (except 1994)
- results exemplary for the wet year 1995 and the dry year 1998 → see figure 4.20 b
- validation results:
  → R^2 for 1995: 0.69, for 1998: 0.78
  → R^2 for the total validation period: 0.64
- validated model for the present state → premise for scenarios studies on land use and climate changes

effects of land use changes:

- model results for conventional / conservation (non-plough) tillage → see figure 4.21 a



a) Type of agricultural tillage b) Increase of forestland

Fig 4.21: Results of model calculations on the influence of land use changes (after POEHLER*, 2006)*

- low influence of tillage on runoff
- Slightly smaller direct runoff in case of conservation tillage
- total runoff regarding behaviour differentiated:
  → higher infiltration rates in the case of conservation tillage
  → higher total runoff
  → on the other hand higher evaporation amounts (the higher the infiltration rates)
- influence of forestland increase marginal → reason: in the study area only a few areas of forestland exist

impacts of climate change:

- boundary conditions:
  - → selected climate model: WEREX (so-called ENKE model)
  - → model period: 2021 – 2050
  - → land use: analogue to present state
  - → in total 10 WEREX realizations

- results of the rainfall-runoff model:
  - → simulated total runoff for United Weißeritz on the basis of 10 WEREX realizations → see figure 4.22
  - → tendential runoff decrease (especially in spring, = period with the greatest climatic changes in future)

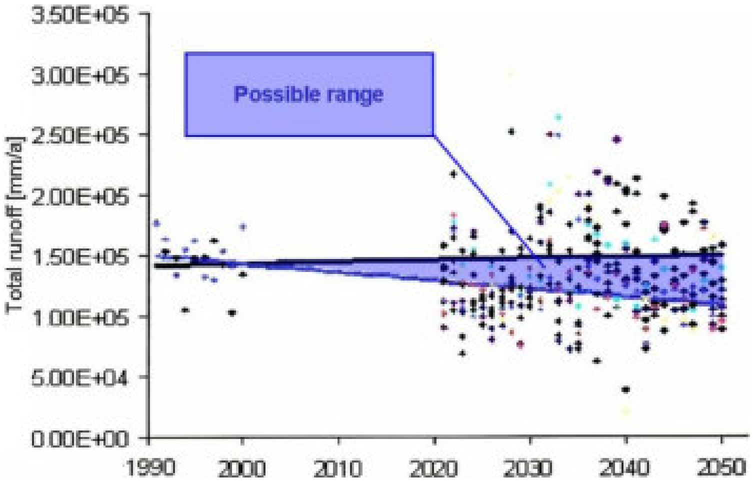


Fig. 4.22:

Simulated total runoff for United Weißeritz on basis of 10 WEREX realizations (after PÖHLER, 2006)

conclusion:

goal (quantification of the impacts of land use and climate changes on runoff behaviour) satisfiable by means of event related and area differentiated rainfall-runoff model only

* **further in Germany usual event related and area differentiated rainfall-runoff models:**

- NASIM (company Hydrotec Aachen)
- ArcEGMO (community development of the Office of Applied Hydrology Berlin and the Potsdam Institute for Climate Impact Research (PIK)

* **example for a conceptual single event related box model – design hydrograph model HQBEMESS:**

▹ ***ambition, model structure and essential model approaches:***

- goal setting: hydrograph modelling on basis of design rainfall and catchment parameters
- model structure and essential model approaches → see figure 4.23

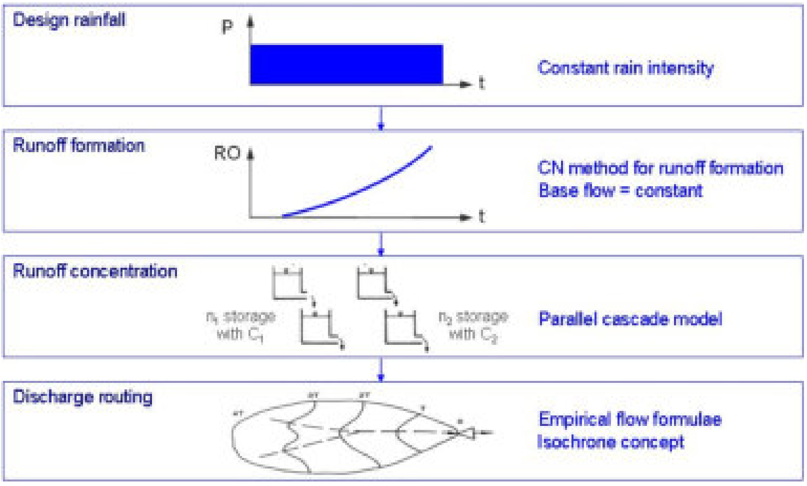


Fig. 4.23:

Model HQBEMESS – model structure and essential model approaches

- ***Mode results:***
  - essential model results of a conceptual, single event related box model (here HQBEMESS) in comparison to a event related and area differentiated rainfall-runoff model (here WaSiM-ETH) → see figure 4.24

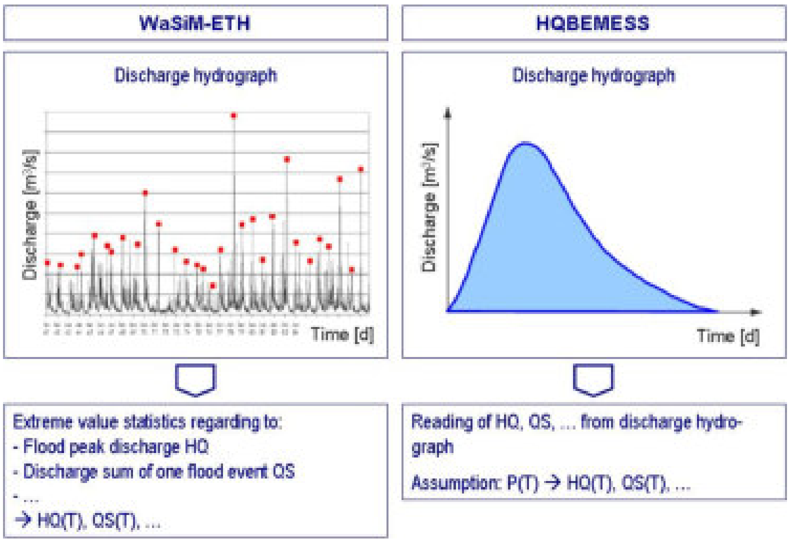


Fig. 4.24:

Model results of a conceptual, single event related box model (here HQBEMESS) in comparison to a event related and area differentiated rainfall-runoff model (here WaSiM-ETH)

- ***application example 1: fictional example→ changes in flood discharges due to urban development:***
  - determining the design flood with a return period of 5 years for an unobserved, uninfluenced catchment in Saxony (rainfall intensity: 106 l / s ha)
  - assessment of changes regarding flood hazard caused by a planned urban development (business park)
  - design of a flood retention basin (RHB) to compensate the changes regarding flood hazards due to urban construction

Area characteristics:

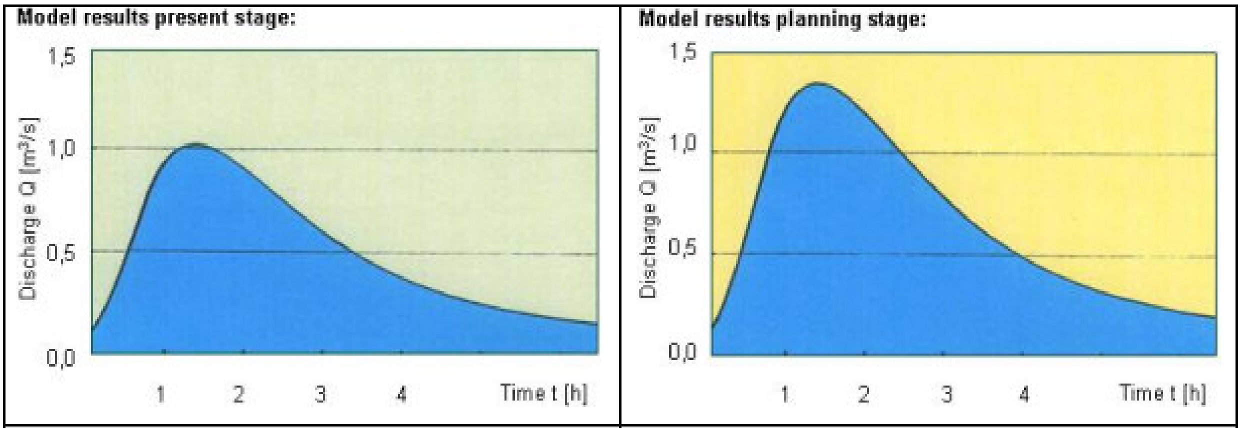
- catchment characteristics → see figure 4.9

Table 4.9: Area characteristics before and after realization of a planned partial urban development

Characteristics, parameter	Present stage (before development)	Planning stage (after development)
Landscape character	Hilly countryside	
Catchment area	2.8 km^2	
Longest flow path from watershed to design point	1.6 km	
Average catchment slope	3.0 %	
Predominant soil type	Sand (with high infiltration capacity)	
Total length of all streams and rivers	2.6 km	2.2 km
Land use percentages:		
- Permanent meadows	15 %	0 %
- Forest	83 %	83 %
- Field and forest trails	2 %	2 %
- Constructed areas	0 %	15 %

Model results:

- HQBEMESS model results for present and planned stages → see figure 4.25



Runoff coefficient: 18 % Discharge maximum: 1.02 m^3/s Discharge sum: 15 350 m^3	Runoff coefficient: 24 % Discharge maximum: 1.33 m^3/s Discharge sum: 19 420 m^3

Fig 4.25: Design model results by means of model HQBEMESS

- flood peak increasing because of land use changes by approximately 30 % (discharge sum: by about 27 %)
- looking for compensatory measures:
  → compensatory measures in many ways possible:
  construction of decentralised infiltration systems
  runoff reduction of sealed areas (for example grass paver, green roofing)
  retention of rain water (use of rainwater)
  retention of surface runoff from flood retention basins → see chapter 6.5.4

  → here: design of a stormwater retention reservoir (RHB) in the case of urban construction

  → hydrological effect of this measure → see figure 4.26

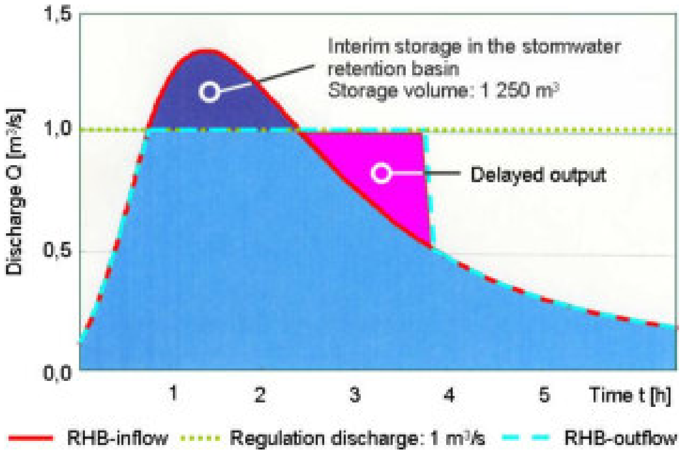


Fig. 4.26:

Design of a stormwater retention basin for compensation of the increased flood discharge peak

- ***application example 2: calculation of flood peak discharges for design of a bridge near the village Frießnitz (Thuringia):***

processing step 1: detection of the drainage systems

- detection of the drainage system = estimation of courses of streams, identification of the watershed, estimation of nodal points (confluences of two or more streams) → basis for a further division into sub-catchments and identification of flow directions → see figure 4.27
- in the example 2 sub-catchments

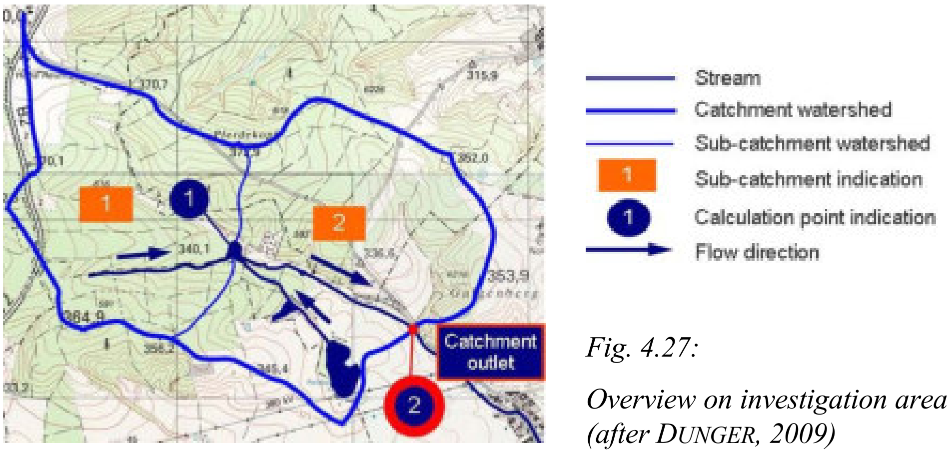


Fig. 4.27:

Overview on investigation area with 2 sub-catchments (after Dunger, 2009)

processing step 1: detection of morphologic parameters for each sub-catchment

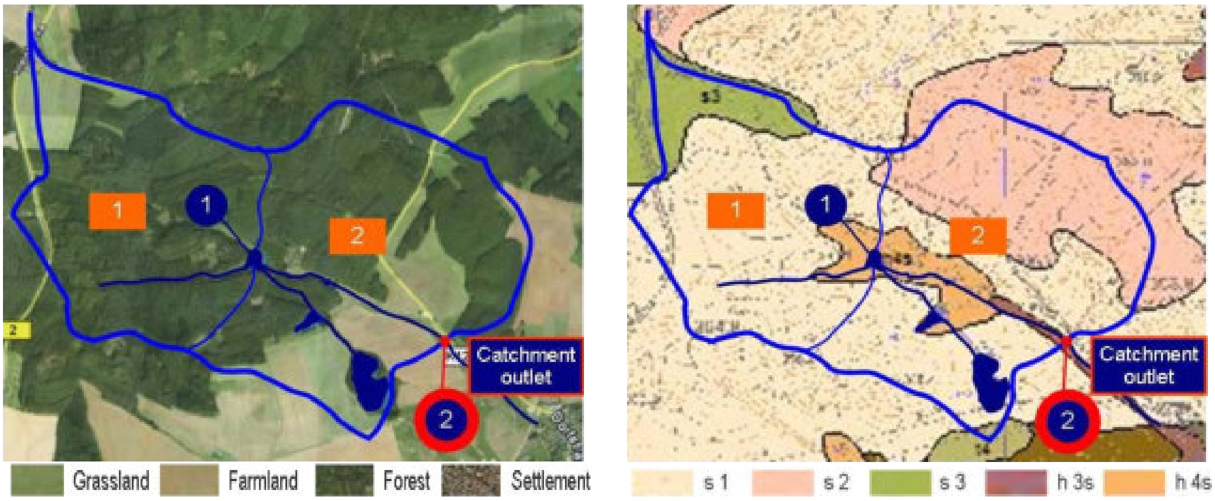
- morphologic parameters → influencing runoff formation and concentration
- area sizes, flow length, slope values
- Values for the two sub-catchments → see table 4.10

Table 4.10: Morphologic parameters of the two sub-catchments

Sub-catchment	1	2
Sub-catchment area [km^2]	1.38	1.89
Maximum flow path highest / lowest point [km]	2.0	1.8
Length of all non-periodic streams [km]	0.9	1.9
Height difference highest / lowest point [m]	64	68

processing step 3: hydrologic characterization of the primary land uses and soil types

- primary land uses and soil types → essential for the runoff formation process (influencing the infiltration process)
- primary land uses and soil types for the two sub-catchments → see figure 4.28, specification → tables 4.11 and 4.12



a) Primary land use types

b) Primary soil types

Fig. 4.28: ***Primary land uses and soil types*** *in the investigation area (after DUNGER, 2009)*

Table 4.11: Land use proportions in the two sub-catchments

Land use proportions Sub-catchment	1	2
- Root crops, wine	0	2
- Cereals, forage crops	0	7
- Permanent Pasture	17	19
- Forest (scattered)	5	4
- Forest (medium density)	76	65
- Farm road (surfaced)	1	1
- Sealed surfaces	1	2

Table 4.12: Soil characterisation in the two sub-catchments

Designation	Characteristics	Soil type after US-SCS
s 1	Sandy loam (sediments of the Lower Buntsandstein)	2
s 2	Loamy sand (sediments of the Middle Buntsandstein)	2
s 3	Sand, loamy - gley (Upper / Middle Buntsandstein sediments)	3
h 3s	Sand to sandy loam - vega (side valleys)	3
h 4s	Sand, loamy – moor gley (Buntsandstein weathering material)	4

processing step 4: determination of the design rainfall

- Basis: in this investigations KOSTRA Atlas
- design return period: 5 and 25 y (regulatory requirement)
- rainfall amounts by KOSTRA Atlas: 42.2 mm for T = 5 a or 54.6 mm for T = 25 a

processing step5: estimation of runoff formation

- in HQBEMESS used method for determination of surface runoff: Curve Number method
- Modeled surface runoff RO:
  - → sub-catchment 1: RO = 6.5 mm for T = 5 a, RO = 10.6 mm for T = 25 a
  - → sub-catchment 2: RO = 5.6 mm for T = 5 a, RO = 10.1 mm for T = 25 a

processing step 6: estimation of runoff concentration

- possible methods: U.S. SCS triangular hydrograph, storage cascade model, …
- transformation of the formatted runoff into a discharge hydrograph → determination of the relevant parameters flood discharge peak HQ, flood rise time t_S, duration of flood event, …
- used in HQBEMESS: storage cascade model
- modeled peak flow discharges HQ (separately for each sub-catchment, it means for sub-catchment 2 without superposition with the flow wave from the sub-catchment 1):
  - → sub-catchment 1: HQ = 258 l/s for T = 5 a, HQ = 475 l/s for T = 25 a
  - → sub-catchment 2: HQ = 360 l/s for T = 5 a, HQ = 667 l/s for T = 25 a

processing step 7: estimation of discharge routing

- superposition of waves, formatted and concentrated in different sub-catchments → not additive, but by taking into account runtime delays

- here: superposition of the discharge curves (waves) of sub-catchments 1 and 2:
  - → total discharge hydrograph at the point 2 = discharge hydrograph of the sub-catchment 2 + run-time corrected discharge hydrograph of sub-catchment 1
  - → time correction by application of path-time law: $\Delta t = s / v$
  - → s → set (stream length between points 1 and 2)), here: 1.1 km
  - → v → estimation for example the application of the empirical MANNING & STRICKLER flow formula (see chapter 5.3, lecture notes “Hydrology I” or chapter 1, lecture notes “Hydrology III) for given stream dimensions and longitudinal stream gradient

- here: v = 2.0 m/s → Δt = 1 100 m / 2.0 m/s = 550 s → discharge wave needs 550 s (about 9 min) from point 1 to point 2

- Results of run-time corrected wave superposition at point 2:
  - → HQ = 617 l/s for T = 5 a
  - → HQ = 1 139 l/s for T = 25 a

- discharge hydrographs for both return periods at catchment outlet → see figure 4.29

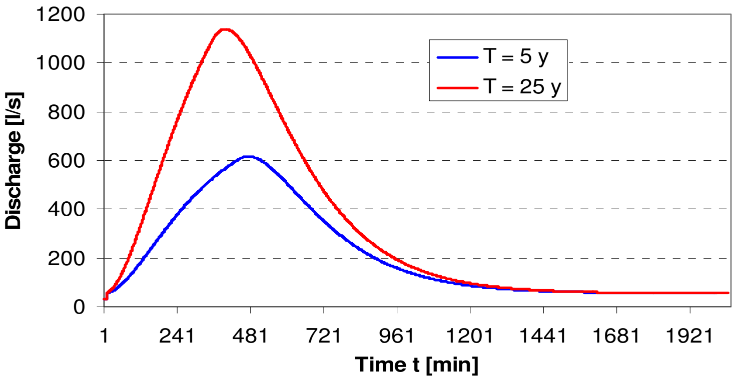


Fig. 4.29:

Discharge hydrographs for both return periods at catchment outlet (after DUNGER, 2009)

- practical application of the rainfall-runoff model HQBEMESS → see seminars 21 to 26 and 30

4.3.5. Opportunities and limitations of flood calculation methods

- Summary overview of the advantages and disadvantages, opportunities and limits of the methods which are documented in chapters 4.3.2 to 4.3.4 → see table 4.13

Table 4.13: Opportunities and limitations of various methods of flood estimation (after ***Barben et al.., 2001)***

Empirical and regionally vaild formulas	Statistical methods	Deterministic models
- Poor comparability of the data derivation - Problems in transnational data delivery - Input data have to be estimated from old maps original literature, which are often not reproducible available in authorities. Thereby exists no general availability of input data and calculation formulas. - Questions of plausibility, that results from different observation duration of measurement series - Data collection (problems in the measurements)	- Nonstationarity of the series → to short time series → to long time series - Postprocessing of series, check of single values - The influence of historical values is difficult to estimate. - Data collection (problems in the measurements): → No reliability of water level - discharge relationships and measurements → Uncertainty in the measurements → Device failure at high discharges → Occasional downtime and completion by adjacent gauges	- Estimation of river cross-sections and roughness - Difficult plausibility control in catchments without a gauge - Problems with the regionalization of model parameters - Data availability (problems in the provision): → To low station density for the measurements of precipitation and other meteorological features → Poor availability of hourly data (precipitations, discharges) → Incorrect data

- conclusions:
  - → There is no universally applicable method.
  - → In dependence on the task (and therefore accuracy demand) and data availability different methods are successful.
  - → still large uncertainties in the case of unobserved catchments

4.4. Flood protection

4.4.1. Definition and components of flood protection

* **definition:**

- flood protection = flood control = sum of all measures to protect the public or of property from flood

* **components of flood protection (so-called three-pillar concept):**

- natural retention
- technical measures
- flood prediction

→ focus: use of natural retention → natural storage capacity in the catchment area

4.4.2. Natural retention

* **content:**

- Rural measures for the use of its water retention capacity of catchment areas, especially by means of:
  - a) runoff reduction:
    → unsealing of areas
    → decentralized infiltration systems
    → erosion protection strip
    → increase of forest areas
    → perennial vegetation
    → conservative tillage
  - b) flood plains (wetlands, grasslands)
  - c) mining activities (mainly using opencast mining residual holes)
- main area of flood protection

* **examples:**

► *erosion protection strip:*

- small structures
- hedges from the perspective of erosion control not parallel to the slope, but hedge curve adapted to the terrain morphology (+/-in the longitudinal direction of the slope) → minimization of erosion
- in combination with buffer strips

► *forest area increase:*

- forest → equalization of discharge behavior
- in Saxony in reaction of flood event in 2002 is planned: increase of forest areas from 28 to 30 %
- effective in the flood formation regions for small and medium floods
- little or no effect at very large flood events

► *perennial vegetation:*

- intercropping → currently mainly clover, herb plants, mustard and vegetable oil
- extensive grassland → from hydrological point of view advantages in comparison to intensive agriculture:
  - → little wheeling (mowing once a year, no fertilizer)
  - → barely grazing
  - → reduced soil compaction
  - → reduced surface runoff formation

► *conservation tillage:*

- continuance of crop residues on the field
- all year round high ground cover
- reduction of surface runoff formation and erosion

► *usage of flood plains:*

- mostly wetlands and grasslands
- larger flow cross-section compared to diked rivers
- deceleration of the displacement velocity of flood waves because of increased hydraulic roughness

► *mining activities:*

- using opencast mining residual holes as reservoirs during flood periods (for example mining residual holes, other retention areas, historic mining equipment
- use of underground mining facilities

4.4.3. Engineering flood protection

* **content:**

- technical measures = constructions
- upper reaches → retention reservoirs, dams
- middle reaches → river development, flood reliefs (for example flood control channels)
- lower reaches → dikes, polders

- primarily as an extension of the flood protection mainly at river narrows and in urban regions

* **examples:**

► *retention reservoirs:*

- single purpose reservoir → serve the flood protection only
- filled in case of flood event
- otherwise empty or partially filled (wetland)

► *dams:*

- multi-purpose reservoir → not only serve the flood protection
- Additional functions: water supply, discharge increase in case of low flow, fisheries, recreation, ...
- details → see reservoir management (chapter 6)

► *river development:*

- embankment → classic engineer technical river development → noted numerous (side) effects

- ecological river restoration → ecological river deconstruction

- rehabilitation of cu-off meander

- enlargement the flow cross-section

- near-natural bank reinforcement

- in case of temporal river construction projects at river bank: if possible construction at one bank only (the second bank with a time lag of several years)

- extension or reinforcement of river narrows

- flood control channels → practiced mainly in large urban areas

► *dikes:*

- flood protection at river narrows
- note damages in case of floods above the design flood level
- dike slots: operational flood protection measure → increase of the retention area
- dike relocating → extraction of flood control area → in most cases flooding of former polder areas (polder = after embankment reclaimed land)

4.4.4. Flood precaution

* **content:**

- aim: minimization of flood damages by risk and damage analysis
- territorial planning → floodplains, development plans
- building precautions (type, material)
- behavioural precaution → inform the population, flood prediction, flood information services, disaster management
- risk precaution → insurance, cost analysis

* **examples:**

► *territorial planning:*

- designation of flood plain areas → use of information of floods in the past
- maps with floodplains at different flood probabilities or return periods
- flood hazard maps → no risk … high risk
- flood consideration in development plans → no development in flood prone areas

► *building precautions:*

- used especially in potential flood areas

- Strategies:
    - → evasion: building on an elevated site and / or shielding of buildings
    - → resist: sealing and / or amplification of the basement and foundation → if necessary note buoyancy
    - → yielding: appropriate use and / or furniture of the flood prone floors
    - → ensure: Protection against contamination of the building and the environment from pollutants

► *inform the population:*

- clarify the real risks and hazards
- make clear that any measures do not provide 100% protection
- After the flood is before the flood!

► *flood prediction, flood information services, disaster management:*

- steps of a flood forecast → see figure 4.30
- strict organization of flood information services
- clear distinction of competencies

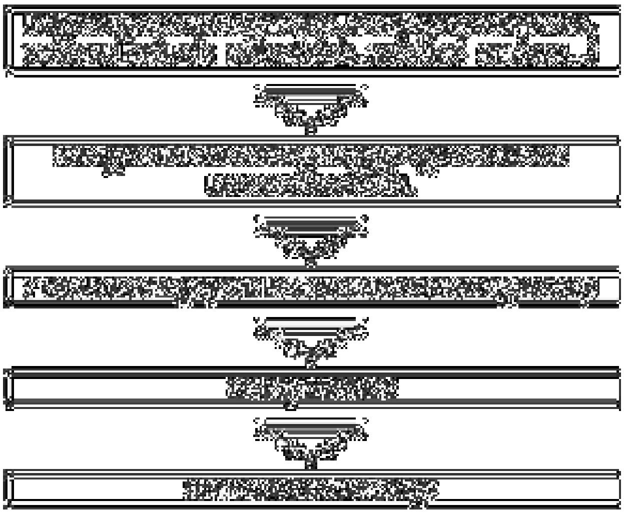


Fig 4.30:

Steps of a flood forecast

- Alarm levels in the flood information services:
  - → activation of alarm levels: if water level at flood gauge reaches the fixed alarm level and further increases are expected
  - → alert stage 1 – message service: at the beginning of the proliferation
    - continuous analysis of meteorological and hydrological situation
    - assessment of trends
    - Verification of flood defense plans, equipment / technology and materials
  - → alert stage 2 – control service: in case of diked rivers → flooding up to the dike bottom or in case of flow obstruction because of icing
    - daily periodic inspections of the waterways, dikes, water management systems, ...
    - removing of barriers
    - inspection of operational readiness of the disaster relief forces
    - alerting of the local water weirs
  - → alert stage 3 – sentry: in case of diked rivers → flooding up to the half dike height, even in case of dangerous ice jam, flotsam or other hazards
    - permanent sentry service
    - preventive measures on recognized hazard points
    - creation of a separate communication survey
    - transport of material and equipment to critical points
    - arrangement of operational flood protection measures
  - → alert stage 4 – flood defense: in case of expected or real damages → breaking of dikes, dike overflow, massive displacement of ice jam or flotsam, areal runoff due to heavy rain
    - operational control of existing risks
    - evacuation of population
    - outsourcing of economic values
    - damage limitation (for example by setting up a second line of defense)
  - → If necessary the highest alert stage can release without delay.

► *insurance:*

- insurance against damage caused by dynamic processes (through the power of flowing water) and by static processes (due to the high water level)
- importance of private precaution

► *economic feasibility studies for flood protection measures:*

- effective use of resources
- support regarding to decision making of flood protection measures, which should priorize
- details → see chapter 4.4.5

4.4.5. Efficient development of flood control facilities

a) aim:

- optimization of the values damages, costs, benefits
- cooperation of hydrologists with planning agencies, insurance companies, territorial authorities and ecologists
- economic evaluation (damage, cost, benefit) always refers to a specified period (period function) of a flood control facility

b) basis:

- flood return period and duration of a flood protection measure
- specific (mean) damage without flood protection
- cost of the flood protection system
- benefit (additional revenue) due to flood protection system
- methodology is independent of these, if the development is realized on base of water level, flood discharge (dikes) or flood discharge sum (reservoirs)

c) determination of the efficient development of flood control facilities (on example of technical flood protection by means of an dike → flood level as design value):

* **identification of the specific damage ss(x):**

- specific damage ss(x) depends on the high water level and the flood duration and the flood moment (f before or after harvest, during the week or on weekends ...) → The specific damage is to be determine hat a flood with defined flood water level causes in average.
- damage groups:
  - → humans, animals
  - → settlements
  - → industry, agriculture and forestry
  - → transport and water management facilities
  - → consequential damages (loss of production ...)
  - → operational flood control
  - → environmental damages
- sources of information: insurance companies, economists, engineers, ecologists and other

- result:
  - → damage curve ss(x) → example, see Figure 4.31
  - → note: damage curve is subject to dynamic (cost development within the design period, territorial development)

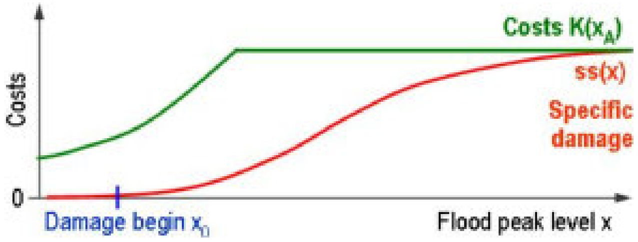


Fig. 4.31:

Curves of the specific damages, costs and additional revenue from flood defenses (after DYCK, ET AL., *1976)*

* Determination of the total damage S_G:

- S_G = Sum of specific damages for the design period n_B <u>without</u> flood protection
- $S_G = f(P_ü, ss(x), n_B)$

← in practice often up to x for $T(x_B) = 10\,000$ y

$$S_G = n_B \int_{x_0}^{\infty} ss(x_i) * P_ü(x_i) \approx n_B \sum_{x_0}^{\infty} ss(x_i) * P_ü(x_i) \qquad (4.25)$$

where S_G - total damage
$P_ü(x_i)$ - exceedance probability of a certain water level x_i
$ss(x_i)$ - specific damage at a certain water level x_i
n_B - design period
x_o - water level at damage begin
$T(x_B)$ - return period for the design water level

- Very large floods provide only a small contribution to the total damage (because $P_ü$ very small).

* calculating the costs of flood protection:

- costs include:
  - → construction cost of the flood protection system
  - → oncosts and maintenance costs resulting during normal operation during the period of use n_B
  - → maintenance costs for equipment damage due to flooding
- results: cost curve $K(x_A)$ for the design water level x_A → see figure 4.31
- Costs increase with increasing design water level x_A in a small degree only, because the construction costs, which have the greatest impact on the cost, almost independent of x_A are generally high.

* calculation of the benefits of flood control:

- benefit $N(x_A)$ = damage reduction S_M + additional revenue R because of flood protection:

$$\mathbf{N(x_A) = S_M(x_A) + R(x_A)} \qquad \mathbf{(4.26)}$$

- damage reduction S_M = total damage S_G – residual damage S_R → Because there is no 100 % flood protection possible, there are taking place damages even after flood development because of very rare floods with water level $x > x_A$ (design water level). → residual damage

- additional revenue: profits due to flood protection because of changed usability of the former flooded areas (industry, agriculture, buildings ...)
- additional revenue often difficult assessable:
  → long-term territorial development over the total operating life is not possible
  → often abrupt increase of ss(x) for x > x_A (due to changes in use)

* **determination of the total advantage and the optimum design size x_{opt}:**

- total advantage V = benefit N - cost K (ascertainable for different x_A → see figure 4.32):

$$V(x_A) = N(x_A) - K(x_A) \tag{4.27}$$

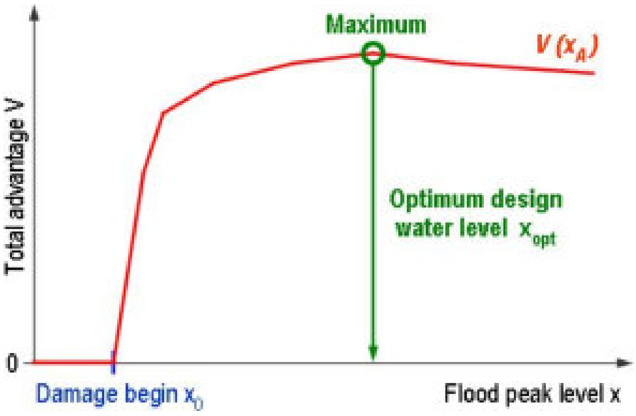


Fig. 4.32:

Total advantage and optimum design level (after DYCK, ET AL., 1976)

* **statements about the flood protection worthiness W_{HW}:**

- determining the cost / benefit ratio for various flood protection measures:

$$W_{HW} = N(x_{opt}) / K(x_{opt}) \tag{4.28}$$

- comparisons of flood protection measures at one location
- decision support regarding prioritization of flood protection measures at various sites

4.4.6. Summarising judgement of success chances of flood protection measures

- natural retention: effective for smaller, frequently occurring floods (up to about 10 years return period))
- technical measures: in most cases not necessary for small floods, effective in the case of medium floods (return period of about 10 up to about 100 years)
- Flood prevention: often the only option in flood-prone areas in case of extreme floods (return periods > 100 years)

→ living with the flood!

4.5. Special literature regarding flood analysis

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5. Low water hydrology

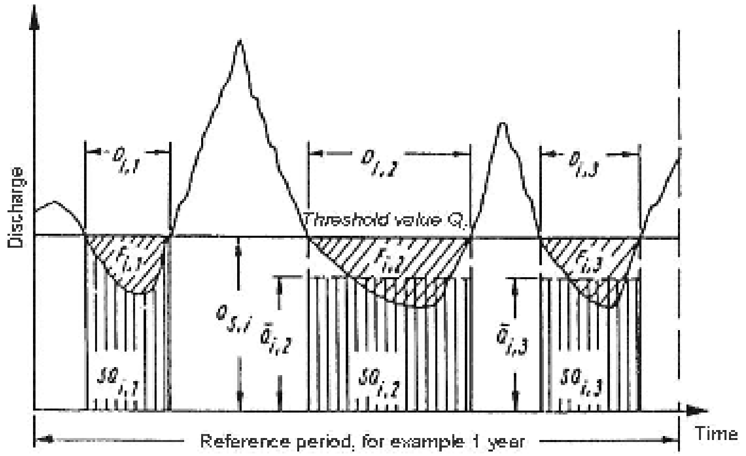
5.1. Importance of low water flow analysis

* **importance of low water analysis:**

- decisive burden period for the water supply (municipal, industrial, agriculture)
- About 25% of public water supply in Germany are realized by surface water (bank filtration, enriched groundwater, river and sea water, dam water).
- important parameter for shipping (water level)
- critical for water quality (reduced dilution effect, disturbance of natural self-purification)

* **low water parameters** (→ see also figure 5.1):

┌─> - low water flow NQ in m^3/s
├─> - duration D of the low-water event in days or months
│ - frequency (return period) of a low water event
├─> - shortage F (deficit) related to a demand value
│ - time interval between the low-water events (deficit backfilling)
│ - more: time of occurrence (growing season, winter ...), spatial extent, low water level
└── main parameters



D -	Low water duration
F -	Shortage during a low water period
Q -	Mean low water discharge
SQ -	Discharge Sum during a low water period

Fig. 5.1:

Characteristic parameters for application of the statistical threshold to a low water flow hydrograph (after D*YCK*, P*ESCHKE, 1995)*

* **distinction between dry period, drought period and low water period:**

▹ ***dry period:***

- meteorological reasons: period of low precipitation P and high potential evapotranspiration ETP → exploitation of water resources → surface water reserves < water demand → water deficiency
- definition of a dry period depends on the climate zone, for example Bali (tropical rain forest zone): t > 6 d, Libya (desert climate): t > 2 a

▹ ***drought period:***

- meteorological causes
- For Central Europe is valid that for 4 consecutive days the temperature maximum is above the long-term average and the humidity at 14:00 clock reaches a maximum of 40 %.

▹ ***low water period:***

- causes:
  → meteorological causes
  → land use and storage properties in the catchment (vegetation, pedology, geology)

- characterization:
  → discharge decrease below a limit (threshold), results from kind and intensity of surface water usage (quantity aspect) as well as from the material loading (quality aspect)
- example: low water discharges of river Elbe → see table 5.1

Table 5.1: Low water discharges of river Elbe, gauge Dresden (since 1874)

Year	Period	NQ [m^3/s]	Water level [cm]	Comments
1874	12.07. - 31.10.	50.8		Because of several dam in Czech Republic at present a minimum low water discharge of 70 m^3/s is guaranteed.
1904	28.07. - 08.09.	57.2		
1911	25.07. - 20.09.	54.3	65	
1934	27.05. - 27.06.	45.9	38	
	27.06. - 30.08.	53.9		
1947	29.07. - 31.10.	35.8	21	
1950	14.08. - 14.09.	51.2		
1952	23.07. - 04.09.	35.8	21	
1954	25.12. - 15.01.	22.5	5	Lowest measured discharge NNQ

- low water process (receding behaviour) in case of comparable meteorological conditions due to the different hydrological, hydrogeological and land use conditions varying considerably (reason: retention capacity) → for example in case of water inflow to a river from thick aquifers receding not as high as in case of poor storage capacity → see lecture "Hydrogeology"

5.2. Estimation of low water probabilities

* **main parameters** (→ see also chapter 5.1)**:**

- discharge NQ, duration D, deficit F
  → theoretical: application of three-dimensional probability distributions
  → but: too complicated, impracticable
  → simplification: reduction of three-dimensional problem to a two-dimensional:
    - constancy of NQ or D and variation of the other parameter (D is often held constant, for example D = 7 d, 15 d, 1 month ... → see figures 5.2 and 5.3)
    - determination of F with the help of the distribution functions of NQ and D

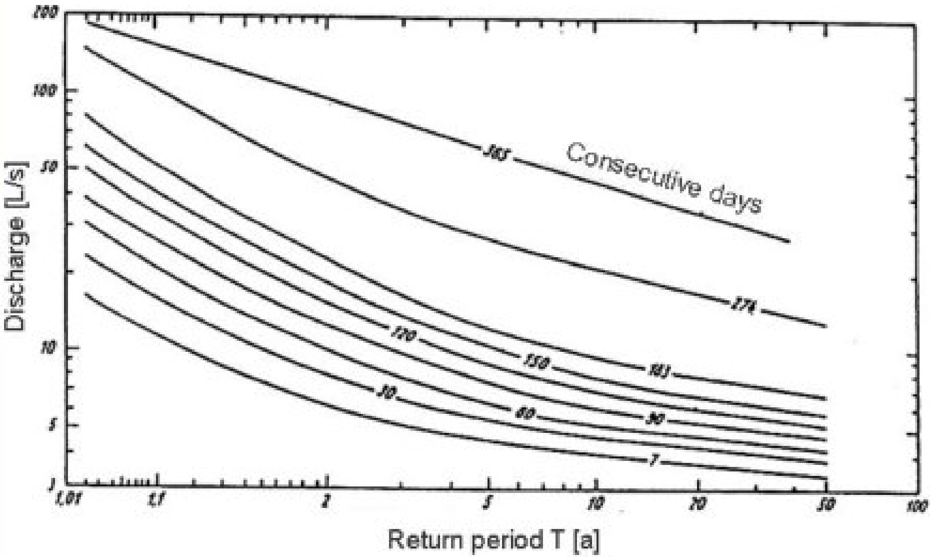


Fig. 5.2:

Low water probabilities of a stream in Tharandt forest (Tharandter Wald, series 1921 – 1959), after DYCK, PESCHKE, 1995)

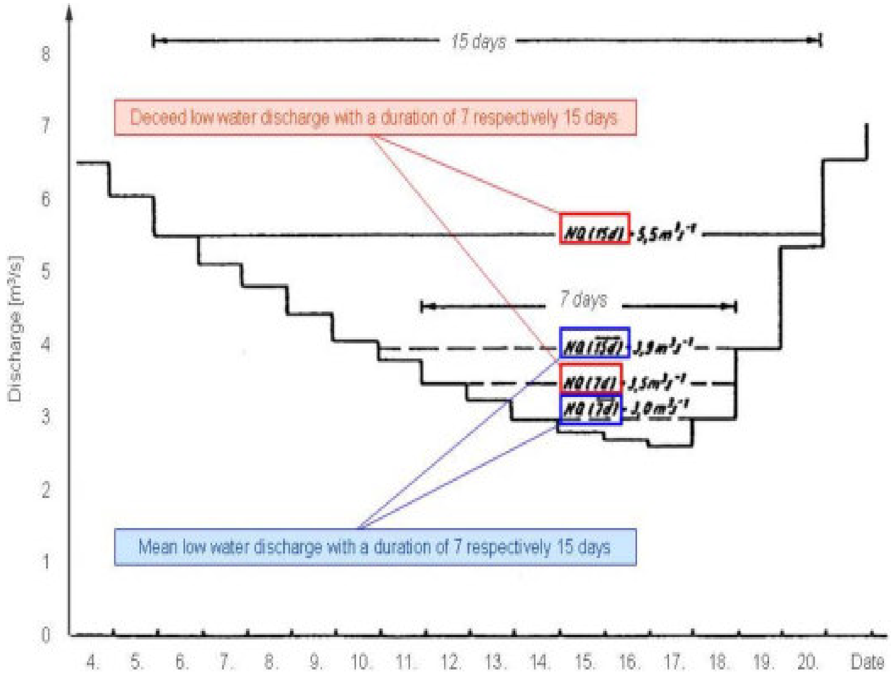


Fig 5.3:

Low-water period with medium and advanced NQ of 7 and 15 days duration (after DYCK, PESCHKE, *1995)*

* **Methodology of low water analysis** at example of minimum monthly average NQ($\overline{1m}$) for gauge Lichtenwalde, river Zschopau, annual series 1921 – 1970:

a) selection of the observation period:

- length of series of observations: preferably > 10 a, here: 1921 – 1970
- source: "Gewässerkundliche Jahrbücher" (German Hydrological Yearbooks)

b) data checking (checking for consistency and homogeneity of the low water collective):

- for low water analysis very important, because there are existing al lot of error sources → see lecture notes "Hydrology I", chapter 5.4.2)
  → alternative: gauge with separate channel for low water flow (small cross-section → see lecture notes "Hydrology I", chapter 5.4.2, especially figure 5.24), in Germany since about 1920 in use → data before 1920 are often inaccurate)

- identification of trends, cyclical behaviour ...

- data checking to statistical independence (the statistical risk of dependence increases with increasing low water duration and in case of aquifers with a high storage capacity)

c) rank of NQ($\overline{1m}$):

- starting with the highest value → see table 5.2)
- order m = 1 ... n (here n = 50)

d) calculation of exeedance probability $P_Ü$:

$$P_Ü = m / (n+1) * 100\ \% \qquad (5.1)$$

where $P_Ü$ - exeedance probability [%]
m - rank of the low water event (m = 1 → maximum NQ, m = n → smallest NQ)
n - total number of low water events

Table 5.2: Collection of low water values $\overline{NQ(1m)}$ (in excerpts of gauge Lichtenwalde, river Zschopau, 1921 - 1970 with exeedance probabilities

Serial No.	$\overline{NQ(1m)}$ [m^3/s]	$P_Ü$ [%]	Lfd. Nr.	$\overline{NQ(1m)}$ [m^3/s]	$P_Ü$ [%]
1	19.4	1.96	44	3.40	86.27
2	14.7	3.92	45	2.90	88.24
3	12.1	5.88	46	2.80	90.20
4	12.1	7.84	47	2.60	92.16
...	...	...	48	2.45	94.12
42	3.62	82.35	49	2.44	96.08
43	3.62	84.31	50	1.68	98.04

e) choice of distribution function, plot of NQ with the corresponding $P_Ü$ in distribution diagram:

- behaviour of low water values is well describable by means of extreme value distribution type III (E III) with lower limit → see figure 5.4
  → is often use in hydrology for low water investigations

f) distribution function adaption:

- free adjustment [illegible]

- analytical fit:
  → parameter estimation
  → iterative estimation improvement possible

- adaption is realized in E III distribution function diagram→ see figure 5.4)

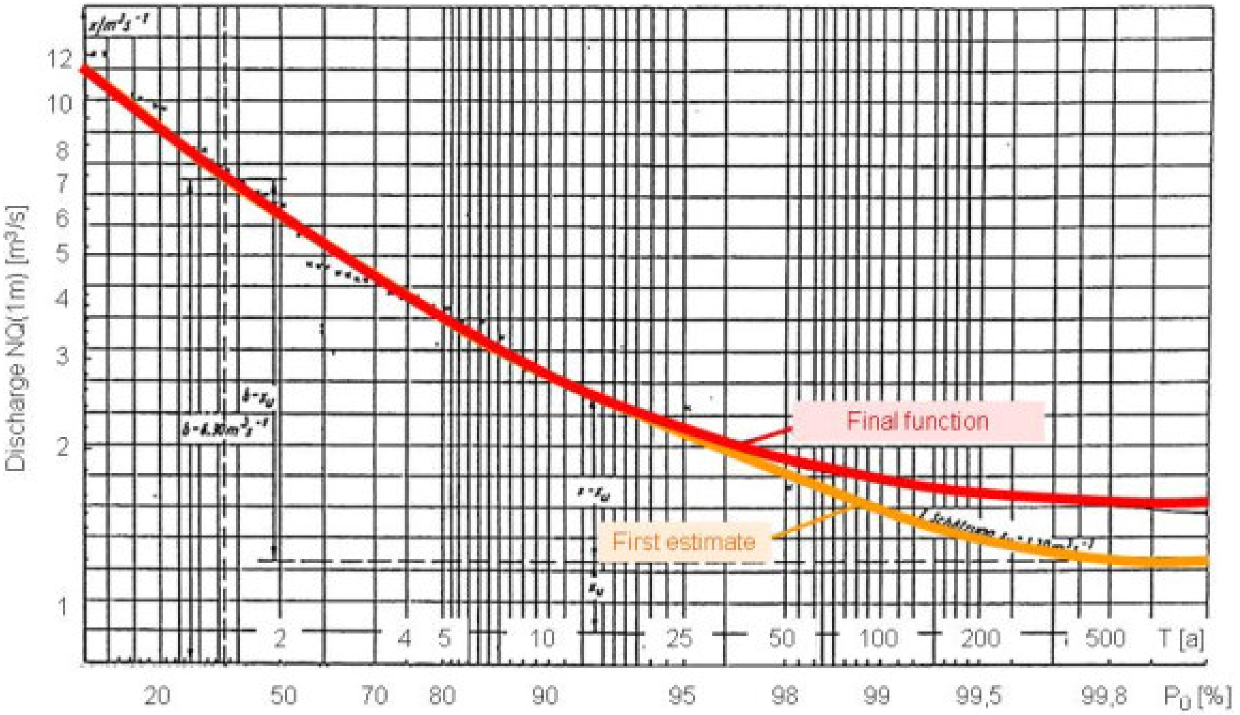


Fig. 5.4: Distribution function of minimum monthly low water discharge means $\overline{NQ(1m)}$ of gauge Lichtenwalde, river Zschopau, 1921 – 1970 (after DYCK, ET AL., 1978)

g) interpretation:

▹ *extrapolation range:*

- for most practical tasks to maximum 50 years limited, because the dynamic development of water management conditions have a greater impact on low water than on floods (for example the development of water intake from rivers or water feeding into rivers)
- extrapolation in case of low water is not as crucial as in floods

▹ *estimation of reliability degree of water supply in a low water period* → see chapter 5.3

5.3. Reliability of water supply during low water periods

- $P_Ü$ may be interpreted as ***reliability degree*** for realization of a specific water demand (example: NQ(1m) = 3,7 m^3/s can be provides with a sureness degree of 80 %, → see figure 5.4)

  → note here, however, the minimum low water discharge from ecological point of view Q_{min}: for example reduces the value mentioned above in case of Q_{min} = 0,5 m^3/s to 3,2 m^3/s

- The low water probabilities are determinable for different low water durations by means the methodology characterized in chapter 5.2. → family of curves (→ see for example figure 5.2)

- For certain sectors of the economy are regulated, how big the ***reliability of supply*** at least has to be (reliability of supply for drinking water > industrial water > irrigation water).

  → reading of the corresponding low water discharge value from the probability distribution function (see figure 5.4)

- low water probability separately for each month ascertainable

  → reliability degree also determinable for single month (for example in dependence on season) → see figure 5.5)

  → important for branches of production with a seasonal varying water demand (for example irrigation) or for those who need to work with high reliability of supply

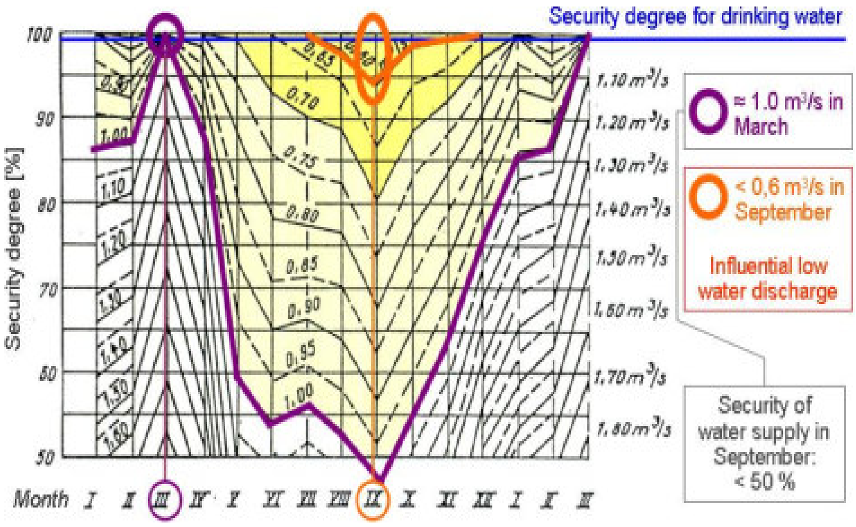


Fig. 5.5:

Reliability degree of water supply in several month for gauge Bad Schandau, river Kirnitzsch, 1912 to 1961 (after D*YCK, ET AL., 1978)*

Application of low water analysis → see seminar 27

5.4. Special literature for low water investigations

Glos, E. und D. Lauterbach (1977):
Regionale Verallgemeinerung von Niedrigwasserdurchflüssen mit Wahrscheinlichkeitsaussage. Mitteilungen des Institutes für Wasserwirtschaft, Heft 37, VEB Verlag für Bauwesen, Berlin

Müller, H.-W. (2002):
Niedrigwasser. Literarion. Utz-Verlag München.

Riggs, H. C. (1972):
Techniques of Water-Resources Investigations of the United States Geological Survey. Book 4: Hydrologic Analysis and Interpretation, Chapter B1: Low-Flow Investigations. United States Government Printing Office, Washington.

6. Hydrologic reservoir planning

6.1. Aims and importance of hydrological reservoir planning

* **main task:**

- bring in accordance water yield and water demand, that means:
  → storing water in times of surplus (during floods → HQ)
  → release of water in times of scarcity (low water periods → NQ)
  → optimal design and management of water storage facilities

- areas and storage capacities of selected reservoirs → see table 6.1

Table 6.1: Areas and storage capacities of selected reservoirs and inland lakes

Reservoir / Inland lake	Area [km^2]	Storage capacity [km^3]
Inland lakes worldwide:		
Akosombo (Volta / Ghana, Upper Volta)	8 480	153
Samara (former Kuibishev, Volga, Russia)	6 450	58
Kariba (Zambezi, Zambia / Zimbabwe)	5 580	181
Bratsk (Angara, Russia)	5 470	169
Aswan (Nil, Egypt / Sudan)	5 250	165
Largest reservoir in eastern Germany:		
Bleiloch (river Saale)	9	0,215
Selected inland lakes:		
Caspian Sea (Russia, Iran), marine	371 800 *	
Dead Sea (Jordan, Palestine), hyper-haline	183 *	
Lake Baikal (Russia)	31 500	23 000
Lake Victoria (Kenya, Tanzania, Uganda)	68 800	
Lake Titicaca (Bolivia)	6 900	
Lake Superior (USA, Canada)	82 414	
Lake Huron (USA, Canada)	59 586	
Lake Balaton (Hungary)	596	
Lake Geneva (Swiss, France)	581	
Lake Bodensee (Germany)	539	48
Lake Schweriner See (Germany)	63	0,354
For comparison:		
All groundwater resources of East Germany		9
All lakes in open-cast mines of East Germany		0,36

* Historical expansion, shrinking in the last decades

* **reservoir types from hydrological point of view** (overview → see figure 6.1)**:**

- single-purpose reservoirs (at present excluding flood retention reservoirs not more widespread)

- multi-purpose reservoirs:
  → provision of drinking and process water (including irrigation water)
  → flood protection and increase of low water (shipping, ecological necessary minimum discharge)
  → other uses: hydroelectric power, recreation, fishing ...

→ different uses may exclude each other → possibly conflicts of interests

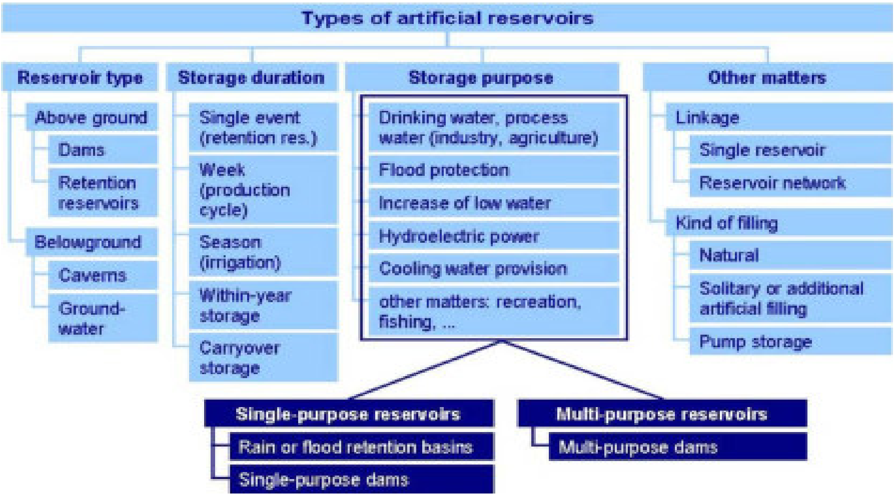
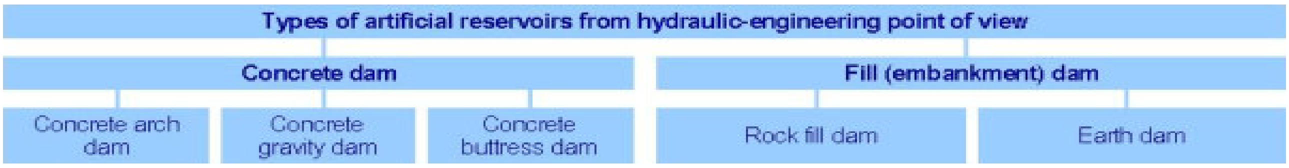


Fig. 6.1:

Reservoir types from hydrological point of view

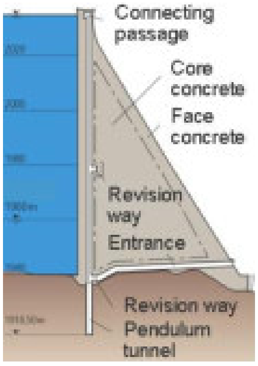
- situation regarding to single-purpose and multi-purpose reservoirs worldwide: 43 % single-purpose reservoirs, 57 % multi-purpose reservoirs
- about 50 % of single-purpose reservoirs → for irrigation, approximately 20 % for energy production, nearly 20 % for water supply, the rest are: flood control and recreation

* **overview on reservoir types from hydraulic-engineering point of view:** → see figure 6.2



a) Concrete arch dam

b) Concrete gravity dam



c) Concrete buttress dam

Fig. 6.2:

Reservoir types from non-hydrological point of view (after Wikipedia, images: Versgui)

6.2. Storage characteristics

* **essential storage characteristics** (→ see also figure 6.3)**:**

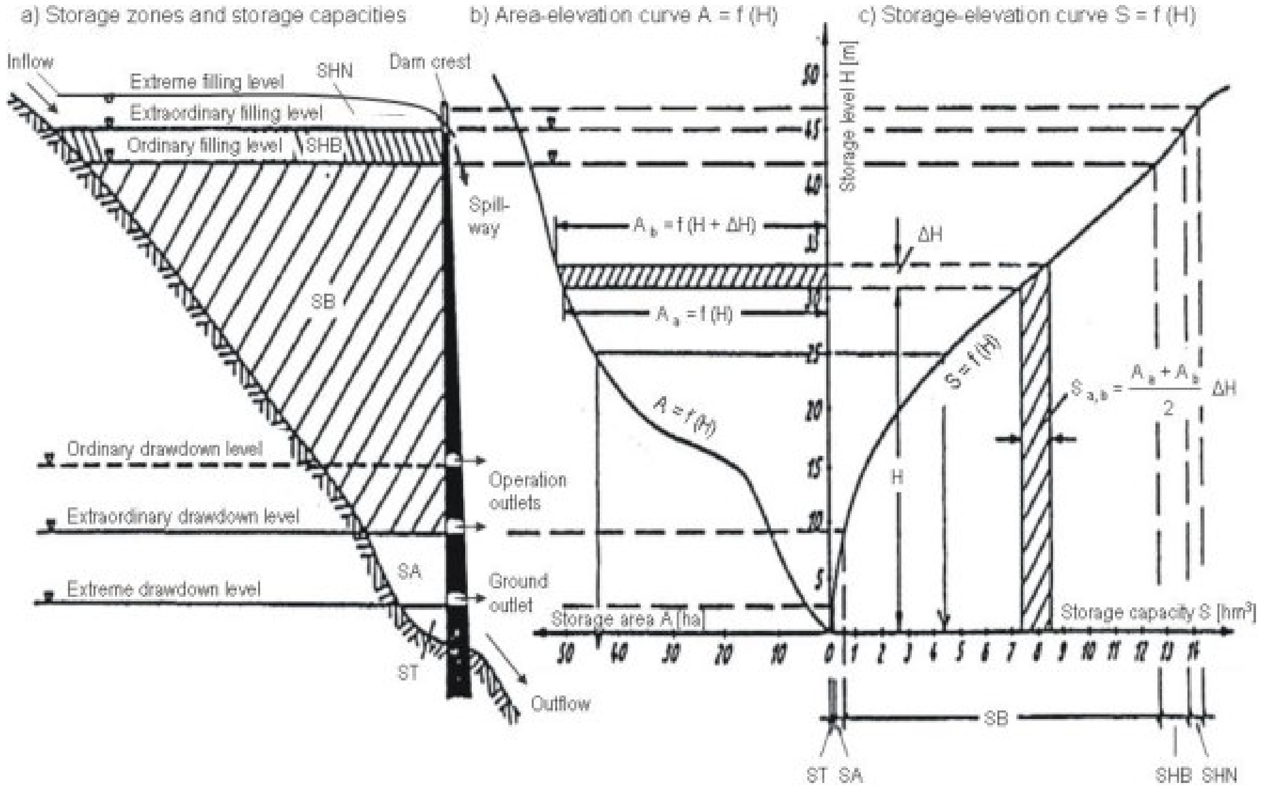
▹ ***dead storage capacity ST:***
- not possible to empty → no regulation possible
- necessary for sedimentation (ST depends on inflow sediment load) and for maintenance of living organism (phytoplankton and zooplankton, microbes, fishes, …)

▹ ***superior storage capacity SA:***
- use (management) in exceptional cases (for example in extreme dry periods) only

▹ ***operation storage capacity SB:***
- operated under normal conditions → permanent usage aspired

▹ ***flood protection capacity SH:***
- sum of manageable (controllable) flood protection volume and uncontrollable flood protection volume (top level of spillway)

▹ ***usable reservoir capacity SN:***
- in most cases: SN = SA + SB
- sometimes: SN = SB



ST – Dead storage zone
SA – Buffer zone
SB – Conservation (operation) zone
SHB – Flood control zone
SHN – Spill zone

Fig. 6.3: Storage characteristics and relationship between water level H, storage area A and storage capacity S (after DYCK, ET AL, 1976)

6.3. Water balance equation for reservoir capacity estimation

continuity equation (balance equation) → see figure 6.4:

$$dS(t)/dt = Q_o(t) + Q_u(t) + P(t) - Q_i(t) - EP(t) - Q_s(t) - Q_{min}(t) - Q_{ü}(t) \qquad (6.1)$$

where: $Q_o(t)$ - Surface water inflow
$Q_u(t)$ - Subsurface water inflow (interflow, groundwater)
$P(t)$ - Precipitation on the reservoir surface
$Q_i(t)$ - Infiltration into the ground (seepage losses)
$EP(t)$ - Evaporation losses (potential evaporation)
$Q_s(t)$ - Regulated outflow (output control)
$Q_{min}(t)$ - Minimum outflow (minimum flow maintenance = minimum discharge from ecological point of view)
$Q_{ü}(t)$ - Spillway
$dS(t)/dt$ - Change in reservoir storage volume = f (storage level, siltation)

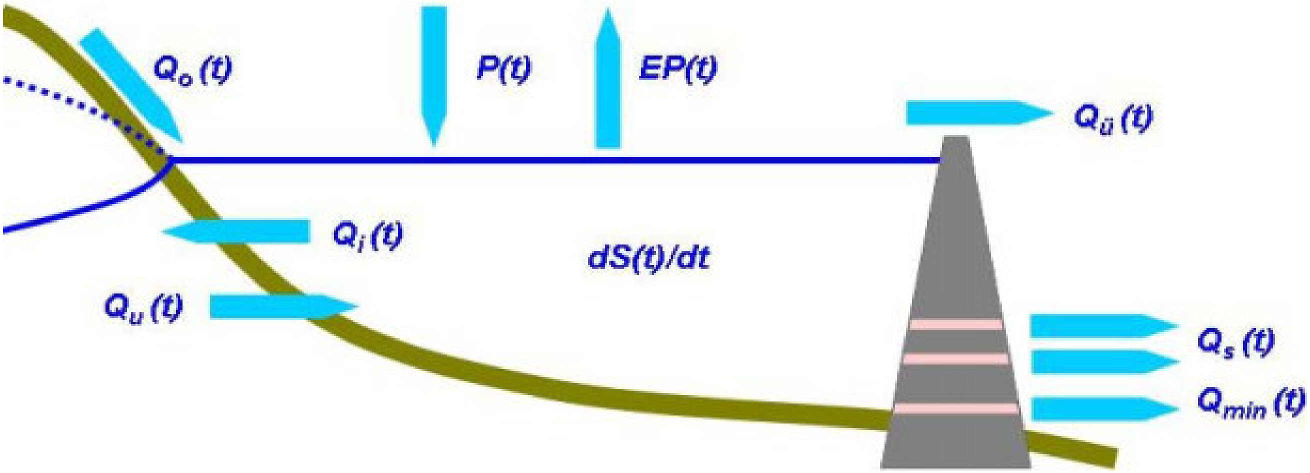


Fig. 6.4: Water balance of a storage reservoir

6.4. Reservoir outflow rules

The reservoirs outflow arises from the storage operation schedule in dependence on main purpose of the reservoir, on catchment conditions, on existing regulation devices of the dam and on development status of hydrology:

a) constant outflow:

- simplest, but in most cases not rational outflow regulation (no consideration of water yield and water demand variations)
- regulated outflow is deceeded in case of empty reservoir and exceeded in case of full reservoir
- example: Aswan dam (Nil / Egypt)
  → equalisation on mean discharge MQ
  → maximum possible compensation by very large storage capacity

b) seasonally caused outflow:

- determination of a monthly variable regulated outflow (depending on seasonal fluctuations in water demand for the different water users, such as agricultural irrigation)

c) reservoir filling caused outflow:

- outflow in dependence on reservoir filling condition
  → reservoir emptying and spillway prevention
  → stable reservoir outflow by considering adaptabilities of the water user (within certain limits)

d) user caused outflow:

- outflow respectively the users role (in case of scarcity of water the user with the lowest security of water supply will be “drained” at first)

e) inflow caused outflow:

- outflow regulation in dependence on expected inflow
- for longer periods almost impossible, but for shorter periods (for example before an expected flood event) often practiced

→ Storage operation schedules consider generally more of these opportunities.

6.5. Hydrological design of reservoirs

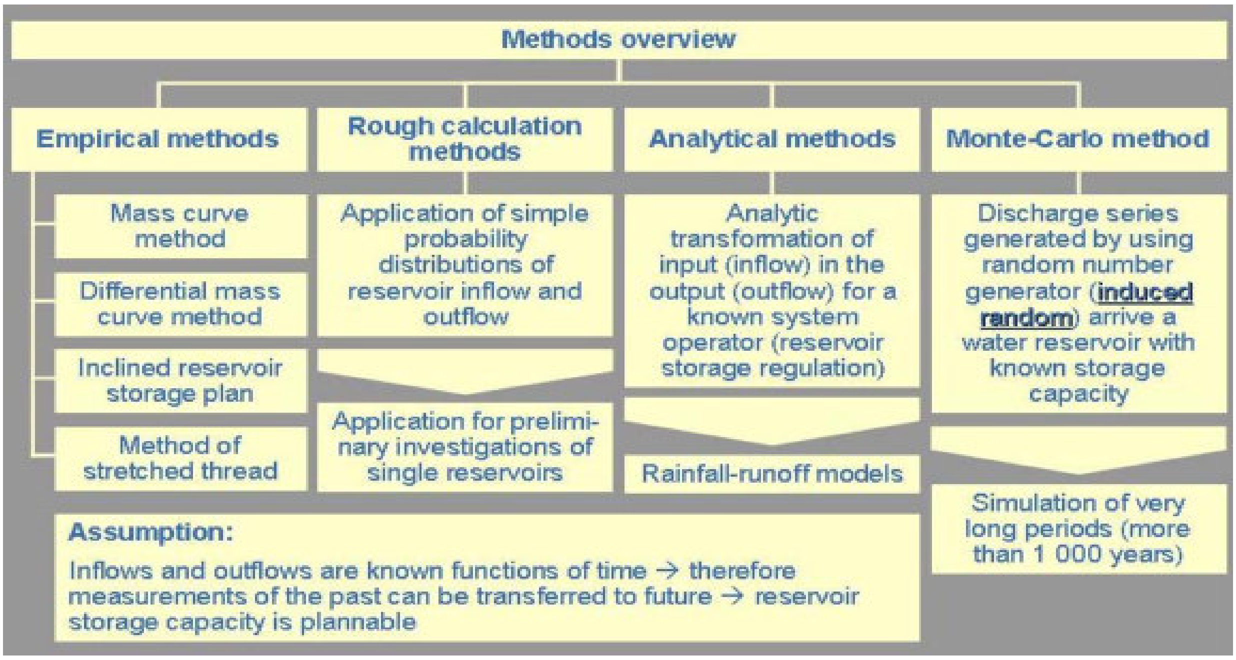
6.5.1. Methodology

- objective: estimation of the optimum (smallest) storage volume for concrete inflow and outflow values

- calculation basis: mostly monthly mean discharge values

* **main methods:**

- methods overview → see figure 6.5



Assumption:

Inflows and outflows are known functions of time → therefore measurements of the past can be transferred to future → reservoir storage capacity is plannable

Fig. 6.5: Overview on methods with respect to the hydrologic design of water storage facilities

6.5.2. Empirical methods

* **methodology:**

▹ ***overview of the main methods:***

- mass curve method, mass curve difference method, method of stretched thread
  → mathematical respectively graphical determination of the optimum storage capacity

▹ ***explanation of the methods using discharges of gauge Lichtenwalde, river Zschopau (1958 – 65):***

→ for general explanation of the methodology sufficiently
→ time series for a real storage capacity calculation too short

a) inflow values to the planned water reservoir:

- here: monthly average discharge values
- corresponding values from German “Gewässerkundliche Jahrbücher” → see column 2 in table 6.2

Table 6.2: Numerical calculation of the summation line SL, of the summation difference line SDL and of the inclined reservoir storage plan for gauge Lichtenwalde, river Zschopau, 1958 – 1965 (extract), v = full, l = empty

Year/ month	Inflow	Σ Inflow month	Σ Inflow total	Mean inflow sum	Summation differences	Outflow summs	Difference	Storage contents	Spillover
	Q_Z $[m^3/s]$	$Z=Q_Z\,t$ $[hm^3]$	ΣZ $[hm^3]$	$\Sigma (MQ\,t)$ $[hm^3]$	(4) - (5) $[hm^3]$	$\Sigma (Q_A\,t)$ $[hm^3]$	(4) - (7) $[hm^3]$	S $[hm^3]$	$Q_Ü$ $[hm^3]$
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
1958:									
January	16,2	42,6	42,6	49,7	- 7,1	39,7	2,9	2,9	
February	43,7	115,0	157,6	99,4	+ 58,2	79,4	78,2	78,2	
March	28,5	74,9	232,5	149,1	+ 83,4	119,1	113,4	113,4	
April	57,3	150,9	383,4	198,8	+184,6	158,8	224,6	224,6	
Mai	33,2	87,3	470,7	243,5	+222,2	198,5	272,2	272,2	
June	11,8	31,0	501,7	298,2	+203,5	238,2	263,5	263,5	
July	69,7	183,5	685,2	347,9	+337,3	277,9	407,3	406,8 v	0,5
August	11,6	30,5	715,7	397,6	+318,1	317,6	398,1	397,6	
Sept.	7,9	20,8	735,5	447,3	+288,2	357,3	378,2	377,5	
October	27,9	73,4	808,9	497,0	+311,9	397,0	411,9	406,8 v	5,1
Nov.	19,0	49,9	858,8	546,7	+312,1	436,7	422,1	406,8 v	15,3
Dec.	18,3	48,1	906,9	596,4	+310,5	476,4	430,5	406,8 v	23,7
1959:									
January	27,6	72,6	979,5	646,1	+333,4	516,1	463,4	406,8 v	56,6
February	17,1	44,9	1024,4	695,8	+328,6	555,8	468,6	406,8 v	61,8
March	34,8	91,5	1115,9	745,5	+370,4	595,5	564,4	406,8 v	157,6
April	22,8	59,9	1175,8	795,2	+380,6	635,2	540,6	406,8 v	133,8
Mai	29,2	76,7	1252,5	844,9	+407,6	674,9	577,6	406,8 v	170,8
June	9,4	24,7	1277,2	894,6	+382,6	714,6	562,6	391,8	
July	7,3	19,2	1296,4	944,3	+352,1	754,3	542,1	371,3	
...	...	...	...	...	...	...	...	...	
1962:									
April	49,3	129,6	2955,2	2582,8	+372,4	2064,4	890,8	395,5	
Mai	25,2	66,2	3021,4	2632,5	+388,9	2104,1	917,3	406,8 v	50,2
June	21,1	31,8	3053,2	2682,1	+371,1	2143,8	909,4	389,9	
July	10,5	27,6	3080,8	2731,8	+349,0	2183,5	897,3	386,8	
August	6,7	17,6	3098,4	2781,5	+316,9	2223,2	875,2	364,7	
Sept.	5,6	14,7	3113,4	2831,1	+282,0	2262,2	850,2	339,7	
...	...	...	...	...	...	...	...	...	
1965:									
January	16,2	42,6	3895,2	4224,2	- 329,0	3374,5	520,7	12,1	
February	10,5	27,6	3922,8	4274,2	- 351,4	3414,2	508,6	0,0 l	
March	58,3	153,2	4076,0	4324,1	- 248,1	3453,9	622,1	113,5	
April	72,1	189,5	4265,5	4374,0	- 108,5	3493,6	771,9	263,3	
Mai	80,9	212,6	4478,1	4426,6	+ 51,5	3533,3	977,8	406,8 v	29,4
June	36,9	97,0	4575,1	4473,9	+101,2	3574,0	1002,1	406,8 v	86,7
July	18,1	47,6	4622,7	4523,8	+ 98,9	3612,7	1010,0	406,8 v	94,6
August	9,3	24,4	4647,1	4573,8	+ 73,3	3652,4	994,7	391,5	
Sept.	7,9	20,8	4667,9	4623,7	+ 44,2	3692,1	975,8	372,6	
October	6,2	16,3	4684,1	4673,6	+ 10,5	3731,8	952,3	349,1	
Nov.	5,7	15,0	4699,1	4718,3	- 19,2	3771,5	927,6	324,4	
Dec.	28,3	74,4	4773,5	4773,5	± 0,0	3811,2	962,3	359,1	

b) conversion of monthly discharges Q_z in m^3/s into mean monthly inflow volumes Z in hm^3:

$$Z = Q_z * \Delta t * 10^{-6} \tag{6.2}$$

where Z - inflow volume [hm^3]
Q_Z - inflow discharge [m^3/s]
Δt - time [s]

- conversion of a 30-day month: 1 m^3/s * 2,592 * 10^6 s = 2,592 hm^3
- values for the example → see column 3 in table 6.2

c) Continuous summation of the monthly inflow volumes:

- see column 4 in table 6.2
- graphical illustration → **mass curve diagram** (→ see figure 6.6)

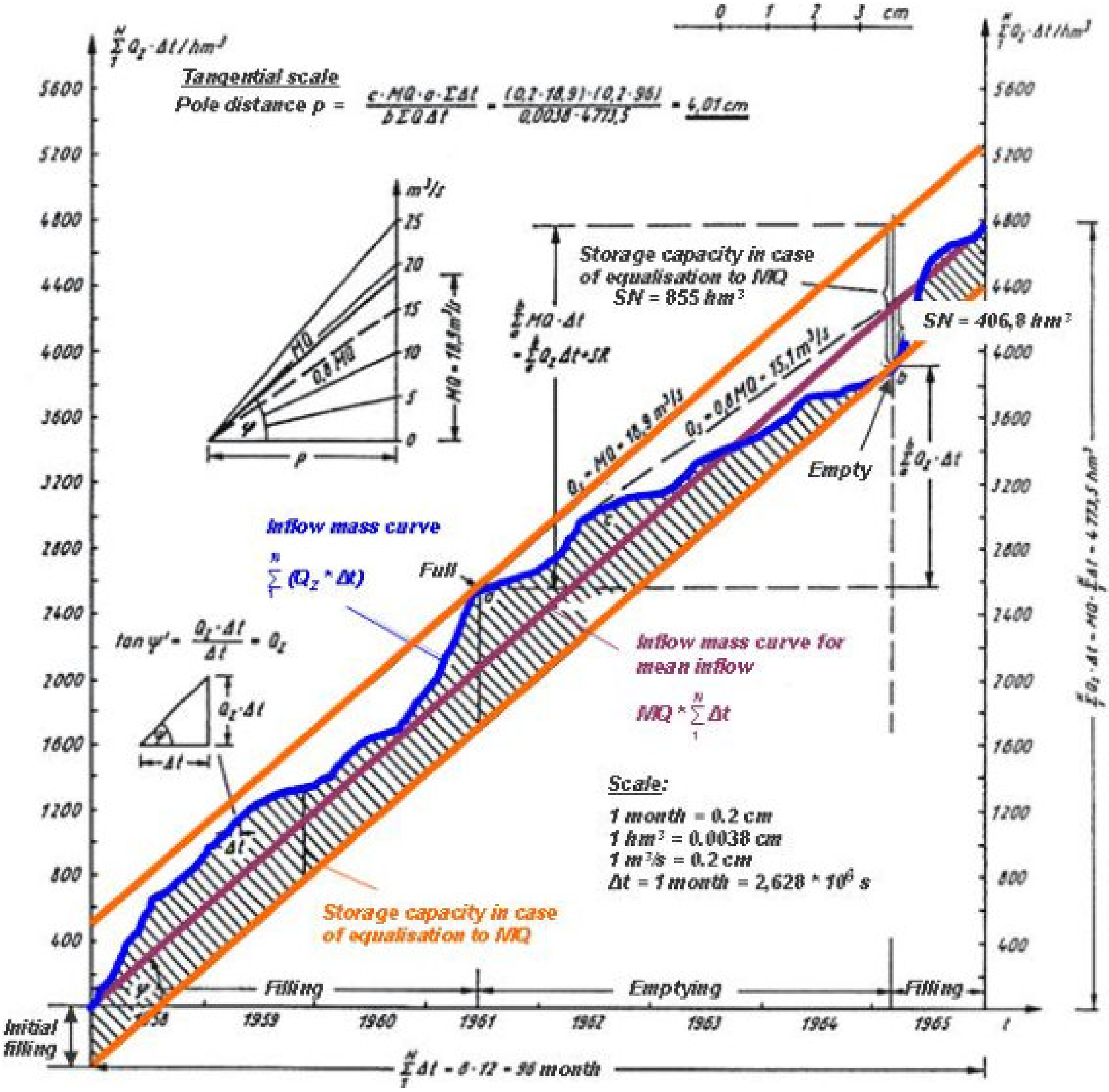


Fig. 6.6: Mass curve diagram with tangential scale for gauge Lichtenwalde, river Zschopau, 1958 – 1965 (after DYCK, ET AL., 1976)

d) estimation of the optimum (smallest) storage capacity SN in dependence on outflow:

▹ example 1:

How big the storage capacity SM of a reservoir has to be, to equalise all discharge variations of the time series 1958 – 1965 to mean discharge MQ? → constant outflow Q_s = MQ:

- determination of the relevant low water period (in this case from the time a to b) → decisive emptying of all water reservoirs (surface, soil, groundwater)
- plot two parallel lines (full and empty) with the increase of MQ
- reading of the storage capacity → SM = 855 hm^3

It is assumed that at the beginning a initial storage filling of about 400 hm^3 is present (otherwise the reservoir would have been rather empty, around October 1963 → intersection point with the MQ line).

▹ example 2:

How big the storage capacity SM of a reservoir has to be, to guaranty a constant outflow of Q_s = 0,8 MQ = 15,1 m^3/s?:

- determination of the slope by means the tangent scale (→ see figure 6.6)
- plot of the outflow mass curve with the slope of 0,8 MQ at the maximum points of the inflow mass curve
- storage capacity estimation: SN = maximum difference between outflow and inflow mass curves → SN = 406,8 hm^3

  → relevant low water period from points c to b in figure 6.6

mass curve plot poor suitable (discharge variations bad imaged) → transition to differential mass curve

differential mass curve:

- estimation by tilting the MQ line in the horizontal position
- graphic determination → see figure 6.7
- computational determination → see table 6.2, column 6 (difference between columns 4 and 5)

inclined reservoir storage plan (→ see figure 6.8):

▹ starting point: differential mass curve (→ see figure 6.7)

▹ process of filling and emptying of the reservoir, and overflow volumes for any outflow values and reservoir volumes determinable:

- plot of two straight lines with the slope of the regulated outflow Q_s and the distance of storage capacity SN, where a start with empty (→ see figure 6.8), partly filled or full reservoir is possible
- projection of the differential mass curve into the inclined reservoir storage plan, whereby in case of spillway the line will continue as long as inflow $Q_Z >$ regulated outflow Q_s
- in case of $Q_Z < Q_s$ → plot of a parallel shifted differential mass curve starting at maximum storage capacity (here 406,8 hm^3]
- total spillway volume results from the sum of single spillways
- computational solution → see table 6.2 (columns 7 to 10)

→ inclined reservoir storage plan offers graphic illustration facility

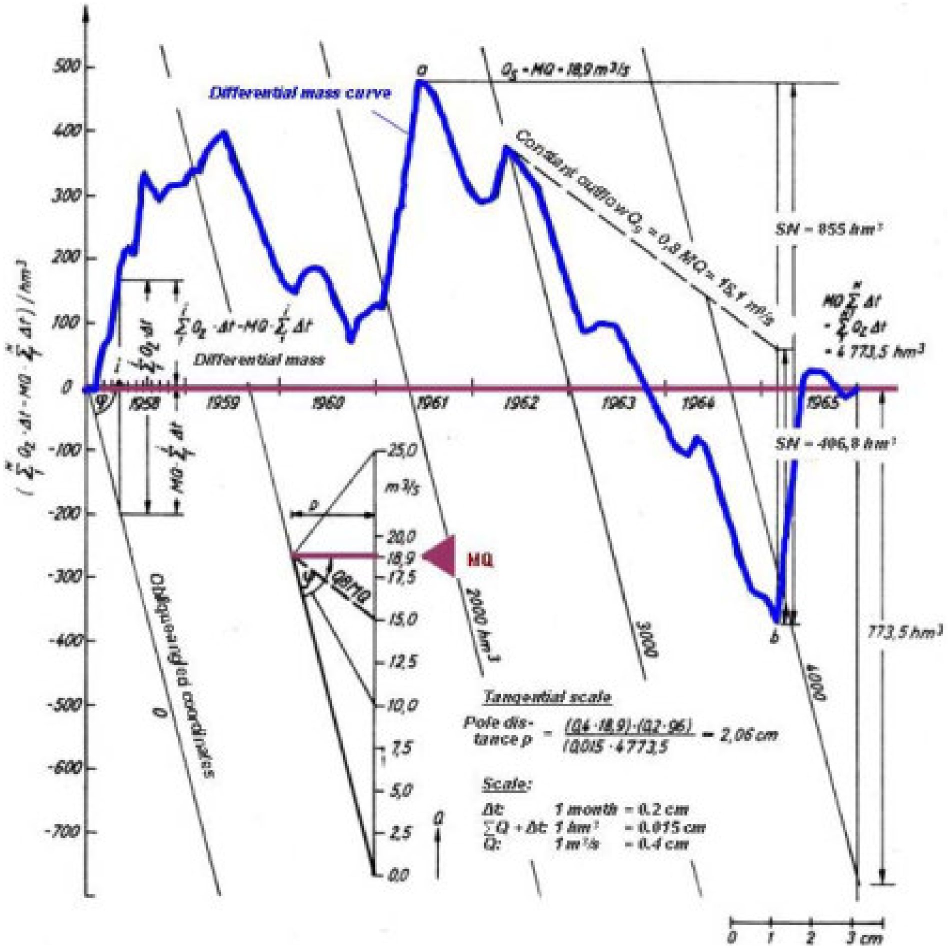


Fig. 6.7:

Differential mass curve for gauge Lichtenwalde, river Zschopau, 1958 – 1965 (after DYCK, *ET AL., 1976)*

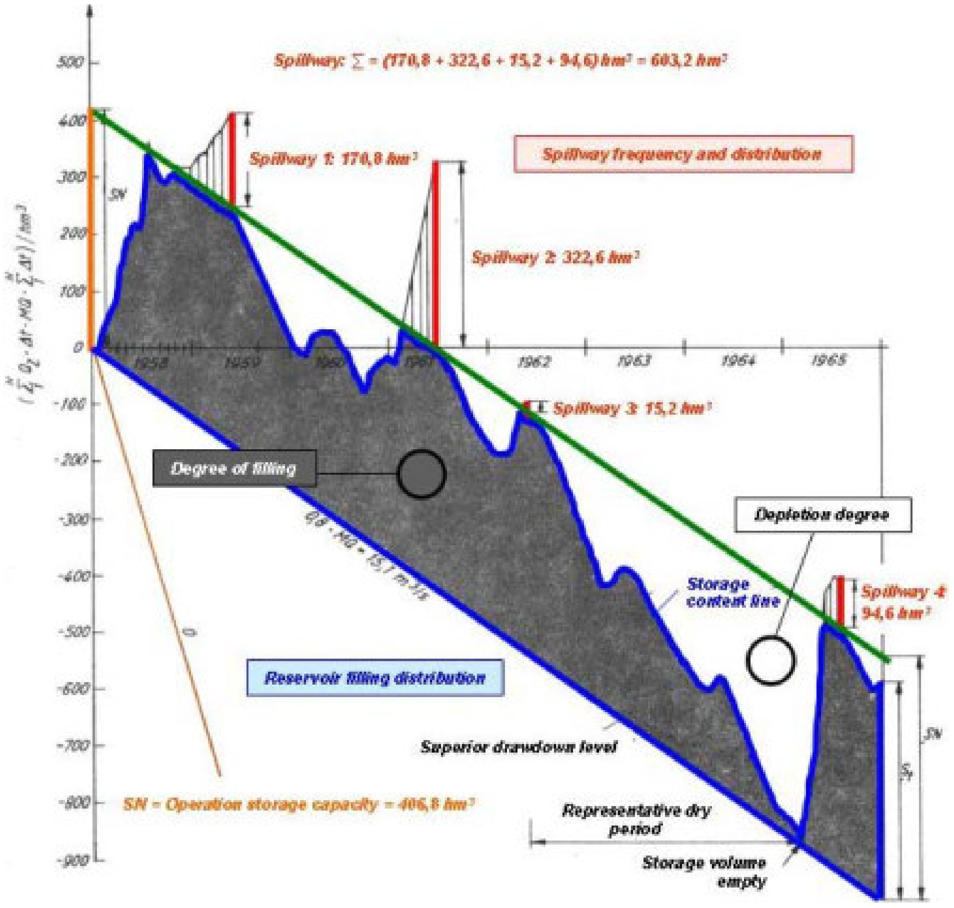


Bild 6.8:

Inclined reservoir storage plan, Q_s = 15,1 m^3/s = 80 %, MQ and SN = 406.8 hm^3 for gauge Lichtenwalde, river Zschopau, 1958 – 65 (after DYCK, *ET AL., 1976)*

Method of the stretched thread:

- starting point: two differential mass curves with a distance of storage capacity SN
- methods illustration → see figure 6.9
- stretched thread = shortest path through the "corridor":
  → sectionally constant reservoir outflow Q_s
  → duration of constant outflow depends on reservoir capacity ("corridor width")
  → optimization by means of graphic programs
  → frequency distribution of Q_s can be determined (assuming a long series of observations)

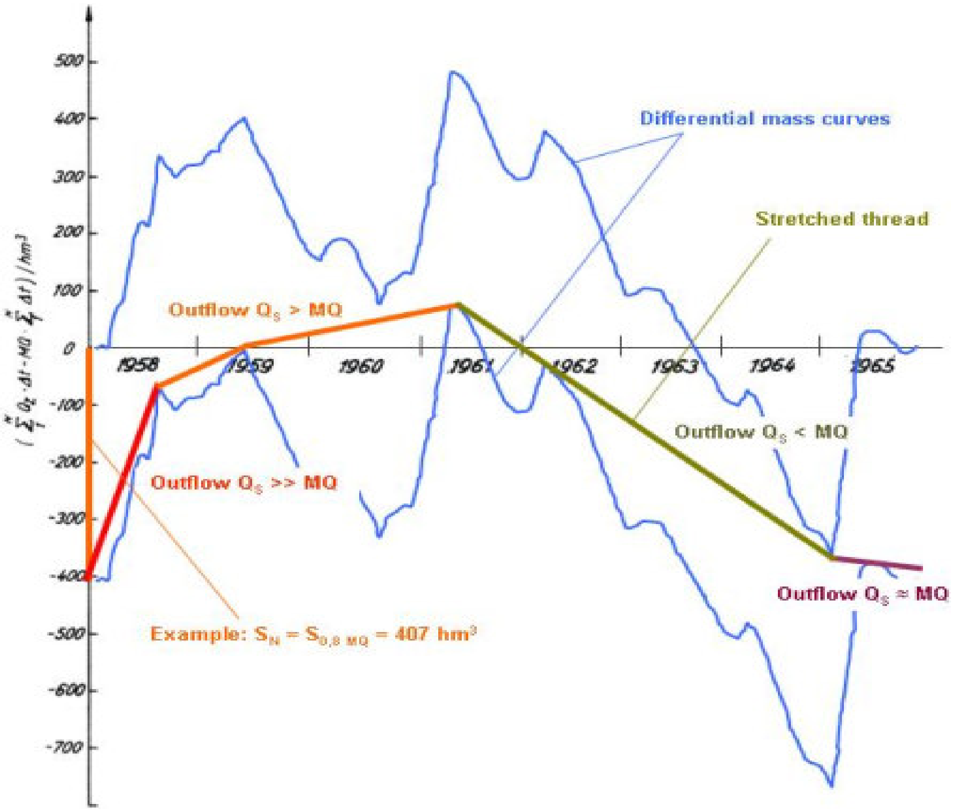


Fig. 6.9:

Method of the stretched thread for gauge Lichtenwalde, river Zschopau, 1958 – 65 (after DYCK, ET AL., 1976)

* ***conclusions for reservoir capacity sizing:***

- optimum reservoir capacity sizing not possible, because the future behaviour of the inflow hydrograph is not known
- long-term predictions (especially territorial development planning) necessary → for example estimation of runoff formation in case of anthropogenic influences

- application of empirical methods for hydrological storage design → see seminar 28

6.5.3. Monte Carlo method

Monte Carlo method (MCM) = method of statistical tests

* **example for method illustration:**

▹ ***task:***

- calculation of a definite integral in the absence of closed analytical solution

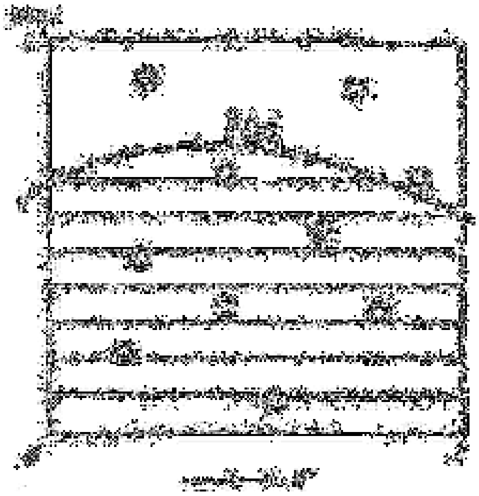


Fig. 6.10: MCM example (DYCK, ET AL., 1976)

▹ ***solution*** *(→ see figure 6.10):*

- solution of the integral = hatched area
- solution by MCM:
  - → generating random pairs of x, f (x)
  - → registration of the numbers of pairs within and outside of the shaded area
  - → solution: The ratio of the value pairs within the hatched area to the total number of values
- precision of the solution increases with increasing number of trials

* **application of the MCM for hydrological reservoir management:**

- generation of continuous discharge series
- Generated discharge series impinges on fictional reservoir with different storage capacities.
- application of the base equation for reservoir capacity estimation (→ see chapter 6.3)
- registration of inflows, outflows, storage fillings, spillways, → frequency distribution

6.5.4. Flood retention reservoirs

- general suitability of the methods described in chapters 6.5.1 to 6.5.3 for the design of flood retention reservoirs (RHB)
- special feature: not all inflows to the reservoir are considered, but floods only

* **control options:**

a) control in case of sufficient storage capacity *(→ see figure 6.11):*

- flood wave can be completely cut to regulated outflow Q_R
- quickly basin emptying after flood event (storage capacity of following flood events!)

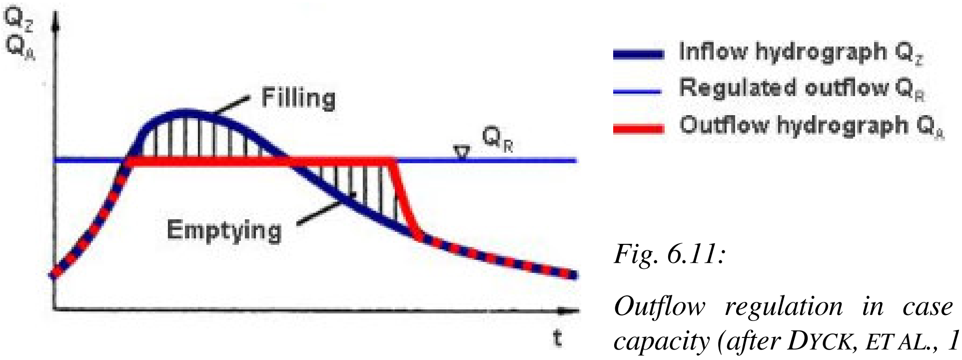
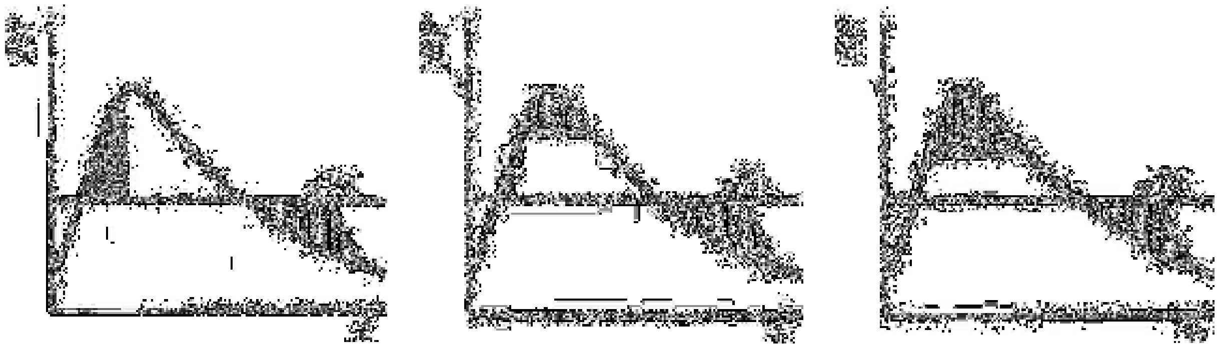


Fig. 6.11:

Outflow regulation in case of sufficient storage capacity (after DYCK, ET AL., 1976)

b) ***control in case of not sufficient storage capacity** (→ see figure 6.12):*

- worst case (→ see figure 6.12 a) → effect of flood retention basin = 0, because $Q_{max} = HQ$
- practical case (→ see figure 6.12 b):
  → cutting of the flood peak by gradually outflow increase Q_R
  → $Q_{max} < HQ$
- ideal case (→ see figure 6.12 c):
  → maximum possible cutting of flood peak
  → Q_{max} → minimum



a) Worst case *b) Practical case* *c) Ideal case*

Fig. 6.12: Outflow regulation in case of not sufficient storage capacity (after DYCK, ET AL., 1976)

* **design of flood retention reservoirs in case of a constant regulated outflow:**

- objective: determination of the storage capacity, which is necessary in case of design flood event
- storage capacity design of a flood retention reservoir in case of a constant regulated outflow bases on application of the simplified continuity equation (→ see chapter 6.3, assumption: $Q_Z > Q_A$):

$$dS(t) / dt = Q_Z(t) - Q_A(t) \qquad (6.3)$$

where $dS(t)$ - Change of storage filling [l/Δt respectively $m^3/(s\ \Delta t)$]
$Q_Z(t)$ - Inflow hydrograph [l/Δt respectively $m^3/(s\ \Delta t)$]
$Q_A(t)$ - Outflow hydrograph [l/Δt respectively $m^3/(s\ \Delta t)$] ($Q_A = Q_R$ = regulated outflow = constant)

- For two consecutive time steps the solution of the continuity equation in differential form is the following (approximate solution):

$$S(t_j) = S(t_{j-1}) + \Delta t\,[Q_Z(t_j) - Q_A(t_j)] \qquad (6.4)$$

where $S(t_j)$ - Storage filling at time t_j [l respectively m^3, in dependence on the unit of the discharges]
$S(t_{j-1})$ - Storage filling at time t_{j-1} [l respectively m^3, in dependence on the unit of the discharges]
$Q_Z(t_j)$ - Inflow to the reservoir at time t_j [l/s respectively m^3/s]
$Q_A(t_j)$ - Outflow out of the reservoir at time t_j [l/s respectively m^3/s]
Δt - Time difference between t_{j-1} and t_j [s]

- Equation 6.4 is applied starting from the moment on which the inflow to the reservoir is greater than regulated outflow ($Q_Z > Q_A$).
- calculation ends in that moment, where $Q_Z < Q_A$

→ example for continuity equation application → see figure 6.13

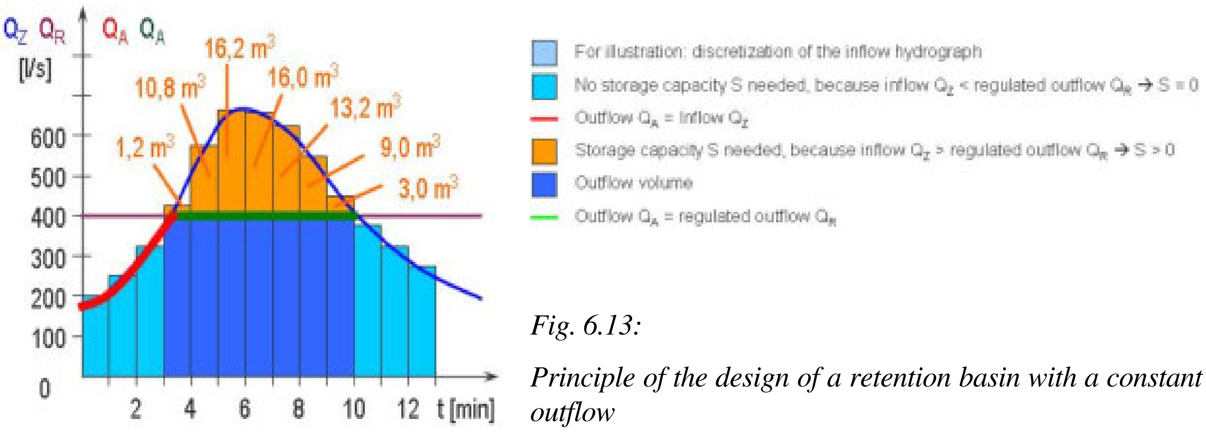


Fig. 6.13:

Principle of the design of a retention basin with a constant outflow

→ example for the calculation of storage capacity → first time interval for $Q_Z > Q_A$ (t = 3 … 4 min):
→ $Q_Z = 420$ l/s, $Q_A = Q_R = 400$ l/s, $\Delta t = 1$ min = 60 s
→ $\Delta S = Q_Z - Q_A = 420$ l/s – 400 l/s = 20 l/s
→ for $\Delta t = 1$ min = 60 s: 1 200 l = 1.2 m^3

- hydrological design of a rainwater retention basin → see seminars 29 and 30

6.6. Special literature for hydrological water reservoir sizing

Dyck, S. u.a. (1976):
Angewande Hydrologie, Teil 1: Berechnung und Regelung des Durchflusses der Flüsse. VEB Verlag für Bauwesen Berlin.

Jain, S. K. and V. P. Singh (2003):
Water resources systems planning and management. Developments in water sciences. Chapter 10: Reservoir sizing, Chapter 11: Reservoir operation. Elsevier

Maniak, U. (2005):
Hydrologie und Wasserwirtschaft. Eine Einführung für Ingenieure. Kapitel 7: Bemessung und Betrieb von Talsperren und Rückhaltebecken. Springer-Verlag Berlin, Heidelberg, 5. Auflage

DIN 19 700 (2004): Stauanlagen, Teil 11: Talsperren. Beuth-Verlag Berlin, Wien, Zürich

Appendix 1

Seminary tasks for „Hydrology III“ course

Seminar 1: Stream flow hydraulics 1 – Discharge measurements by means of weir and bucket and stopwatch

Seminar 2: Stream flow hydraulics 2 – Discharge measurement by means of current meter

Seminar 3: Stream flow hydraulics 3 – Discharge measurement by using tracer

Seminar 4: Stream flow hydraulics 4 – Longitudinal and cross-sectional measurements

Seminar 5: Stream flow hydraulics 5 – Estimation of cross-sectional parameter from cross-sectional measurements

Seminar 6: Stream flow hydraulics 6 – Streaming and shooting flow

Seminar 7: Stream flow hydraulics 7 – Vegetation mapping and photo documentation in the field

Seminar 8: Stream flow hydraulics 8 – Mapping of hydraulic narrows in the field

Seminar 9: Stream flow hydraulics 9 – Hydraulic parameter determination on base of filed experiments

Seminar 10: Stream flow hydraulics 10 – Calibration of roughness coefficients by means of discharge measurement results

Seminar 11: Stream flow hydraulics 11 – Estimation of equivalent sand roughness by sieve analysis

Seminar 12: Stream flow hydraulics 12 – Estimation of hydraulic roughnesses by empirical methods (Cowan, Einstein / Horton, Sellin)

Seminar 13: Stream flow hydraulics 13 – Flow velocity and discharge calculations for different boundary conditions by GMS formula

Seminar 14: Stream flow hydraulics 14 – Flow velocity and discharge calculations by Colebrook & White

Seminar 15: Flood analysis 1 – Application of stochastic conception

Seminar 16: Flood analysis 2 – deterministic conception 1: Estimation of infiltration ability of the soil surface (in-situ and laboratory methods)

Seminar 17: Flood analysis 3 – deterministic conception 2: Estimation of effective and total porosity

Seminar 18: Flood analysis 4 – deterministic conception 3: Regionalisation methods I

Seminar 19: Flood analysis 5 – deterministic conception 4: Regionalisation methods II

Seminar 20: Flood analysis 6 – deterministic conception 5: Estimation of design rainfall by means of KOSTRA atlas

Seminar 21: Flood analysis 7 – deterministic conception 6: Parametrisation of a conceptual rainfall runoff model

Seminar 22: Flood analysis 8 – deterministic conception 7: Runoff modelling for different rainfall distributions, rainfall tolerance ranges and return periods

Seminar 23: Flood analysis 9 – deterministic conception 8: Runoff modelling for different rainfall durations

Seminar 24: Flood analysis 10 – deterministic conception 9: Runoff modelling by considering of flow routing effects (isochrone method)

Seminar 25: Flood analysis 11 – deterministic conception 10: Runoff modelling for different initial soil moisture conditions

Seminar 26: Flood analysis 12 – deterministic conception 11: quantitative assessment of the influence of land use changes in a catchment by means of rainfall runoff model

Seminar 27: Low water analysis

Seminar 28: Hydrologic reservoir planning 1 – Application of empirical methods

Seminar 29: Hydrologic reservoir planning 2 – Design of a rainwater retention basin by using simplified continuity equation

Seminar 30: Hydrologic reservoir planning 3 – Design of a rainwater retention basin by means of rainfall runoff model

Hydrological seminar 1:

Stream flow hydraulics 1 – Discharge measurements by means of weir and bucket and stopwatch

Objective:

Discharge measurement of creek Kleinwaltersdorfer Bach near the bridge beside café´ ”Waldcafe´” by means of v-notch (triangular) weir and, if possible, bucket and stopwatch

Methodology: → see lecture notes “Hydrology I”, chapter 5.2.2

Measurement equipment:

- plastic v-notch weirs with different sizes
- mineral sealing material
- small spade
- ruler, folding ruler respectively measuring tape (maximum 5 m long)
- bubble level
- stones or something like that for preparation of tide gauge zero
- scaled bucket
- stopwatch
- chest waders, gumboots

Measurements prearrangement and procedure:

- mapping of flow profile (cross-sectional profile) → not for seminar 1. but for seminars 4 and 10 necessary
- measuring point: pipe in the direction of inflow, bridge of creek Kleinwaltersdorfer Bach near the bridge beside café´ ”Waldcafe´”
- measuring point preparation: removal of stones, branches … (if necessary)
- weir installation and weir sealing (ground and embankments)
- fixation of tide gauge zero → transformation of the deepest point of the triangular notch horizontally into the inflow side → the horizontal distance of tide gauge zero should be at least four times higher than the vertical v-notch insection → stabilisation of tide gauge zero (no vertical displacement during measurement!)
- start of measurement if steady-state conditions (constant water level at inflow side) occurred
- measurement of head on calibrated tide gauge zero → measurement at least two times
- if possible: realization of a bucket measurement (water volume per time) → measurement at least two times

Tasks:

1. Documentation of measurements preparations (sketches with measuring points, measuring configuration, steps of measurements preparations, …)
2. Please register the measured values! Evaluate possible error sources and their influences on the measurement accuracy!
3. Please calculate the discharge values in result of the two measuring methods!
4. Compare and discuss the results!

Hydrological seminar 2:

Stream flow hydraulics 2 – Discharge measurement by means of current meter

Objective:

Discharge measurement of creek Kleinwaltersdorfer Bach near the bridge on local connection road to federal highway B 101 by means of current meter

Methodology: → see lecture notes "Hydrology I", chapter 5.2.2

Measurement equipment:

- current meter with complements (diverse propellers, rod, ground stop, spare batteries)
- measuring tape (minimum length: 10 m)
- small spade
- ruler and folding ruler
- sledgehammer and pegs for measuring tape fixing
- stopwatch
- chest waders, gumboots

Measurements prearrangement and procedure:

- measuring point: creek Kleinwaltersdorfer Bach, bridge near the bridge on local connection road to federal highway B 101. downstream
- choice of measurement cross-section
- measuring point preparation: removal of stones, branches … (if necessary)
- cross-sectional measurement → stream width
- definition of increment numbers
- definition of increment widths and centres of each subsection
- depth measurements at left and right increment boundaries and in the centre of each subsection
- definition of numbers and vertical positions of the measurement points in the centre of each increment (one-point- or more-point-measurement method)
- choice of propeller and notification of propeller number (stamped in propeller)
- current meter fixation at the chosen measurement points and register of rotation impulses (at least two measurements on each point, measuring time: 1 minute)

Tasks:

1. Documentation of measurements preparations (sketches with measuring points, measuring configuration, steps of measurements preparations, …)

2. Please note all relevant current meter information and all current meter parameter and register these in table Ü 2.1!

3. Please register the measured values (increment numbers, increment widths, number and position of the measurement points in the centre of each increment, rotation impulses) and record these in table Ü 2.1!

Table Ü 2.1: Current meter information

Manufacturer: ....

Current meter type (rod or float current meter): ....

Propeller number.: .... Propeller diameter: .... cm

Current meter equation and parameters: $v = a + b * n$

$n \leq$	v [.......] =	+	* n [s^{-1}]
$< n <$	v [.......] =	+	* n [s^{-1}]
$n \geq$	v [.......] =	+	* n [s^{-1}]

Propeller number.: .... Propeller diameter: .... cm

Current meter equation and parameters: $v = a + b * n$

$n \leq$	v [.......] =	+	* n [s^{-1}]
$< n <$	v [.......] =	+	* n [s^{-1}]
$n \geq$	v [.......] =	+	* n [s^{-1}]

3. Please calculate flow velocities (point by point, increment mean, total mean), increment cross-sectional areas and discharges! Please calculate the total discharge value!

4. Compare and discuss the results!

5. Is the chosen creek segment optimal for a current meter measurement? Is the used method (current meter) in actual case optimal?

Table Ü 2.2: Discharge measurement report for current meter measurement

Increment number	Distance from point zero	Increment width	Depth on left increment boundary	Depth in the increment centre	Depth on right increment boundary	Cross-sectional area		Vertical distance of measuring point over ground	Impulse number	Measurement time	Flow velocity point by point	Mean increment flow velocity	Mean increment discharge value	
	[cm]	[cm]	[cm]	[cm]	[cm]	[cm^2]	[m^2]	[cm]	[ ]	[s]	[m/s]	[m/s]	[m^3/s]	[l/s]
1														
2														
3														
4														
5														
Total	Total cross-sectional area [cm^2 bzw. m^2] ►							Total discharge [m^3/s bzw. l/s] ►						

Hydrological seminar 3:

Stream flow hydraulics 3 – Discharge measurement by using tracer

Objective:

Discharge measurement of creek Kleinwaltersdorfer Bach near the bridge beside café´ ”Waldcafe´” by means of tracer

Methodology:

→ see lecture notes “Hydrology I”, chapter 5.2.4

Measurement equipment:

In laboratory:
- 500 g sodium chloride
- accurate scales
- 4 PVC bottles with 1 l capacity
- further equipment: spoon, brush, …

For field experiment:
- 4 PVC bottles with 1 l tracer
- scaled bucket
- a few mg Uranine (sodium fluorescein, powdery)
- conductivity electrode with calibration liquid
- measuring tape (maximum 50 m long)
- stopwatch
- chest waders, gumboots

Prearrangement in the laboratory:

- solution of precise 400 g sodium chloride in precise 4 l distilled water
- filling of 4 PVC bottles with 1 l capacity

Measurements prearrangement and procedure:

- measuring point: creek Kleinwaltersdorfer Bach near the bridge beside café´ ”Waldcafe´” (analogous to seminar 1)
- proposals regarding injection and registration points
- realization of a preliminary Uranine test → final definition of injection and registration points
- measuring of cross-sectional area at registration point (for seminar 3 not necessary, but for seminars 4 and 10)
- measuring of the distances between injection point and registration point (in the creels centre)
- conductivity electrode calibration
- transfer of 1 l salt tracer in the scaled bucket and tracer injection at injection point as momentary impulse
- measurement of electric conductivity at registration point and notation of all measured values (if possible each 5 s)
- singular experiment replication

Tasks:

1. Please document all works in laboratory and in the field!

2. Please cause the section of measurements definition! Which criteria have to bee taken into consideration in connection with section of measurements definition, to get representative values for flow velocity and discharge?

3. Register the tracer curves for both measurements! Please register the measured electric conductivities LF in tables Ü 3.1 and Ü 3.2!

Table Ü 3.1: Measured electric conductivity at registration point, tracer experiment 1

t [s]	LF [µS/cm]	t [s]	LF [µS/cm]	t [s]	LF [µS/cm]	t [s]	LF [µS/cm]
5		135		265		395	
10		140		270		400	
15		145		275		405	
20		150		280		410	
25		155		285		415	
30		160		290		420	
35		165		295		425	
40		170		300		430	
45		175		305		435	
50		180		310		440	
55		185		315		445	
60		190		320		450	
65		195		325		455	
70		200		330		460	
75		205		335		465	
80		210		340		470	
85		215		345		475	
90		220		350		480	
95		225		355		485	
100		230		360		490	
105		235		365		495	
110		240		370		500	
115		245		375		505	
120		250		380		510	
125		255		385		515	
130		260		390		520	

Table Ü 3.2: Measured electric conductivity at registration point, tracer experiment 2

t [s]	LF [µS/cm]	t [s]	LF [µS/cm]	t [s]	LF [µS/cm]	t [s]	LF [µS/cm]
5		135		265		395	
10		140		270		400	
15		145		275		405	
20		150		280		410	
25		155		285		415	
30		160		290		420	
35		165		295		425	
40		170		300		430	
45		175		305		435	
50		180		310		440	
55		185		315		445	
60		190		320		450	
65		195		325		455	
70		200		330		460	
75		205		335		465	
80		210		340		470	
85		215		345		475	
90		220		350		480	
95		225		355		485	
100		230		360		490	
105		235		365		495	
110		240		370		500	
115		245		375		505	
120		250		380		510	
125		255		385		515	
130		260		390		520	

4. Please transform the electric conductivities at registration point into chloride concentrations by using formulae Ü 3.1 and Ü 3.2:

- ***for Cl concentrations ≤ 1 g/l:*** $C_{Cl} = 486.85 * LF + 10.51 * LF^2 - 1.516$ ***(Ü 3.1)***

  where: C_{Cl} - chloride concentration [mg/l]
  LF - electric conductivity [mS/cm]

- ***for Cl concentrations > 1 … 20 g/l:*** $C_{Cl} = 0.5449 * LF + 0.0006184 * LF^2 - 0.04343$ ***(Ü 3.2)***

  where: C_{Cl} - chloride concentration [g/l]
  LF - electric conductivity [mS/cm]

5. Please plot the tracer curve! Estimate analogue to seminar 11. lecture notes of course „Hydrology I“ the moment where 50 per cent of tracer amount has reached registration point!

6. Please calculate the flow velocity (distance velocity) by equation 5.16 lecture notes of course „Hydrology I“!

7. Calculate the discharge by equation 5.18 lecture notes of course „Hydrology I“!

8. Discuss the results considering the following aspects:
   - applicability of tracer method regarding the specific creek conditions
   - fault probability analysis (qualitative)
   - variance between the two measurements
   - variance in comparison to the discharge values estimated in seminar 1

Hydrological seminar 4:

Stream flow hydraulics 4 – Longitudinal and cross-sectional measurements

Objective: Measurements of longitudinal and cross-sectional conditions of creek Kleinwaltersdorfer Bach near the bridge beside café´ ”Waldcafe´” and near the bridge on local connection road to federal highway B 101 (investigation points analogue to seminars 1 – 3)

Methodology:

→ see lecture notes for course “Hydrology III”, chapters 1.3.2 and 1.4.1

Measurement equipment:

- levelling instrument Ni 040 A with appendage (tripod, levelling rod, …)
- measuring tape (maximum 50 m long)
- chest waders, gumboots

Measurements prearrangement and procedure:

- measurement points: near the bridge beside café´ ”Waldcafe´” and near the bridge on local connection road to federal highway B 101 (analogue to seminars 1 – 3)
- search of a optimum position for the levelling instrument → from that point it should be possible to visible the whole cross-section and the longitudinal profile for about 50 m
- placing and horizontal adjustment of levelling instrument (please use the spirit level of the levelling instrument)
- definition of a project-related reference altitude point → should be a clear defined point whose geodetic altitude is constant during the experiment (for example bridge foundation, bridge railing, …)
- levelling rod positioning on project-related reference altitude point and altitude read out (read out example → see lecture notes “Hydrology I”, figure 6.17) → reference fixe point
- cross-sectional measuring for the condition of about 1 metres in height above riverbed → flooded during flood events (about 1 metres in height above riverbed is wilfully determined and has nothing to do with the obtained flood levels)
- measurement of longitudinal profile above 50 m upstream and downstream of the bridge, measurement approximately each 10 m

Tasks:

1. Please document all works in the field!

2. Please define for both investigation points the cross-section positions! Please prepare location drawings → see figure Ü 4.1

3. Please define the project-related reference altitude points in horizontal and vertical direction! Estimate the horizontal and vertical positions by means of measuring tape and levelling instrument! Please register the measured values in tables Ü 4.1 und Ü 4.2 and complete the site plans (figure Ü 4.1)!

Site plan (top view) for investigation site near café´ ”Waldcafe´”:

Site plan (top view) for investigation site near the bridge on local connection road to federal highway B 101:

Fig. Ü 4.1: Site plans

3. Define the measurement points in horizontal direction across cross-section and estimate the distances from zero point! The measurement points should be include all relevant conditions (creek bed, water level, embankments, altitude of flooding area). Please register the measured values in tables Ü 4.1 to Ü 4.3! The cross-section should include measurement points above approximately 1 metres in height above riverbed because of flooding areas.

Table Ü 4.1: Geodetic measurements values for creek Kleinwaltersdorfer Bach cross-section at the bridge near café´ "Waldcafe´"

Point No.	Short characteristics	Distance fixed point [m]	Altitude reading levelling instrument [m]	Altitude difference to fixed point [m]

Table Ü 4.2: Geodetic measurements values for creek Kleinwaltersdorfer Bach cross-section upstream the bridge on local connection road to federal highway B 101

Point No.	Short characteristics	Distance fixed point [m]	Altitude reading levelling instrument [m]	Altitude difference to fixed point [m]

Table Ü 4.2: Geodetic measurements values for creek Kleinwaltersdorfer Bach cross-section downstream the bridge on local connection road to federal highway B 101

Point No.	Short characteristics	Distance fixed point [m]	Altitude reading levelling instrument [m]	Altitude difference to fixed point [m]

3. Please plot the cross-sections by means of spreadsheet and graphics software (individual at laptop or personal computer).

4. Please record the longitudinal profiles at the two investigation sites! The longitudinal profiles range from downstream part of the bridges to distances of about 50 m upstream. Measure the vertical and horizontal differences of the deepest points of river bed (in comparison to project-related reference altitude points)! Realise the measurements upstream for about 5 points (each approximately 10 m and at important points on hydraulic point of view)! Please register the measured values in tables Ü 4.3 and 4.4! Complete the site plans (figure Ü 4.1)!

Table Ü 4.3: Geodetic measurements values of longitudinal profile for creek Kleinwaltersdorfer Bach at the bridge near café´ "Waldcafe´"

Point No.	Short characteristics	Distance fixed point [m]	Altitude reading levelling instrument [m]	Altitude difference to fixed point [m]

Table Ü 4.4: Geodetic measurements values of longitudinal profile for creek Kleinwaltersdorfer Bach at the bridge on local connection road to federal highway B 101

Point No.	Short characteristics	Distance fixed point [m]	Altitude reading levelling instrument [m]	Altitude difference to fixed point [m]

5. Characterise the hydraulic situations at bridge culverts by means of visual inspections (cross-sectional changes and differences regarding to hydraulic roughnesses!

Hydrological seminar 5:

Stream flow hydraulics 5 – Estimation of cross-sectional parameter from cross-sectional measurements

Objective:

Geometric parameter estimation on base of cross-sectional and longitudinal profiles information (seminar 4) for creek Kleinwaltersdorfer Bach near the bridge beside café´ ”Waldcafe´” and near the bridge on local connection road to federal highway B 101 and also dissipation of statements regarding compacted or segmented cross-sections at both investigation sites

Methodology:

→ see lecture notes "Hydrology III", chapters 1.3.2 and 1.4.1

Necessary information:

- cross-sectional and longitudinal profiles → from seminar 4
- perhaps cross-sections from seminars 1 – 3

Tasks:

1. Approximate the two irregular cross-sections by regular forms! Estimate the watered cross-sectional areas and the cross-sectional areas up to the beginning of the flooded area!

2. Please calculate the wetted perimeter for both cross-sections! Indicate the wetted perimeter for the water levels for the measured water levels (measurement results of seminars 1 – 4) and for the situation where flooding starts!

3. Estimate the mean water depth for both cross-sections also for the water levels for the measured water levels (measurement results of seminars 1 – 4) and for the situation where flooding starts!

4. Calculate the hydraulic radii for the areas of the two investigation sites!

5. Estimate the river ground slope for the areas of the two investigation sites!

6. Dissipate statements regarding compacted or segmented cross-sections on base of cross-sectional measurements in case of a flood event with a high return period were flood peak has a height of more than 1 m above streambed! What means that in general (qualitative) for hydraulic calculations?

7. Which accuracy has data out of remote sensing investigations regarding river geometry and channel segmentations? Which information are derivable by means of remote sensing data?

Hydrological seminar 6:

Stream flow hydraulics 6 – Streaming and shooting flow

Objective:

Estimation of the kind of water movement (streaming or shooting flow) at the investigation sites of seminars 1 – 4

Methodology:

→ see lecture notes "Hydrology III", chapter 1.1

Necessary information:

- mean flow velocities, cross-sectional areas and width of water levels → from seminars 1 – 4
- flow velocity coefficient → see table 1.1

Tasks:

1. Existing streaming or shooting flow conditions at time of discharge measurements of seminars 1 – 4?

2. Which amount the mean flow velocities must have in case of shooting flow under the given conditions (cross-sections and width of water levels at the moment of measurements)?

3. Which amount the river ground slopes must exist to get flow velocities high enough for shooting flow (in accordance to seminar 2)? Please use GMS formula (equation 1.19)! Estimate STRICKLER coefficient by using table 1.2 and photos in chapter 1.7!

4. Is shooting flow possible if the creeks bed would get a river training out of smooth down cement respectively smoothes concrete?

Hydrological seminar 7:

Stream flow hydraulics 7 – Vegetation mapping and photo documentation in the field

Objective:

Vegetation record and photo documentation of creek Kleinwaltersdorfer Bach in the area between road Hainichener Straße in direction of café´ ”Waldcafe´” and district Freiberg-Neufriedburg from hydraulic point of view

Methodology → see lecture notes “Hydrology III”, chapter 1.4.4

Measurement equipment:

- digital camera with photo numbering
- measuring tape (maximum 30 m long)
- folding ruler
- levelling rod
- chest waders, gumboots

Measurements prearrangement and procedure:

- walk at course of stream (in creek bed)
- vegetation mapping section by section
- indication and notation of mapping points in a small-scale map
- vegetation categorisation (vegetation groups respective chapter 1.4.4)
- vegetation measurement
- preparation of a photo documentation → plot the photo points in a small-scale map
- generalisation regarding to vegetation sections

Tasks:

1. Please plot the vegetation mapping points and the points of photo documentation in the map section in figure Ü 7.2 (general map → see figure Ü 7.1)! In case of photo documentation please plot the points and the photograph direction! Please document all observations detailed as if you will never come back to the mapping points! Please mention date and time!

2. Categorise the vegetation according to vegetation groups from hydraulic point of view corresponding to chapter 1.4.4!

3. Measure the vegetation respective horizontal extension and growth height (the latter in case of trees in dependence on the height regarding to stem, thick branches, leaves) up to a level which is flood relevant! Please register the measured values in table Ü 7.1! Please include special hydraulic conditions to photo documentation!

4. Please realise a generalisation to vegetation sectors and mark them in figure 7.4! Identify and mark special critical areas from hydraulic point of view!

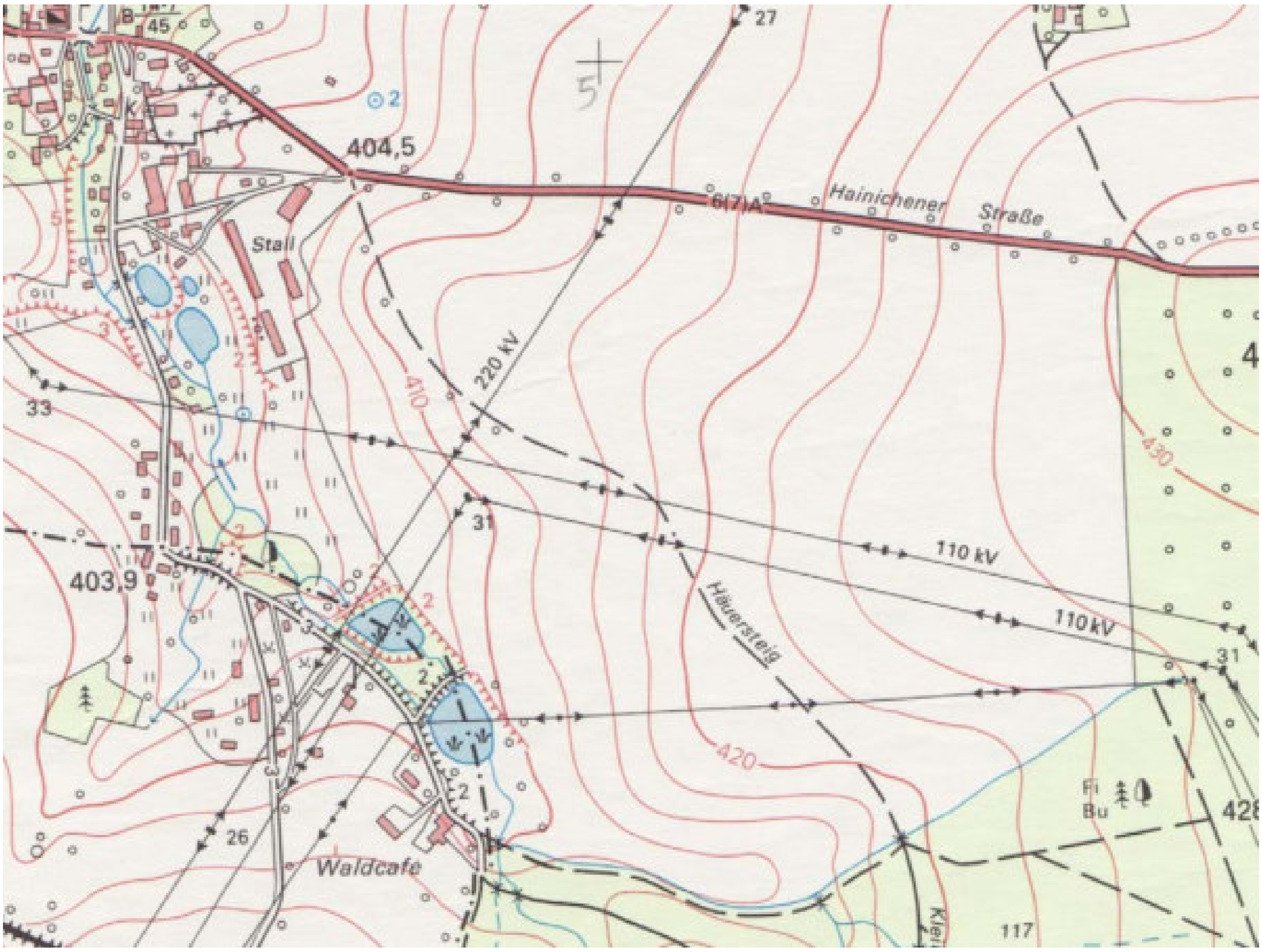


Fig. Ü 7.1: General map of the investigation area

Fig. Ü 7.2: Map sections for vegetation mapping points (continuation → see next page)

Fig. Ü 7.2: Map sections for vegetation mapping points

Fig. Ü 7.3: Map sections for photo documentation (continuation → see next page)

Fig. Ü 7.3: Map sections for photo documentation

Fig. Ü 7.4: Map sections with vegetation sectors (continuation → see next page)

Fig. Ü 7.4: Map sections with vegetation sectors

Table Ü 7.1: Vegetation mapping results (*continuation → see next page)*

Mapping point No.	Kind of vegetation	Vegetation group respective chapter 1.4.4	Hydraulic case respective chapter 1.4.4 (if vegetation group 2)	Horizontal extension	Vegetation height (in case of trees differentiated)	No. of photograph

Continuation of table Ü 7.1: Vegetation mapping results (continuation → see next page)

Mapping point No.	Kind of vegetation	Vegetation group respective chapter 1.4.4	Hydraulic case respective chapter 1.4.4 (if vegetation group 2)	Horizontal extension	Vegetation height (in case of trees differentiated)	No. of photograph

Continuation of table Ü 7.1: Vegetation mapping results

Mapping point No.	Kind of vegetation	Vegetation group respective chapter 1.4.4	Hydraulic case respective chapter 1.4.4 (if vegetation group 2)	Horizontal extension	Vegetation height (in case of trees differentiated)	No. of photograph

Hydrological seminar 8:

Stream flow hydraulics 8 – Mapping of hydraulic narrows in the field

Objective:

Mapping and photo documentation of hydraulic narrows in creek Kleinwaltersdorfer Bach upstream the bridge on local connection road to federal highway B 101

Methodology:

→ see lecture notes "Hydrology III", chapters 1.4.1 and 1.4.2

Measurement equipment:

- digital camera with photo numbering
- measuring tape (maximum 30 m long)
- folding ruler
- levelling rod
- chest waders, gumboots

Measurements prearrangement and procedure:

- walk at course of stream (in creek bed)
- sift of hydraulic narrows
- indication and notation of hydraulic narrows in a small-scale map
- hydraulic narrows measurements
- preparation of a photo documentation

Tasks:

1. Please plot the hydraulic narrows in map Ü 8.1! Note the mapping date!

2. Measure the hydraulic narrows (cross-sections, extension) and record the values into table Ü8.1!

3. Characterise the hydraulic conditions in the narrows areas regarding to the used construction materials and surface roughnesses (in case of bridges, pipe passages and something like that). In case of narrows because of vegetation please characterise the vegetation characteristics (analogue seminar 7. here in seminar 8 distinctive hydraulic narrows only)!

4. Please prepare a photo documentation of hydraulic narrows!

5. Which hydraulic narrow is on your opinion the most critical? Please involve the cross-section and the surface roughness into the evaluation!

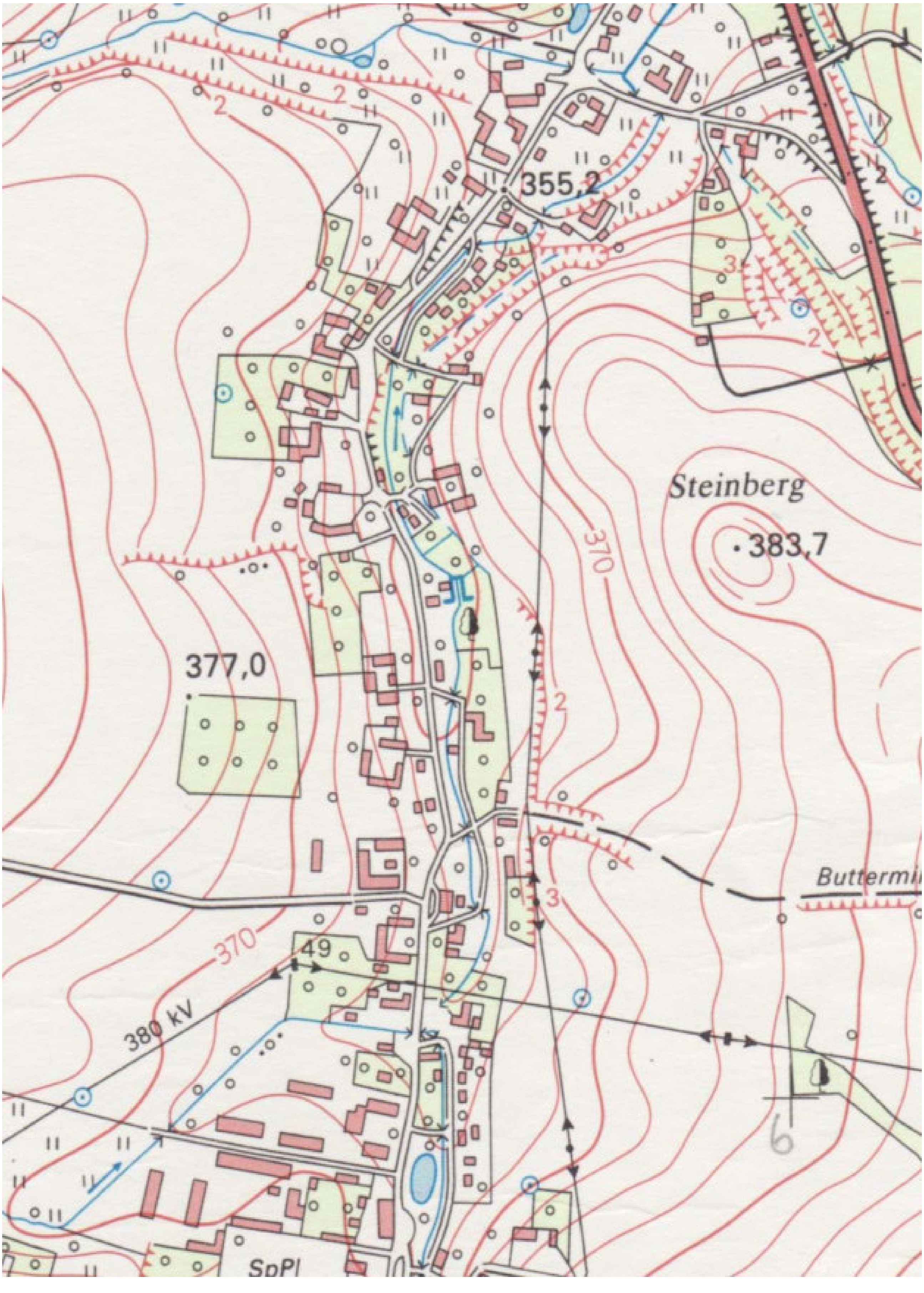
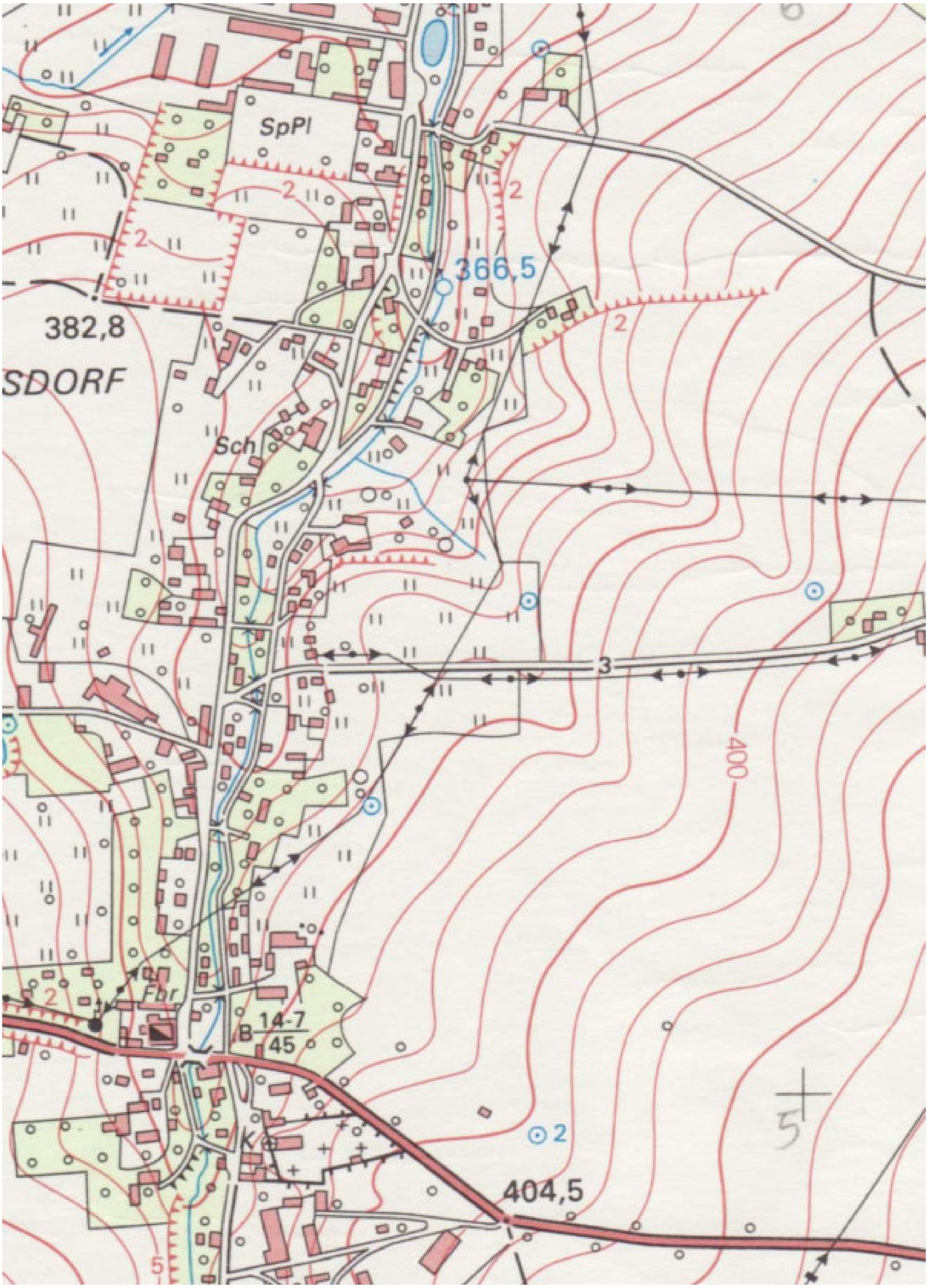


Fig. Ü 8.1: Map of the northern part of the investigation area (southern part → next page)



Continuation of figure Ü 8.1: Map of the southern part of the investigation area

Table Ü 8.1: Mapping results regarding to hydraulic narrows

Mapping point No.	Kind of hydraulic narrow	Width in direction of inflow to the narrow	Width at the smallest part of the narrow	Narrow height (if bridge, pipe passages, …)	Hydraulic characteristic (used construction materials, kind of vegetation, …)	No. of photograph

Hydrological seminar 9:

Stream flow hydraulics 9 – Hydraulic parameter determination on base of filed experiments

Objective:

Photograph and documentation of hydraulic roughnesses at creek Kleinwaltersdorfer Bach upstream the bridge on local connection road to federal highway B 101

Methodology:

→ see lecture notes "Hydrology III", chapters 1.4.3 and 1.7

Measurement equipment:

- digital camera with photo numbering
- folding ruler
- chest waders, gumboots

Measurements prearrangement and procedure:

- walk at course of stream (muss not take place in creek bed)
- mapping of hydraulic roughnessees separately for river bed, embankments and forelands
- indication and notation of creek sections with nearly uniform hydraulic roughnesses in a small-scale map
- preparation of a photo documentation
- assignment of STRICKLER coefficients

Tasks:

1. Please plot the creek sections with nearly uniform hydraulic roughnesses in the map Ü 9.1! Note the mapping date!

2. Estimate STRICKLER coefficients on base of tables given in chapter 1.4.3 an in picture series in chapter 1.7 separately for river bed, embankments and forelands! In case of forelands assume a flood peak of maximum 1 m above creek bed. Register the observations and the estimated STRICKLER coefficients in table Ü 9.1! Document the creek sections by photos also separately for river bed, embankments and forelands!

3. Which creek section is on your opinion the most critical?

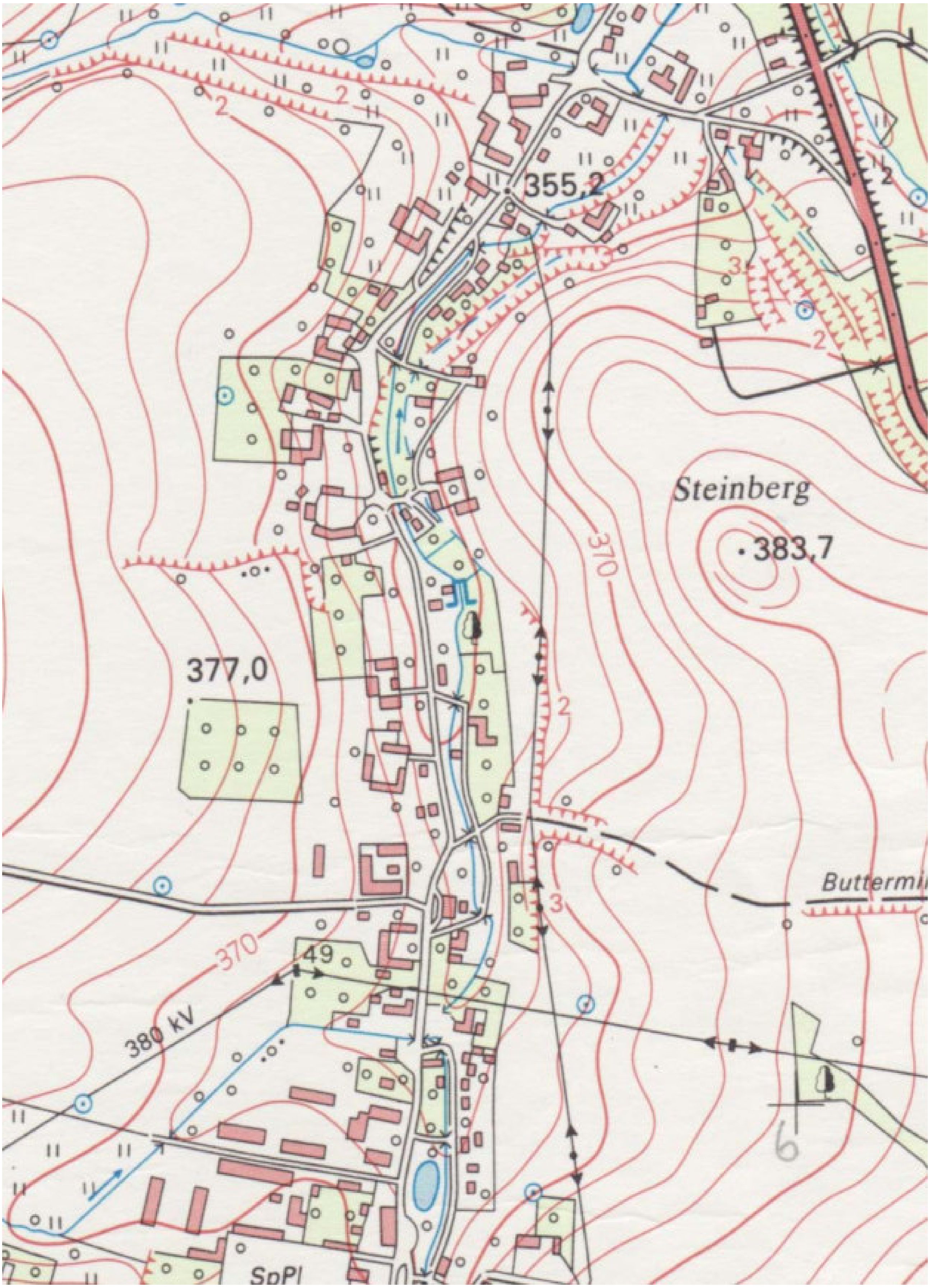


Fig. Ü 9.1: Map of the northern part of Kleinwaltersdorf village (southern part → next page)

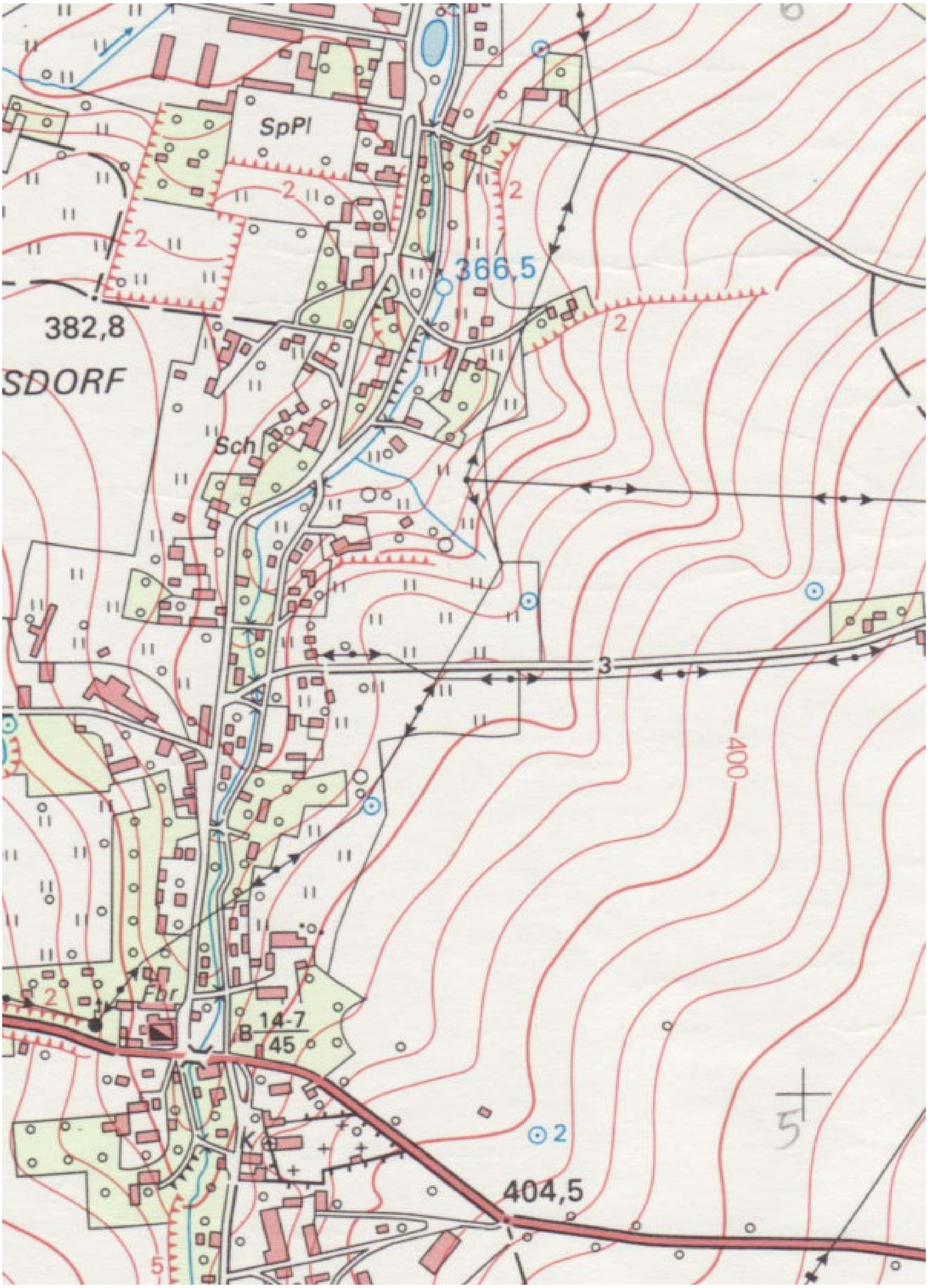


Fig. Ü 9.1: Map of the southern part of Kleinwaltersdorf village (southern part → next page)

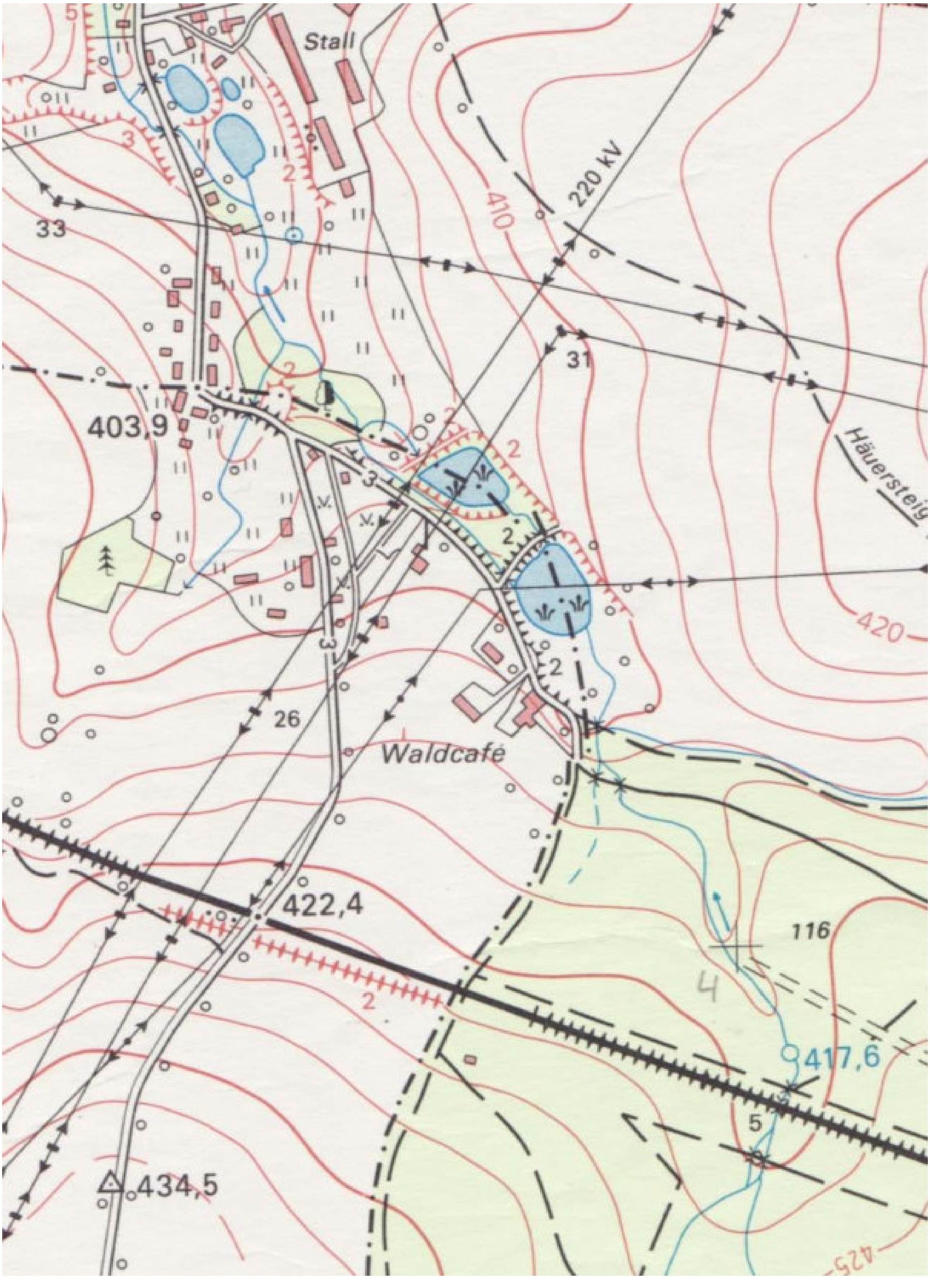


Fig. Ü 9.1: Map of café´"Waldcafe" area (southern part → next page)

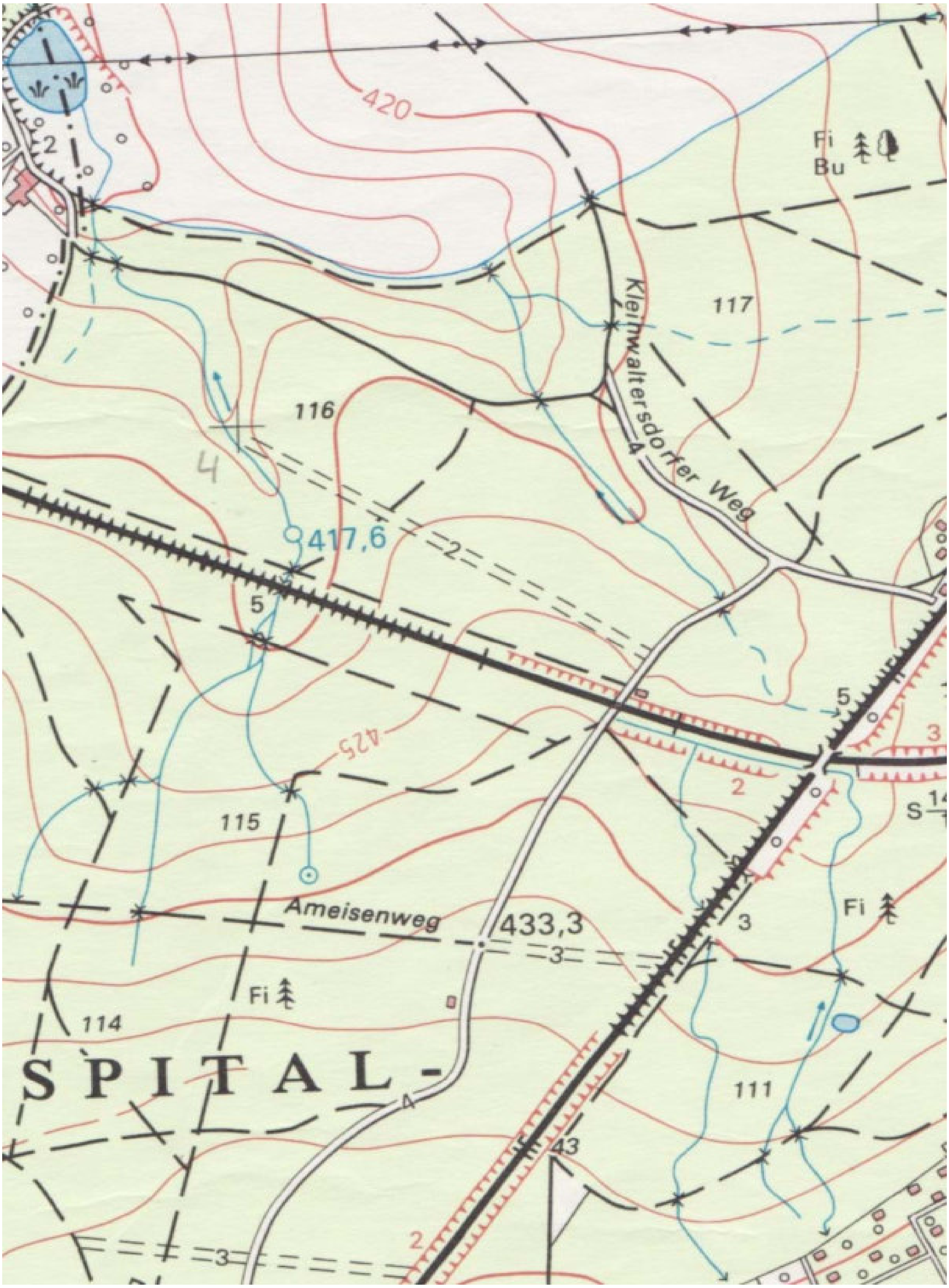


Fig. Ü 9.1: Map of creek Kleinwaltersdorfer Bach spring area

Table Ü 9.1: Mapping results of STRICKLER coefficients

Mapping point No.	Hydraulic characteristics	STRICKLER coeff. k_{St} [$m^{1/3}/s$] on creek ground	STRICKLER coeff. k_{St} [$m^{1/3}/s$] left embankment	STRICKLER coeff. k_{St} [$m^{1/3}/s$] right embankm.	STRICKLER coeff. k_{St} [$m^{1/3}/s$] left foreland	STRICKLER coeff. k_{St} [$m^{1/3}/s$] right foreland	No. of photograph

Hydrological seminar 10:

Stream flow hydraulics 10 – Calibration of roughness coefficients by means of discharge measurement results

Objective:

Calibration of STRICKLER roughness coefficients k_{St} by using of discharge and measuring results for both measuring points from seminars 1 to 5

Methodology:

→ see lecture notes "Hydrology III", chapters 1.4.3.1 and 1.5.1.1

Necessary information:

- mean flow velocity and discharge values → from seminars 1 – 3
- hydraulic radii→ from seminar 5
- creek ground slope → from seminar 5

Tasks:

1. Estimate STRICKLER coefficients for both measuring points by using the developed information from seminars 1 to 5!

2. Compare and discuss the results! Especially look at the differences of both discharge measurement methods (weir, tracer) near the bridge beside café´ "Waldcafe´". Include the estimated STRICKLER coefficients from seminar 9 in the discussion!

3. Please try a calibration of the following COWAN factors: characterisation of river bed material, irregularities of river bed material, changes in cross-sectional figuration, influence of barriers within the watered cross-section, vegetation influences and meander degree, even if the Problem occurs, that you have to parameterise an equation with several unknown variables. Please work out a plausible solution!

4. Please try to modify the calibrate STRICKLER coefficients by means of cross-sectional weighted discharges to include differences in regard to hydraulic roughnesses of creek ground and embankments! Please use for this task the EINSTEIN / HORTON method!

Hydrological seminar 11:

Stream flow hydraulics 11 – Estimation of equivalent sand roughness by sieve analysis

Objective:

Equivalent sand roughness estimation of a given sediment sample by means of characteristic values out of a grain size distribution curve

Methodology:

→ see lecture notes "Hydrology III", chapter 1.4.3.2

Working steps:

- set of sieves composition by considering of grain gradation (visual sediment classification), notation of sieve sizes in table Ü 11.1 (column 1)

Table 11.1: Overview on experimental determined values by means of sieve analysis

Sieve size [mm] (1)	Mass [g] of empty sieve or container (2)	Mass sieve + sediment [g] (3)	Sediment retained [g] (4) = (3) – (2)	Sediment retained [%] (5)	Percent passing (6)
pan					
Total sediment weight:			g	=	100 %

- Weights of sieves and container (lowermost container), results notation in table Ü 11.1 (column 2)
- estimation of total sediment weight, results notation in table Ü 11.1. last table row
- pouring the sediment sample into the top sieve and realisation of the sieve experiment, duration: in dependence on available time, maximum 30 min
- weighting of each sieve with its retained sediment (columns 3 and 4), calculation of the sediment retained in % of total mass (column 5), calculation of recent passing related to total mass (column 6)
- plot of grain size distribution (→ see figure Ü 11.1) and take readings of the values which are necessary of equivalent sand roughness

For help → see for example:
http://www.uic.edu/classes/cemm/cemmlab/Experiment%206-Grain%20Size%20Analysis.pdf

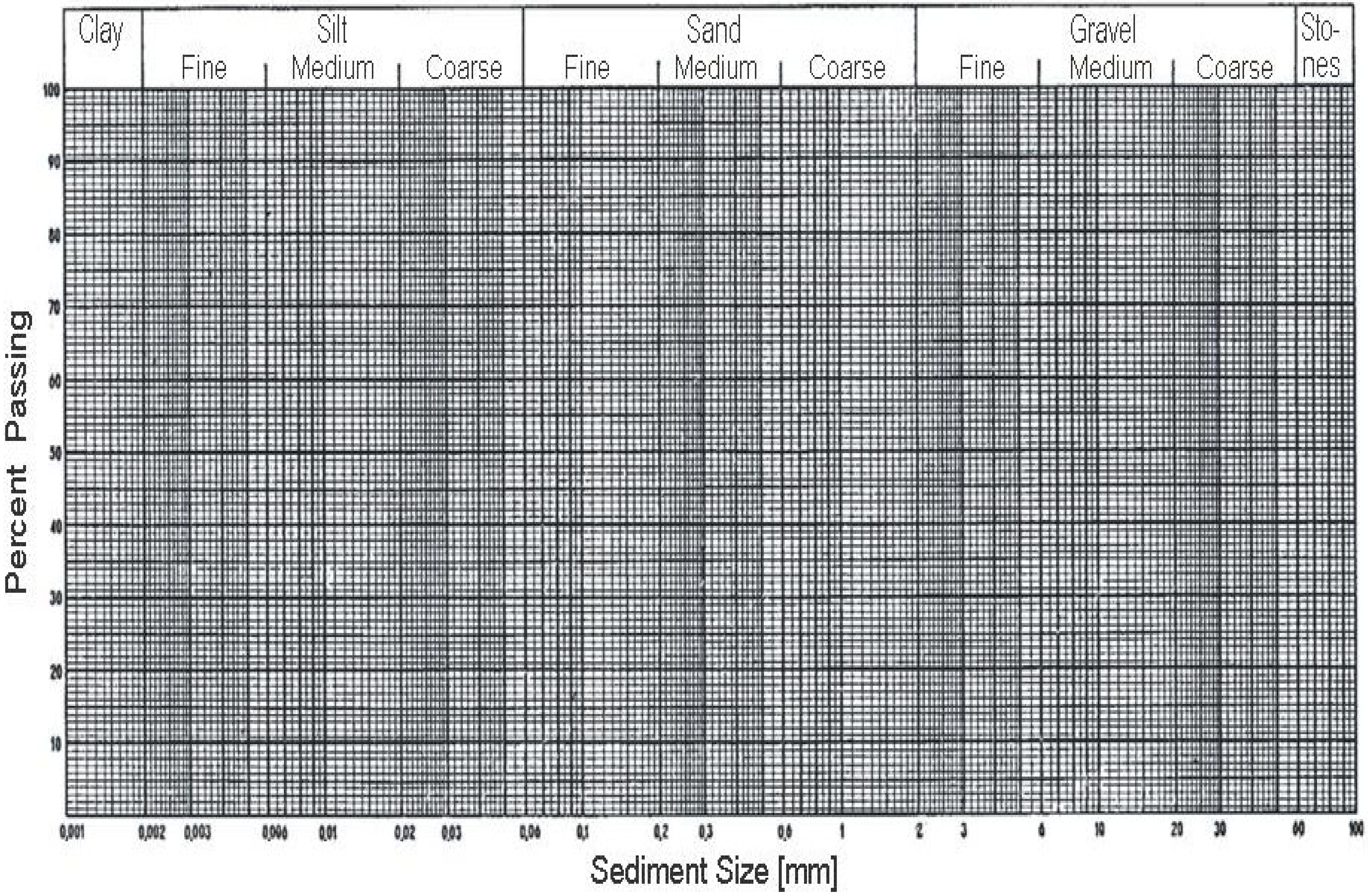


Fig. Ü 11.1: Grain size distribution curve

Tasks:

1. Please document all working steps regarding the grain size distribution curve preparation for the given sediment sample!
2. Calculate equivalent sand roughness by means of all methods you find in table 1.5 of “Hydrology III” lecture notes!

Hydrological seminar 12:

Stream flow hydraulics 12 – Estimation of hydraulic roughnesses by empirical methods (COWAN, EINSTEIN / HORTON, SELLIN)

Objective:

Estimation of hydraulic roughness coefficients by STRICKLER k_{St} for both measurement points of seminars 1 to 5 for two hydraulic conditions: i) water level at the top of river bed and ii) 1 meter above river bed (flooding of forelands) by means of empirical methods by COWAN, EINSTEIN / HORTON, and SELLIN

Methodology:

→ see lecture notes "Hydrology III", chapter 1.5.1.1

Necessary information:

- cross-sectional measurement results → from seminars 4 and 5
- vegetation measurement results → from seminars 7 and possibly 8
- wetted perimeters and hydraulic radii→ from seminars 5 and 9
- channel segmentation indications → from seminar 5
- photos of photo documentations → from seminars 7 – 9

Tasks:

1. Estimate for both cross-sections (bridges on local connection road to federal highway B 101 and beside café´ "Waldcafe´") STRICKLER roughness coefficients by COWAN weighting method! Please document all assumptions regarding to COWAN factors! Apply the COWAN method for the two hydraulic conditions: i) water level at the top of river bed and ii) 1 meter above river bed (flooding of forelands)!

2. Calculate discharges for both cross-sections and both water levels by means of GMS formula by using STRICKLER roughness coefficients which you have determined in seminar 1!

3. Calculate discharges for both cross-sections and both water levels by means of GMS formula by using STRICKLER roughness coefficients which are weighted by EINSTEIN / HORTON or (if necessary) by SELLIN! Explain the application of the used weighting method!

4. Compare the results respectively STRICKLER roughness coefficients, flow velocities and discharges and interpret them! Look for reasons regarding results differences!

5. Which method or which methods are in your opinion most suitable with respect to existing hydraulic conditions to calculate absolutely reliable flow velocities and discharges?

Hydrological seminar 13:

Stream flow hydraulics 13 – Flow velocity and discharge calculations for different boundary conditions by GMS formula

Objective:

Flow velocity and discharge calculations by GMS formula for different objectives and boundary conditions

Methodology:

→ see lecture notes "Hydrology III", chapter 1.5.1

Given information:

- river cross-section → see figure Ü 13.1
- ground and embankments materials:
  - → river ground: coarse gravel, here and there small stones
  - → river embankments and forelands: loam
- vegetation:
  - → river ground: without any vegetation
  - → river embankments: reeds (perennial, mean height: 1.5 m)
  - → left foreland: groups of trees, bushes (mean height: 3 m)
  - → right foreland: single trees, stem diameter approximately 0.5 m, grass vegetation
- river ground slope: 0.2 %
- maximum river expansion: 11 m above river ground (see figure Ü 13.1)

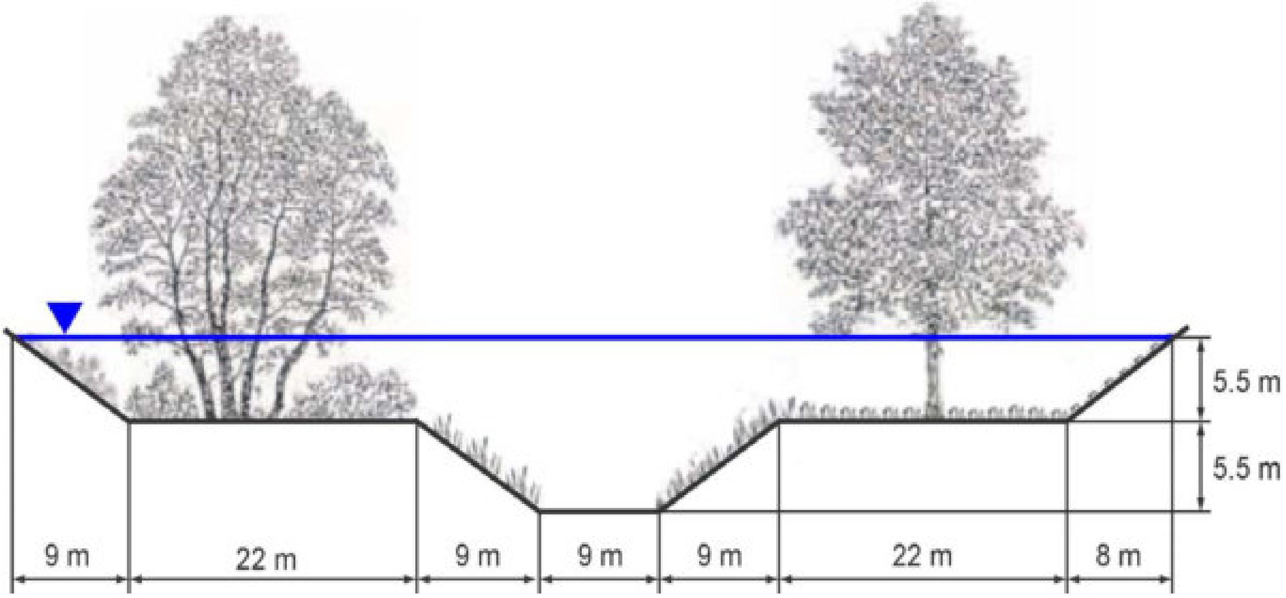


Fig. Ü 13.1: River cross-section

Tasks:

1. Estimate the following discharge values:
   - maximum discharge within river bed
   - maximum discharge in the whole river cross-section including forelands

   Please document all assumptions, methods and calculation steps! Does exists an erosion risk in the forelands? Is the application of GMS formula allowed under the given conditions? Explain your decision!

2. Calculate the water level where half of the total discharge (maximum discharge in the whole river cross-section including forelands) estimated in task 1 is flowing off!

   Please document all assumptions, methods and calculation steps!

Hydrological seminar 14:

Stream flow hydraulics 14 – Flow velocity and discharge calculations by COLEBROOK & WHITE

Objective:

Flow velocity and discharge calculation by COLEBROOK – WHITE method for a rectangle river profile by using equivalent sand roughness k_S from seminar 11

Methodology:

→ see lecture notes "Hydrology III", chapters 1.3.2 and 1.5.2

Given information:

- compact, non-segmented flow cross-section, 24 m wide
- maximum water level (river bed only, without forelands): 1.7 m
- equivalent sand roughness → see results of seminar 11
- river ground slope: 0.15 %
- no relevant vegetation
- straight river pathway, nearly no meander

Tasks:

1. Please estimate the maximum river discharge inclusive of variations result by using different methods for equivalent sand roughness calculation! Please document all assumptions, methods and calculation steps!
2. Calculate the water level where half of the total discharge estimated in task 1 is flowing off! Please document all assumptions, methods and calculation steps!
3. Is the application of COLEBROOK – WHITE method allowed under the given conditions? Explain your decision!

Hydrological seminar 15:

Flood analysis 1 – Application of stochastic conception

Objective:

Estimation of flood peak discharges with return periods of 10, 50 and 100 years by means of statistical analysis of measured annual peak discharges

Methodology:

→ see lecture notes "Hydrology III", chapter 4.3.3

Given information:

- annual flood peak discharge values HQ(a) for an observation series from 1951 – 1990 → see table Ü 15.1

Table Ü 15.1: Annual maximum values of peak discharges HQ(a)

Year	HQ(a) [m^3/s]	Year	HQ(a) [m^3/s]	Year	HQ(a) [m^3/s]	Year	HQ(a) [m^3/s]
1951	298	1961	980	1971	174	1981	453
1952	1020	1962	529	1972	227	1982	218
1953	417	1963	118	1973	436	1983	595
1954	241	1964	593	1974	1190	1984	336
1955	288	1965	219	1975	577	1985	632
1956	143	1966	868	1976	647	1986	445
1957	774	1967	768	1977	480	1987	354
1958	652	1968	693	1978	932	1988	623
1959	266	1969	392	1979	436	1989	319
1960	506	1970	227	1980	615	1990	477

Tasks:

1. Calculate the nonexeedance probabilities and record them in table Ü 15.2 and in distribution function diagram of extreme value distribution type I (→ see figure Ü 15.1)!

2. Determine the flood peak discharges with return periods of T = 10 y, 50 y and 100 y by means of the following methods:
   - free adjustment
   - moment method
   - GUMBEL method

3. Indicate the maximum extrapolation range from hydrological point of view!

4. Evaluate the results regarding the following aspects:
   - For which hydrologic design tasks the return periods of 10 y, 50 y and 100 y are relevant?
   - For which values (for which method) do you decide! Explain your decision!
   - Which strategy do you suggest in case of a determination of a flood peak discharge HQ(200)?

Table Ü 15.2: Nonexeedance probability of annual flood peak discharges, series 1951 – 1990

Order m [ ]	Annual flood peak discharge HQ(a) [m^3/s]	Nonexeedance probability P_u [%]
1		
2		
3		
4		
5		
6		
7		
8		
9		
10		
11		
12		
13		
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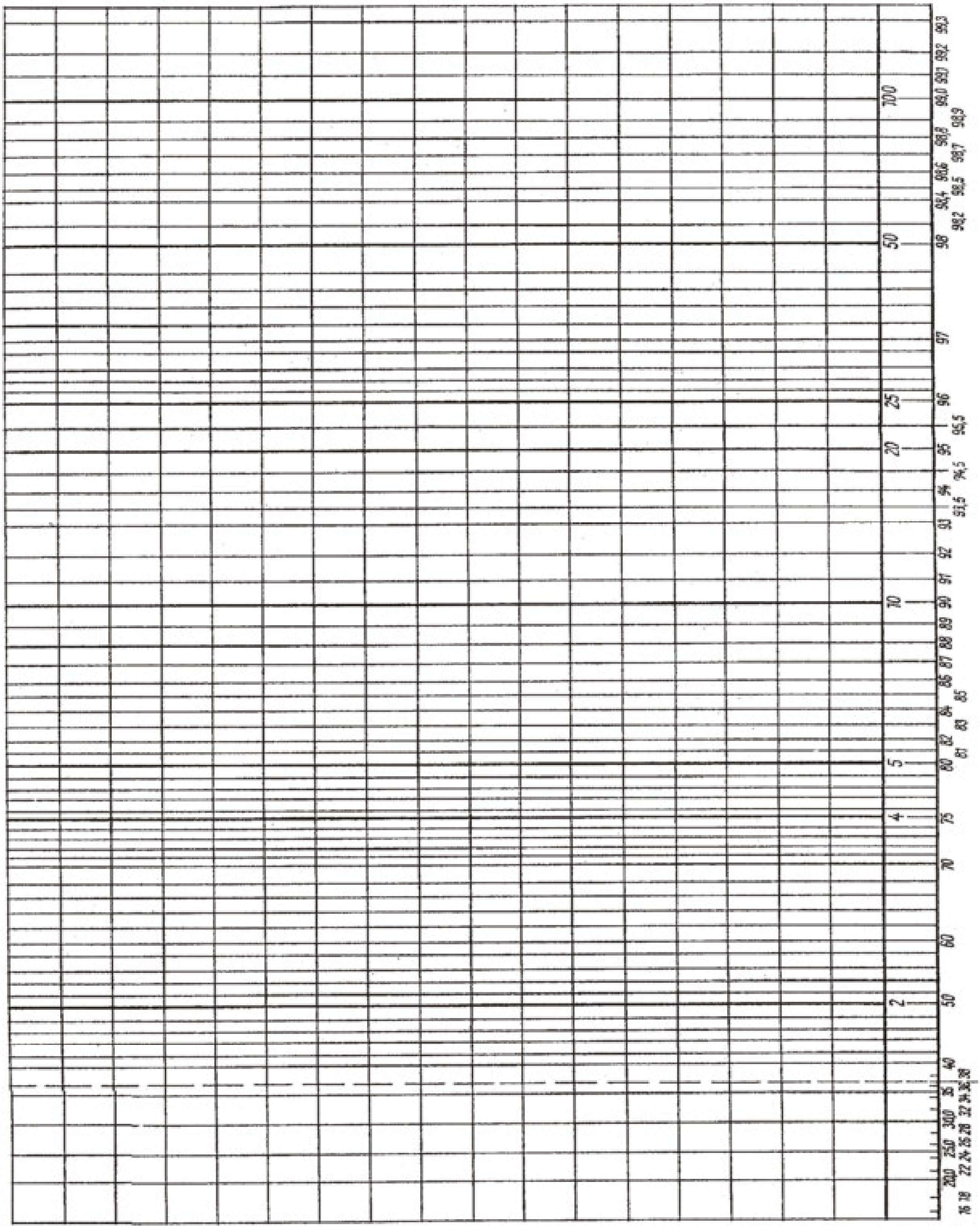


Fig. Ü 15.1: Probabilities of the annual maximum flood discharges HQ, series 1951 – 1990 and adjusted probability distributions in EI distribution function diagram

Hydrologic seminar 16:

Flood analysis 2 – deterministic conception 1: Estimation of infiltration ability of the soil surface (in-situ and laboratory methods)

Objective:

Determination of the infiltration capacity at the soil surface near Otto Meißer building by means of double ring infiltrometer in-situ and in the laboratory by means of DARCY conductivity test

Methodology:

→ see lecture notes “Hydrology I”, chapter 5.6.3 or lecture notes “Hydrogeology”

In-situ experiment (double ring infiltrometer):

Location sketch of the experimental sites:

Sketch of experimental setup:

Working steps in-situ infiltration experiment:

- selection of a test site with grass cover
- carefully hammer down of the two infiltrometer rings
- impoundment of the outer infiltrometer ring, impoundment height maximum 2 – 3 cm, refilling during the experiments if necessary!
- realization of a constant impoundment height during the experiment of maximum 2 – 3 cm in the inner infiltration ring (permanent control!)
- registration of the infiltration amounts at regular time intervals
- The experiment has to run as long as stationary conditions are observed (constant infiltration rates).

Working steps DARCY conductivity test in the laboratory:

- extraction of a undisturbed and representative sediment sample in the field (undisturbed = conservation of the natural bulk density) → hammer down of a core sampler in the soil → expose and close the core sampler corresponding figure Ü 16.1

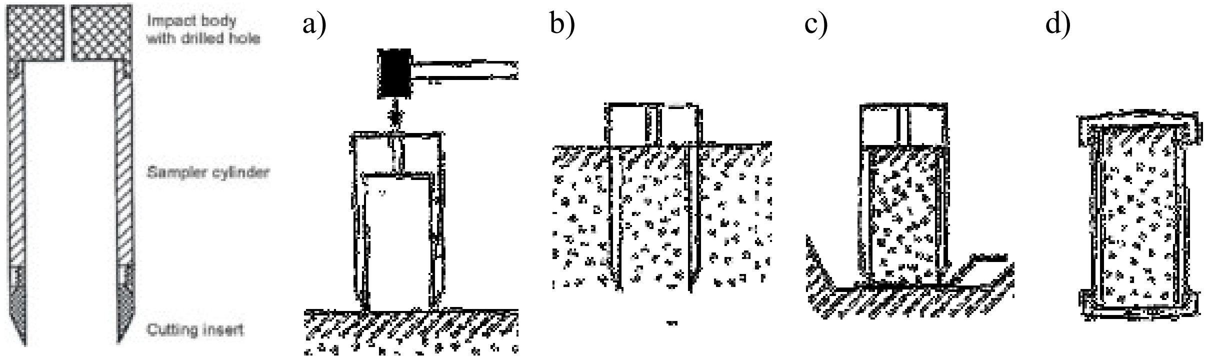


Fig. Ü 16.1: Sediment sampling by means of sample cylinder

- estimation of the sampler cylinder dimensions (diameter and length)
- Installation of the sample in the flow apparatus (experimental setup → see figure Ü 16.2)

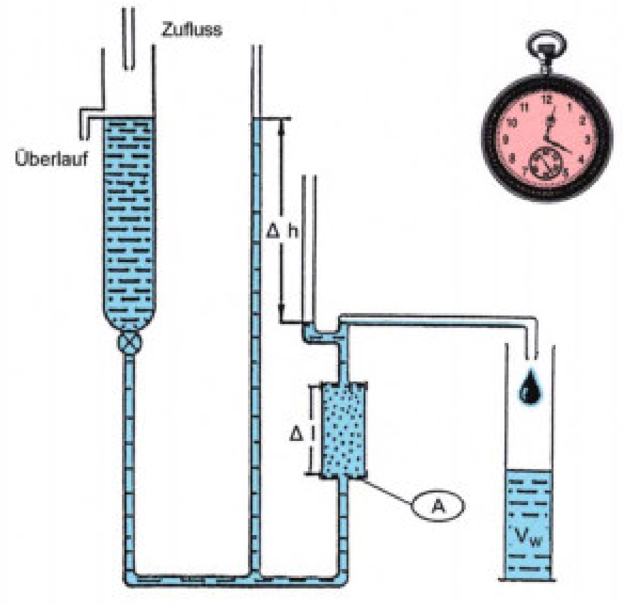


Fig. Ü 16.2:

Experimental design of the flow test apparatus with constant pressure level to deter-mine the saturated hydraulic conductivity (k_f value)

- saturation of the sample with water (The experimental setup guarantees a water saturation from the bottom upwards, so that the air can escape completely.)
- realization of the experiment with a constant pressure head (see also Figure Ü 16.2):
  → experimental conditions:
    - stationary flow conditions (constant flow)
    - $d_{Cylinder} = (10 \ldots 20)\, d_{max,\ Sample}$
    - Hydraulic gradient $\Delta h / \Delta l = 0.1 \ \ldots\ 0.2$ m/m → laminar flow
    - Complete saturation of the sample at the beginning
    - T_{Water} = constant
    - permeability of the filter layer (sample cylinder is inserted between two filter) should be at least 100 times higher than the k_f value of the sample

  → test execution:
    - setting of a $\Delta h / \Delta l = 0.1 \ldots 0.2$ m/m
    - wait for stationary conditions

  → calculation of k_f using equation Ü 16.1 (application of DARCY's law):

$$k_f = \frac{Q \cdot \Delta l}{A \cdot \Delta h} \qquad (\text{Ü } 16.1)$$

where:
k_f - filtration coefficient [m/s]
Q - flow rate [m^3/s]
A - flow area (= cylinder area) [m^2]
Δl - sample length [m]
Δh - pressure head difference [m]

→ note: Start the measurements after complete sample saturation!
There must be no air (air bubbles) in the system!
→ setting the pressure difference Δh at U-manometer by adjusting the height at the inlet into the glass measuring cylinder (consider the compliance of the experiment preconditions!)
→ target of a water volume V, which is to be measured (for example V = 50 ml, 100 ml ...)
→ close the valve at the outlet of the glass measuring cylinder
→ registering the time that is needed to reach the predetermined volume of water
→ after finishing the experiment → accurately gauging of the water volume V in the measuring glass cylinder

Tasks:

1. Calculate the k_f values that result according to the two tests!

2. Interpret the values in terms of magnitude and in terms of differences between the two tests!

3. Discuss sources of error and compliance with the test conditions!

4. What method and therefore which value would you prioritize if the data would be needed for the parameterization of a rainfall-runoff model?

Please note: sample after finishing the experiment → dry box → is still needed for Seminar 17!

Hydrological seminar 17:

Flood analysis 3 – deterministic conception 2: Estimation of effective and total porosity

Objective:

Determination of effective and total porosity for the same sample which was used in seminar 16 (location besides the building Otto-Meißer-Bau) by means of ZUNKER apparatus

Methodology (see also Hydrogeology course materials):

- determination of total porosity:

$$n = 1 - \frac{V_s}{V_g} = 1 - \frac{m_s}{V_g * \rho_s} \qquad \textit{(Ü 17.1)}$$

Where: n - total porosity [ ]
V_s - volume of solids [cm^2]
V_g - total volume [cm^3]
m_s - dry mass of the sediment sample [g]
ρ_s - sediment true density ($\rho_s = m_s / V_s$) [g/cm^3]

→ estimation of m_s:

  - removal of the pore water by drying the sample in an oven at T = 105 ° C to constant mass → then weighting

→ estimation of ρ_s:

  - ρ_s is assumend as constant, because of a very small range:
    → sands: 2.63 ... 2.65 g/cm^3
    → clays: 2.65 ... 2.80 g/cm^3

→ estimation of V_g: for soil samples in most cases cylinder dimensions → volume calculation by cylindrical formula

- estimation of effective porosity n_e and the residual water content n_r by ZUNKER apparatus:

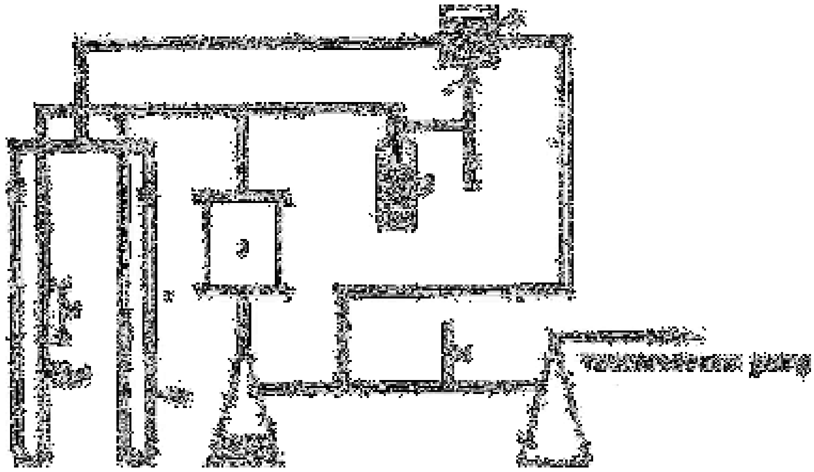
→ construction of ZUNKER apparatus → see figure Ü 17.1

→ determination algorithm:

  - drying of the sediment sample at 105 ° C to constant mass
  - determination of dry mass m_s
  - determination of the total porosity by formula Ü 17.1
  - saturation of the sediment sample with water (from bottom to top)
  - extraction of the gravitational water at defined negative pressure (duration about 30 min)
  - determination of wet mass m_m
  - determination of residual water content (adhesive water) n_r by equation Ü 17.2:

$$n_r = \frac{V_{HW}}{V_g} = \frac{m_m - m_s}{\rho_w \cdot V_g} \qquad \textit{(Ü 17.2)}$$

where: n_r - residual water content [ ]
V_{HW} - adhesive water volume (residual water volume) [cm^3]
V_g - total volume [cm^2]
m_m - wet mass [g]
m_s - dry mass [g]
ρ_w - water density [g/cm^3]



1 - Core cylinder with undisturbed sample
2 - wash bottle
3 - wet gravel filter
4 - crushing clip for vacuum regulation

Fig. Ü 17.1:

Schematic illustration of ZUNKER apparatus

→ estimation of effective porosity n_e:

$$\boldsymbol{n_e = n - n_r} \tag{Ü 17.3}$$

where: n - total porosity [ ]
n_e - effective porosity [ ]
n_r - residual porosity [ ]

Working steps:

- drying of the sediment to be tested at 105 ° C to constant mass (already realized)
- determining the dimensions of the sample cylinder (diameter, length) and calculation of the total volume V_g
- weighing the dry sample → determination of the dry mass m_S
- sample saturation (from bottom to top → avoid air inclusions)
- Extraction of water from the drainable pores by ZUNKER apparatus:
  → installation of water-saturated sample into the ZUNKER apparatus
  → generating a negative pressure which corresponds to pF 1.8 (corresponds to 0.5 m water column)
  → field capacity → boundary between effective porosity and residual porosity
    - vacuum regulation by means of crushing clip
    - vacuum registration by means of U-manometer (p_1)
  - sample drainage by constant negative pressure until the moment where drain is finishing
- once again sample weighting → estimation of wet mass m_m
- emptying and drying of the sample cylinder and weighting of the cylinder and of all screwing and seals

Tasks:

1. Determine the total porosity, the residual water content and the effective porosity!

2. Characterize the soil with regard to water transport and water storage characteristics! How would you rate on the basis of the determined values (including the k_f value of seminar 16), the reaction of the soil in regard to surface runoff during heavy rain events?

Hydrological seminar 18:

Flood analysis 4 – deterministic conception 3: Regionalisation methods I

Objective:

Estimation of flood peak discharges for an unobserved catchment west of urban region of Jena (ditch Gollichsgraben including dry valley Münchenrodaer Grund up to the outlet to river Leutra) by means of envelope curve after WUNDT and "Rational Formula"

Methodology:

→ see lecture notes "Hydrology III", chapter 4.3.4.2

Given information:

- map of the investigation area with indication of the calculation point (lowest point of the catchment area) → see figure Ü 18.1

Tasks:

1. Locate the surface catchment on base of the given map (figure Ü 18.1) and determine the catchment area of ditch Gollichsgraben including dry valley Münchenrodaer Grund up to the outlet to river Leutra!

2. Estimate the flood peak discharge HQ(100) for a return period of 100 years by means of envelope curve after WUNDT!

3. Determine the HQ (100) value by means of the modified rational formula for Bavaria! Determine based on the map of the study area (Figure Ü 18.1) the necessary information! Regarding the precipitation amounts, you can have recourse to KOSTRA values represented in table Ü 18.1.

   Table Ü 18.1: KOSTRA heavy rain values P [mm] for Jena region for an event with a return period of 100 years

Rain duration	5 min	10 min	15 min	20 min	30 min	45 min	60 min	90 min	2 h
Rain amount [mm]	16.9	22.4	26.5	29.8	35.3	41.7	47.0	50.7	53.5
Rain duration	3 h	4 h	6 h	9 h	12 h	18 h	24 h	48 h	72 h
Rain amount [mm]	57.7	60.9	64.7	71.0	75.0	82.5	90.0	101.0	110.0

4. Compare and interpret the flood peak values estimated by the two methods!

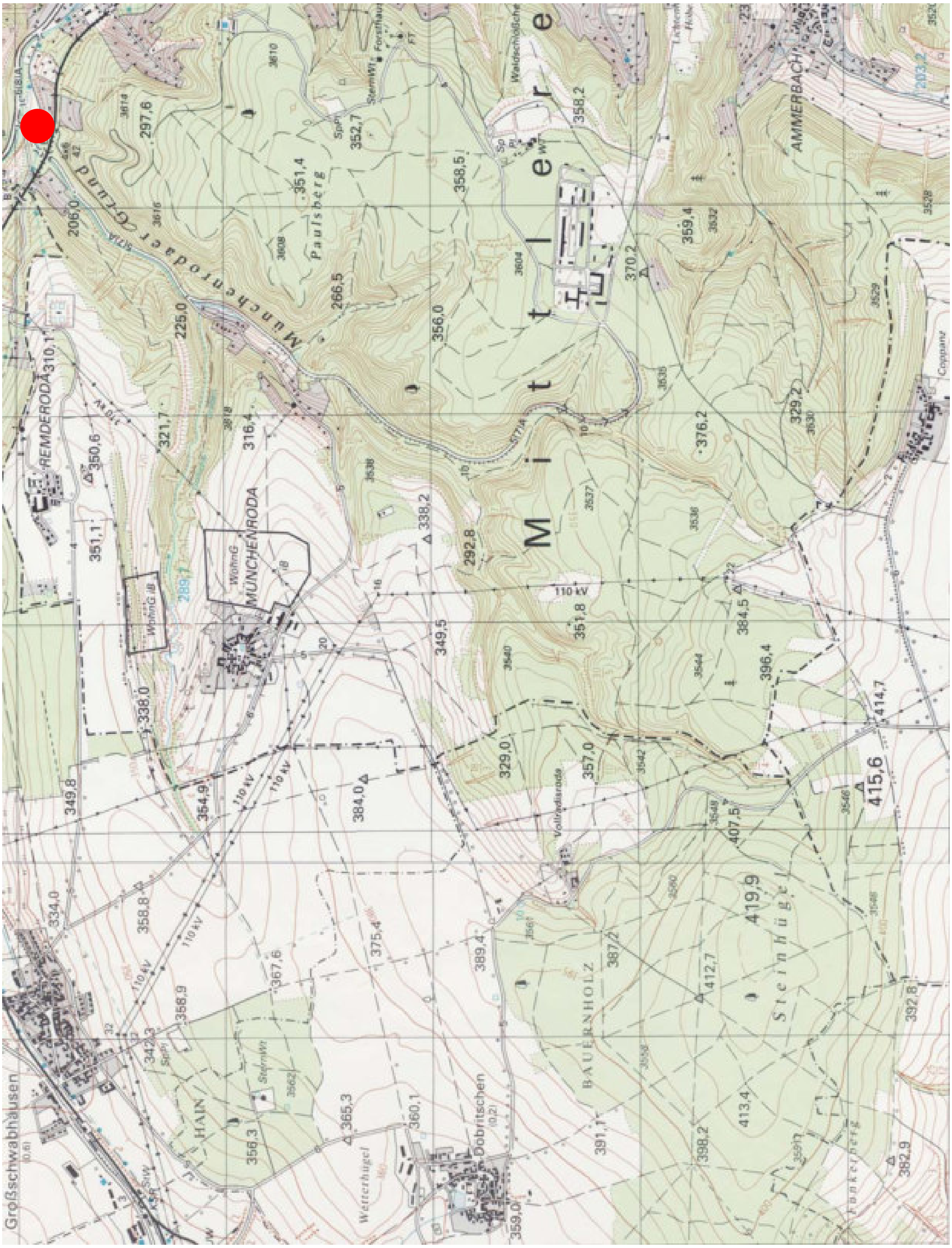


*Fig. Ü 18.1: Map of the investigation area with calculation point (1 quadrate in the map = 1 * 1 km)*

Hydrological seminar 19:

Flood analysis 5 – deterministic conception 4: Regionalisation methods II

Objective:

Estimation of flood peak discharges for an unobserved catchment west of urban region of Jena (ditch Gollichsgraben including dry valley Münchenrodaer Grund up to the outlet to river Leutra) by means of index flood method and extended flood index method after LAUTERBACH/GLOS and estimation of mean discharge by application of NAU atlas (precipitation-discharge-difference atlas of former GDR)

Methodology:

→ see lecture notes "Hydrology III", chapter 4.3.4.2

Given information:

- analogue seminar 18

Tasks:

1. Determinate flood peak discharges with return periods of 5, 10, 20, 25, 50 and 100 years by application of index flood method! For the estimation of the reference value HQ(2.33) you can use approximately the dependences that have been derived for the river Weisse Elster.

2. Determinate flood peak discharges with return periods analogue task 1 by extended flood index method after LAUTERBACH/GLOS! To determine the length of the stream (ditch Gollichsgraben) you can use the figure Ü 18.1. With regard to the determination of the catchment gradient it is sufficient, when you estimate the gradient for each grid crossing point (figure Ü 18.1). The horizontal distance along the highest gradient should be about 200 – 300 m. Enter the determined values in table Ü 19.1! the local factors α and β are given in figure Ü 19.1.

3. Estimate the mean discharge MQ for the catchment on base of total runoff information R of NAU atlas (precipitation-discharge-difference atlas) of former GDR! Figure Ü 19.2 contains information on total runoff R. Document the solution process! As the average flow value is classified in comparison to the following regions:
   - German average
   - East German average
   - German regions with the lowest discharge values
   - German regions with the lowest discharge values

   Use for classification the given values in lecture notes "Hydrology I" (chapter 2.2) and "Hydrology II" (chapter 1.2.4)! What would be the mean discharge values (in l/s) for watersheds of this size in the above called regions?

4. Compare and interpret the by means of regionalisation methods estimated flood peak discharges including the results of seminar 18! Which or what procedures would you prioritize? Which or what procedures seem to be not suitable, to specify flood peak discharges!

Table Ü 19.1: Catchment gradient (estimated at the grid crossing points of figure Ü 18.1

Crossing point No. horizontally	Crossing point No. vertically	Length difference Δl [m]	Height difference Δl [m]	Catchment gradient [m/m]
			Mean:	

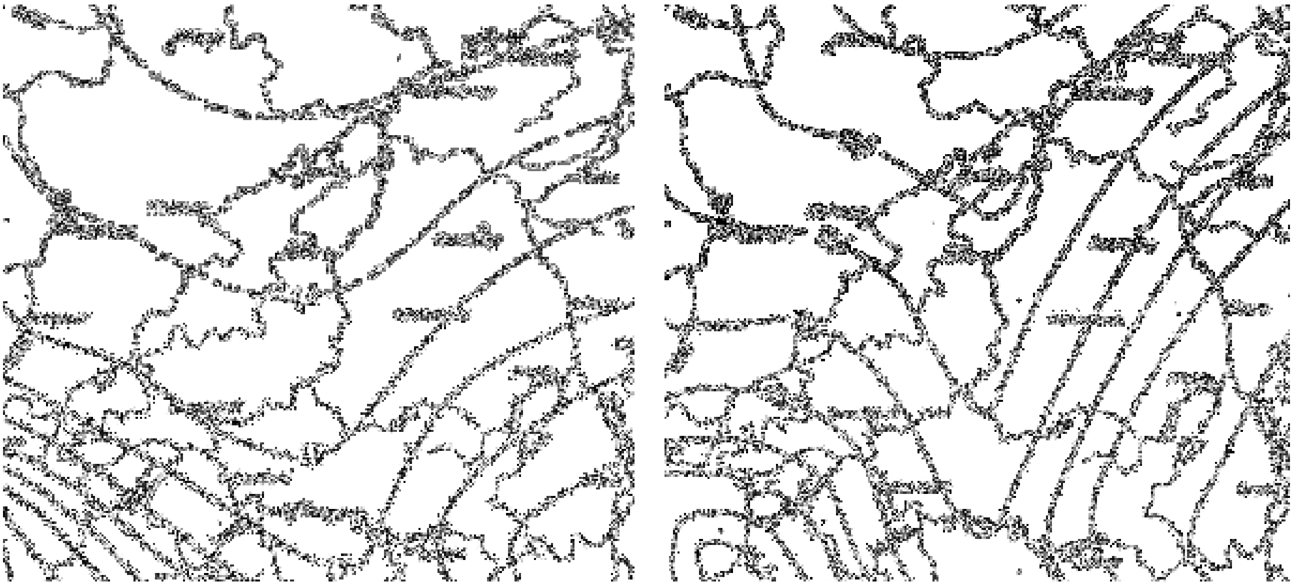


Fig. Ü 19.1: ***Local factors of LAUTERBACH-GLOS method fort he investigation area***

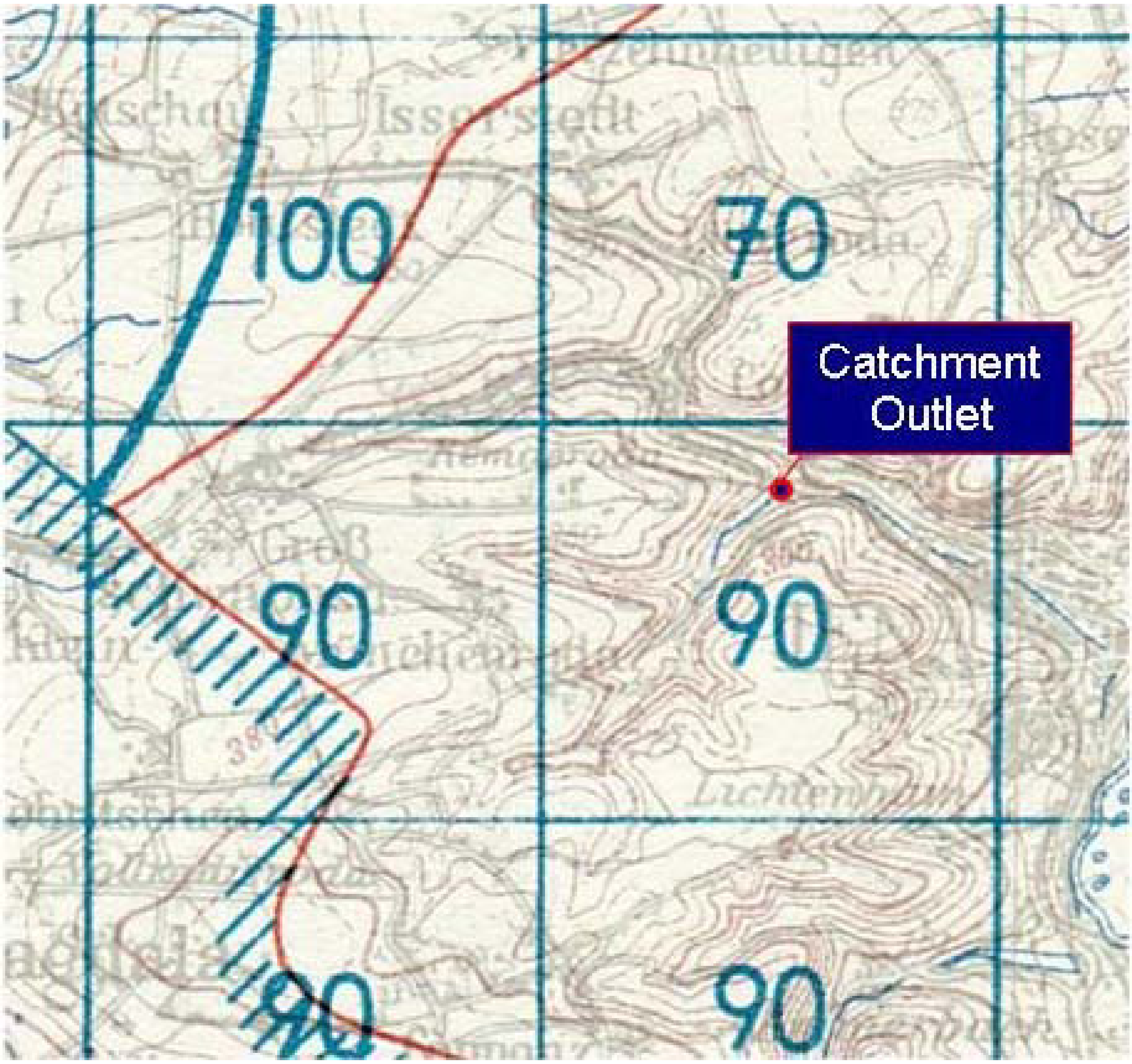


Fig. Ü 19.2: Total runoff R (after NAU atlas of former GDR)

Hydrological seminar 20:

Flood analysis 6 – deterministic conception 5: Estimation of design rainfall by means of KOSTRA atlas

Objective:

Estimation of design rainfall values for different durations and different return periods for the unobserved catchment west of urban region of Jena (ditch Gollichsgraben including dry valley Münchenrodaer Grund up to the outlet to river Leutra) by means of KOSTRA atlas of German Weather Survey DWD

Methodology:

→ see lecture notes "Hydrology I", chapter 3.5 and lecture notes "Hydrology II", seminar 2

Given information:

- DWD programme KOSTAB (design rainfall calculation programme of German Weather Survey)

Tasks:

1. Determine for the selected location heavy rainfall amounts with durations of 5 min to 72 h and return period of 0.5 to 100 years by means of KOSTRA atlas of German Weather Survey!

   proceed as follows:
   - determination of KOSTRA coordinates for the location (pdf file Uebung20-KOSTRA-Karten)
   - start of KOSTAB programme (Uebung20-KOSTAB.EXE)
   - reading the heavy rainfall amounts for the rainfall durations and return periods given by the programme (pdf file Uebung20-KOSTRA-Karten)
   - input of the rainfall data in KOSTAB programme → Note!: input „." (dot) instead of „," (comma) for decimal numbers! Do not a typing error During the processing, otherwise restart and re-entry!
   - after program execution (Last value: rainfall amount for a rainfall duration of 72 hours with a return period of 100 years) → creation of a results table (file-name: table) → view possible by using Notepad or loading into text or spreadsheet programs

2. Filter out the rainfall amounts for the indicated periods of rainfall recurrence intervals from 5, 20 and 100 years and enter the values in the table Ü 1.20!

3. Determine for the 3 return period the heavy rainfall mounts , taking into account the tolerance amounts specified by German Weather Survey DWD:
   - Amount of tolerance ± 10 % for return periods ≤ 5 years
   - Amount of tolerance ± 15 % for return periods > 5 … ≤ 50 years
   - Amount of tolerance ± 20 % for return periods > 50 years

   Enter the results in table Ü 20.1!

Table Ü 20.1: KOSTRA heavy rainfall amounts P [mm] for rainfall durations between 5 minutes and 72 hours for return periods of 5, 20 and 100 years (including amounts of tolerance)

Return period T	T = 5 a			T = 20 a			T = 100 a		
Amount of tolerance	- 10 %	± 0 %	+ 10 %	- 15 %	± 0 %	+ 15 %	- 20 %	± 0 %	+ 20 %
Duration D = 5 min									
D = 10 min									
D = 15 min									
D = 20 min									
D = 30 min									
D = 45 min									
D = 60 min									
D = 90 min									
D = 2 h									
D = 3 h									
D = 4 h									
D = 6 h									
D = 9 h									
D = 12 h									
D = 18 h									
D = 24 h									
D = 48 h									
D = 72 h									

Hydrological seminar 21:

Flood analysis 7 – deterministic conception 6: Parametrisation of a conceptual rainfall runoff model

Objective:

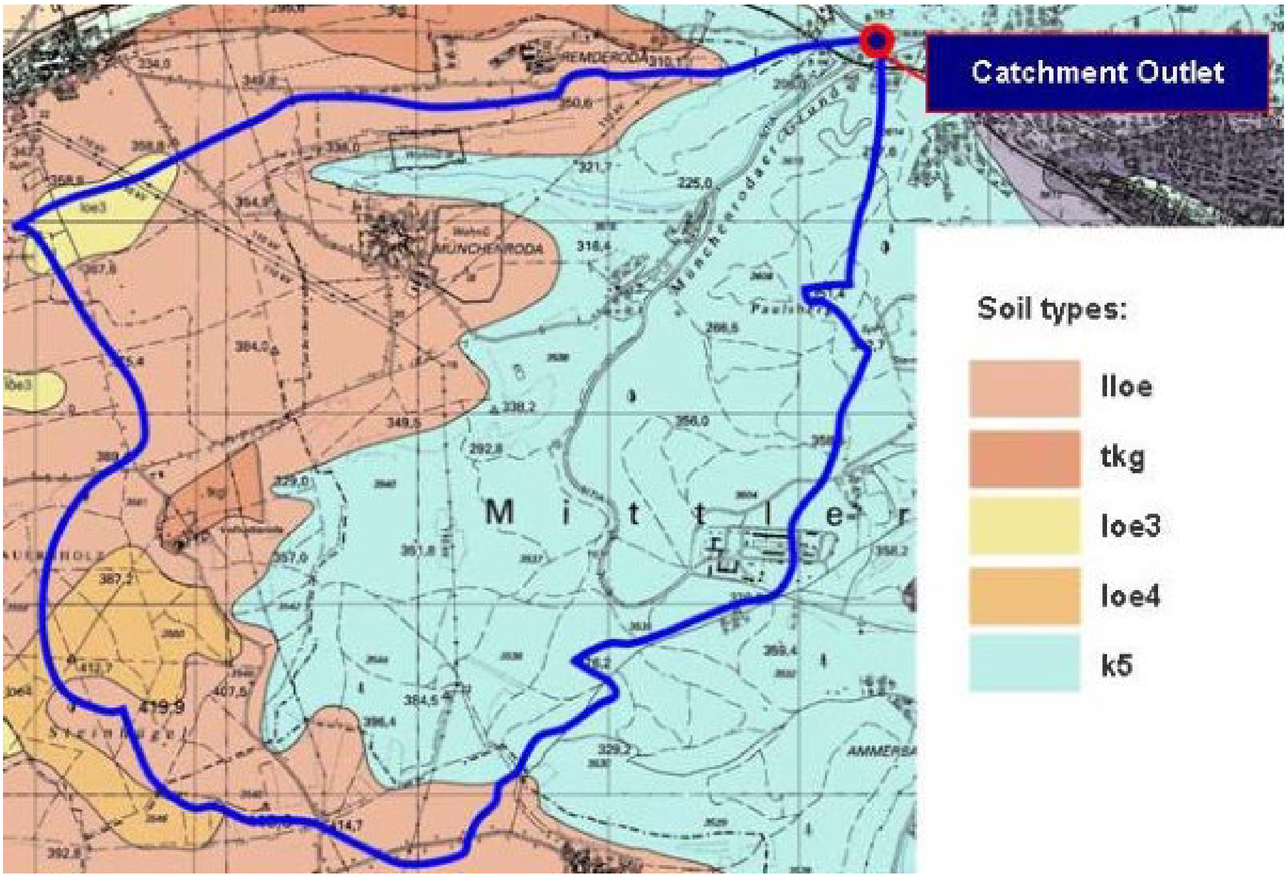
Creation of the conceptual rainfall-runoff model HQBEMESS for the unobserved catchment west of urban region of Jena (ditch Gollichsgraben including dry valley Münchenrodaer Grund up to the outlet to river Leutra) and modelling of a 100-year flood event

Methodology:

→ see lecture notes "Hydrology III", chapter 4.3.4.3 and lecture notes "Hydrology I", chapter 5.6.2

Given information:

- topography of the study area from seminar 18
- KOSTRA heavy rainfall amounts from seminar 20
- main soil types in the study area → see figure Ü 21.1



lloe – loess loam   tkg - clay, loamy clay, very stony

loe3 – loess chernozem (clayey)   loe4 – loess podzoluvisol   k5 - loam, very stony

Fig. Ü 21.1: Main soil types in the study area

- hydrological Characteristics of the main soil types regarding Curve Number CN → see table Ü 21.1

Table 21.1: Curve Number for the main soil types in the catchment area

Main soil type	Sign	Curve Number type
Loess loam	lloe	2
Clay, loamy clay, very stony	tkg	4
Loess chernozem (clayey)	loe3	3
Loess podzoluvisol	loe4	3
Loam, very stony	k5	2

- main types of land use in the study area → see figure Ü 21.2

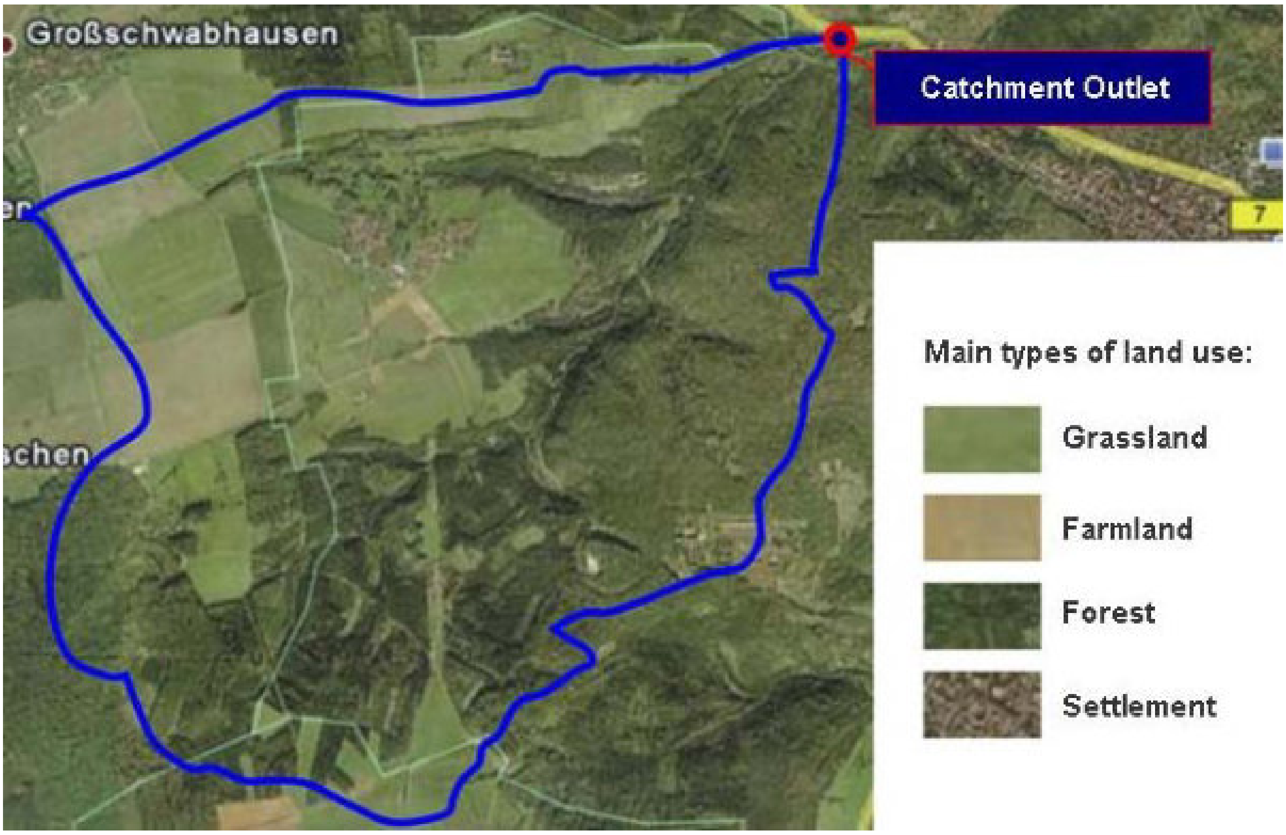


Fig. Ü 21.2: Main types of land use in the study area

- catchment size, distances, altitude differences → see seminars 18 and 19
- design rainfall duration: unknown
- rainfall amounts corresponding to seminar 20 (table 20.1, without tolerances)
- initial soil content of the area at the time the rain starts: medium (usage of the model internal soil moisture adaption)
- temporal variability of precipitation: no variability (constant rain intensity)

Tasks:

1. Create the conceptual rainfall-runoff model HQBEMESS for the unobserved catchment west of urban region of Jena (ditch Gollichsgraben including dry valley Münchenrodaer Grund up to the outlet to river Leutra)!

   Proceed as follows:

   - start the programme HQBEMESS (folder Uebung21-Programm) → creation of a new input file
   - identification of the main land use percentages in the catchment (digital by means of map sheet georeferencing of figure Ü 21.2 or analog by planimetry) → You can do this also by using jpg file „Hauptnutzungen“ in folder „Sonstiges“ or by using the software GOOGLE EARTH.
   - assignment of the main land uses of the HQBEMESS usages:
     - → Make assumptions about the distribution of agricultural land with respect to cereals, forage crops and root crops!
     - → Make assumptions about the distribution of grassland to permanent grassland and meadow (roughly estimation of grazing animals in 0. Näherung by means of GOOGLE EARTH)!
     - → Subdivide the forest land with regard to its vegetation density!
     - → Make assumptions about the sealing degree of the settlements! The unsealed settlement areas are classificable to permanent grassland.
   - assignment of representative curve-number types (see table Ü 21.1) to the main land use types (possibly area-weighted averaging)
   - enter the KOSTRA rainfall amounts for a return period of 100 years
   - explanation of the definition of the kind of runoff concentration and of the landscape character of the considered catchment

2. Write down all the main parameter assumptions, input data and necessary steps towards their estimation!

3. Model the 100-year flood event! You can find the model results in the folder „Ergebnis“, file name corresponding to the selected identification number. Interpret the results respectively to the following criteria:

   - Is the model internal calculated design rainfall duration plausible?
   - Is the model internal calculated runoff coefficient plausible?
   - Is the wave form plausible (plot the discharge wave by means of EXCEL)?
   - Compare and interpret the flood peak discharges which have been estimated by means of rainfall-runoff model HQBEMESS in comparison to the values from regionalisation methods of seminars 18 and 19!

Hydrological seminar 22:

Flood analysis 8 – deterministic conception 7: Runoff modelling for different rainfall distributions, rainfall tolerance ranges and return periods

Objective:

Quantification of the influence of different rainfall distributions, rainfall ranges of tolerance and return periods on the shape of the hydrograph, and the height of the flood peak in case of a heavy rainfall event by means of the conceptual rainfall-runoff model HQBEMESS for the unobserved catchment west of urban region of Jena (ditch Gollichsgraben including dry valley Münchenrodaer Grund up to the outlet to river Leutra)

Methodology:

→ see lecture notes "Hydrology III", chapter 4.3.4.3

Given information:

- input and output files from seminar 21 according to the conditions in the catchment area for a constant rain intensity with 100 years return period
- KOSTRA heavy rainfall amounts from seminar 20

Tasks:

1. Model the hydrographs and flood peak values of a 100-year storm event for the following rainfall distributions:
   - distribution after DVWK (German society for water management)
   - PECHER rainfall distribution with maximum rainfall intensity at the beginning
   - PECHER rainfall distribution with maximum rainfall intensity at the end

   Describe the event-based process of the rainfall intensities for the three rainfall distributions! Enter the model results with respect to flood peak discharges HQ and flood rise time t_S in the table Ü 22.1! Underline the rainfall distribution, for which the largest peak being modeled!

   Table 22.1: Model results for different rainfall distributions

Rainfall distribution	HQ [m^3/s]	T_S [min]
Constant rain intensity (from seminar 21)		
Distribution after DVWK		
PECHER rainfall distribution with maximum intensity at the beginning		
PECHER rainfall distribution with maximum intensity at the end		

   Interpret the model results in regard to flood peak discharges, flood rise times and wave forms (graphical plots for example by EXCEL)!

2. Quantify the uncertainties that arise from the KOSTRA heavy rainfall values! Enter the flood peak discharges for the 100-year storm event taking into account the tolerances of KOSTRA heavy rain amounts! Use therefore the precipitations from seminar 20 (table Ü 20.1)! Enter the model results with respect to the flood peak discharges HQ in table Ü 22.2!

Table 22.2: Flood peak discharges HQ for different rainfall distributions considering KOSTRA heavy rain tolerances

Rainfall distribution	HQ [m^3/s] for P = 80 %	HQ [m^3/s] for P = 0 % (from table 22.1)	HQ [m^3/s] for P = 120 %
Constant rain intensity			
Distribution after DVWK			
PECHER rainfall distribution with maximum intensity at the beginning			
PECHER rainfall distribution with maximum intensity at the end			

Discuss the results from the following points:

- Is the correlation between flood peak discharge and heavy rain amount linear? What factors are responsible for the rainfall - flood peak discharge - behaviour?
- What element of uncertainty (rainfall distribution or rain amount) is prevail in regard to result uncertainty?

3. Select the rainfall distribution, which produces the greatest flood peak discharge! Use the 0 % tolerance precipitation only! Model for these rainfall distribution flood peak discharges with return period of 1, 5, 10, 20 and 50 years! Use the KOSTRA heavy rains from seminar 20! Enter the modeled results in the table Ü 22.3!

Table 22.3: Flood peak discharges HQ in dependence on return period for the rainfall distribution with the greatest flood peak s

Rainfall distribution	T = 1 a	T = 5 a	T = 10 a	T = 20 a	T = 50 a

Discuss the values! Compare the flood peak values modelled by rainfall-runoff model HQBEMESS with the results of the seminars 18, 19 and 21!

Hydrological seminar 23:

Flood analysis 9 – deterministic conception 8: Runoff modelling for different rainfall durations

Objective:

Quantification of the influence of different rainfall durations (constant rain intensity, return period: 100 years) on hydrograph and level of flood peak discharge by means of the conceptual rainfall-runoff model HQBEMESS for the unobserved catchment west of urban region of Jena (ditch Gollichsgraben including dry valley Münchenrodaer Grund up to the outlet to river Leutra)

Methodology:

→ see lecture notes "Hydrology III", chapter 4.3.4.3

Given information:

- input and output files from seminar 21 according to the conditions in the catchment area for a constant rain intensity with 100 years return period
- KOSTRA heavy rainfall amounts from seminar 20

Tasks:

1. Model the hydrographs and flood peak values of a 100-year storm event for all rainfall durations according to KOSTRA atlas (rainfall durations between 5 minutes and 72 hours)! Enter the model results with respect to the flood peak discharges HQ and flood rise times t_S in table Ü 23.1! Mark the rainfall duration with the greatest flood peak discharge!

Table 23.1: Model results for different rainfall durations D (constant rain intensity, return period: 100 years)

D [min]	5	10	15	20	30	45	60	90
HQ [m^3/s]								
t_S [min]								
D [min]	120	180	240	360	540	720	1080	1440
HQ [m^3/s]								
t_S [min]								

2. Interpret the model results in regard to flood peak discharges, flood rise times and wave forms (graphical plots for example by EXCEL)!

Hydrological seminar 24:

Flood analysis 10 – deterministic conception 9: Runoff modelling by considering of flow routing effects (isochrone method)

Objective:

Sub-basin differentiated modelling of a 100-year flood event, taking account of run-time effects using the conceptual rainfall-runoff model HQBEMESS for the unobserved catchment west of urban region of Jena (ditch Gollichsgraben including dry valley Münchenrodaer Grund up to the outlet to river Leutra)

Methodology:

→ see lecture notes "Hydrology III", chapter 4.3.4.3

Given information:

- topography of the study area from seminar 18
- KOSTRA heavy rainfall amounts from seminar 20
- main soil types in the study area from seminar 21
- hydrological characteristics of the main soil types in regard to curve-number type from seminar 21
- main types of land use in the study area from seminar 21
- initial soil content of the area at the time the rain starts: medium (usage of the model internal soil moisture adaption)
- temporal variability of precipitation: no variability (constant rain intensity)

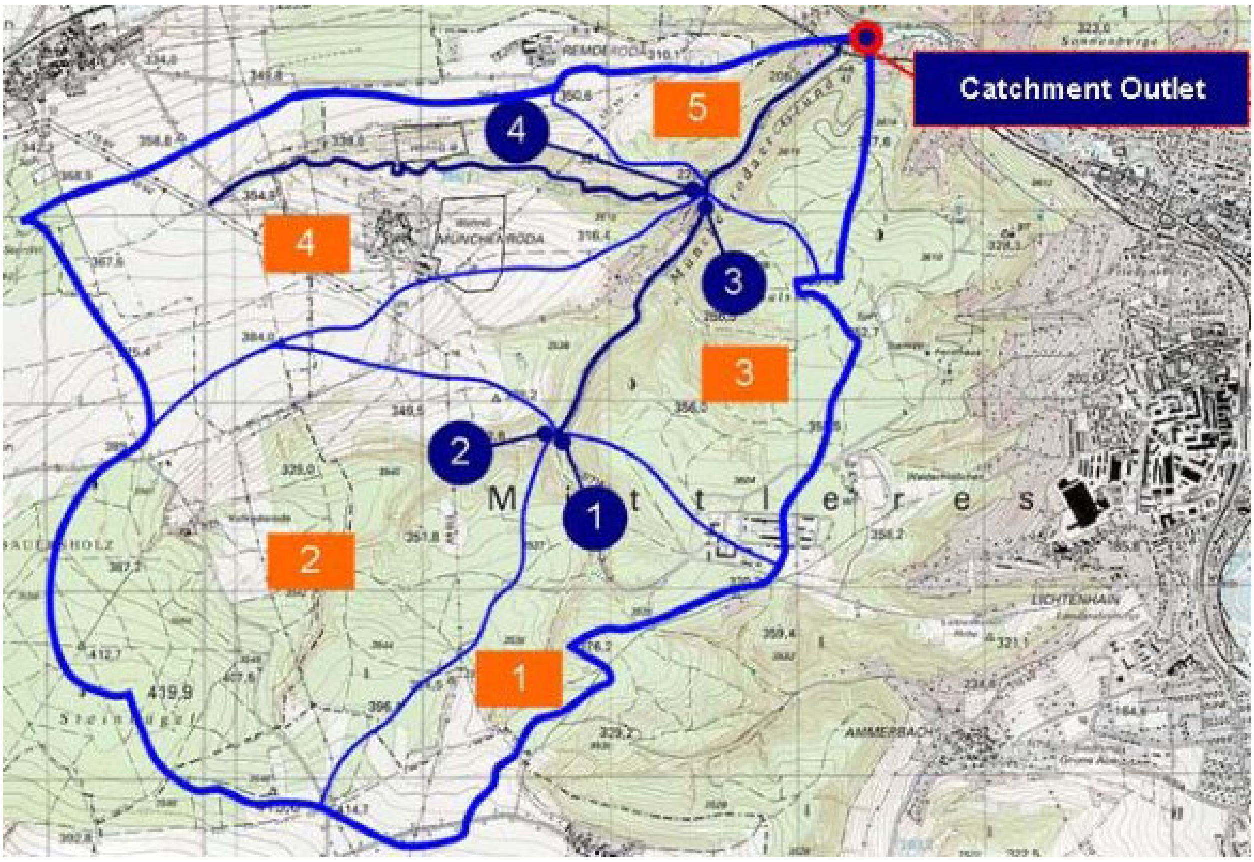
Tasks:

1. Realize a sub-basin-orientated division of the catchment, that was considered up to now integrated, so that you are able to model flood peak discharges for all points represented in figure Ü 24.1.

2. Build the model HQBEMESS for the 5 sub-catchments analogue to seminar 21! Please use the information given in the seminar 21! Document all essential parameter assumptions, input values and the necessary steps towards their destiny!

3. Model the 100-year heavy rain event sub-catchment orientated first in case of an unknown rainfall duration! You can find the model results in the folder "result", file name in accordance to the selected identification number. Enter the model results with respect to the peak flood discharges HQ and rainfall durations D in table Ü 24.1!

4. Identify the die design rainfall duration (rainfall duration, which leads in the whole catchment area to the largest flood peak) by means of sub-catchment area weighted rainfall durations modelled for the sub-catchments. Enter the area-weighted rainfall duration D in the table Ü 24.2! Model the 100-year heavy rain event sub-catchment orientated for the weighted dedign rainfall duration! You can find the model results in the folder "result". Enter the model results with respect to the peak flood discharges HQ in table Ü 24.2!



Calculation point 1: South-southeastern tributary valley to Münchenrodaer Grund
Calculation point 2: West-southwestern tributary valley to Münchenrodaer Grund from Vollradisroda
Calculation point 3: Münchenrodaer Grund before the confluence with Gollichsgraben
Calculation point 4: Gollichsgraben at the mouth to Münchenrodaer Grund
Calculation point 5: Catchment outlet (mouth of Münchenrodaer Grund to Leutra)

Fig. Ü 24.1: Sub-catchment orientated division of the study area

Table 24.1: Model results for the 5 sub-catchments (separately, without discharge wave superpositions) in case of unknown rainfall duration (return period: 100 years)

Sub-catchment	Flood peak discharge HQ [l/s]	Rainfall duration D [min]
1		
2		
3		
4		
5		
Area-weighted mean of design rainfall duration D →		

Table 24.2: Model results for the 5 sub-catchments (separately, without discharge wave superpositions) for the design rainfall duration (return period: 100 years)

Sub-catchment	Flood peak discharge HQ [l/s]	Rainfall duration D [min]
1		
2		
3		
4		
5		
Area-weighted mean of design rainfall duration D →		

5. Calculate the flow times between points 1/2 and 3 and between points 3/3 and 5. Assume a mean flow velocity of the flood wave of 2 m/s (this value has been calculated in advance by means of a hydraulic calculation formula, for example GMS formula). Round the flow times to whole minutes.

6. Realize a run time corrected superposition of the modelled hydrographs in accordance to task 4. You can use the model ISOCHRON (in folder „Programm“). Enter in table Ü 24.3 at first the type of wave superposition (synchron = without run time delays → isochrone delay = 0 min, isochrone = witht run time delays → isochrone delay > 0 min)! Consider that the HQBEMESS modelled hydrographs refer he deepest point in the sub-catchment (example: the hydrograph for the sub-catchment 1 refers to the calculation point 1). Superpose the single waves by means of programme ISOCHRON and enter the results in table Ü 24.3!

Table 24.3: Modelled flood peak discharges for the design rainfall duration considering wave run times (100 years return period)

Calculation point	Flow path length [km]	Isochrone delay [min]	Flood peak discharge HQ [l/s]
1			
2			
1 + 2			
3			
4			
3 + 4			
5 (Catchment outlet)			

6. Interpret the model results! Respond to the following aspects:
   - In this case, a significant difference in the flood peak discharges is determined in case of run time corrected superposition in comparison to not run time corrected superposition (additive superposition)? For which cases a marked difference is to be expected?
   - How much is the difference between the modeled flood peak values for the total area when comparing the model results for a sub-catchment orientated simulation with the results of a integrated simulation (result from seminar 21)? Interpret the results! Looking for reasons for the differences!

Hydrological seminar 25:

Flood analysis 11 – deterministic conception 10: Runoff modelling for different initial soil moisture conditions

Objective:

Quantification of the influence of soil moisture conditions at the beginning of the flood event (dry, moderate, wet) on flood peak discharge on example of a constant rain with a return period of 100 years using the conceptual rainfall-runoff model HQBEMESS for the unobserved catchment west of urban region of Jena (ditch Gollichsgraben including dry valley Münchenrodaer Grund up to the outlet to river Leutra)

Methodology:

→ see lecture notes “Hydrology III”, chapter 4.3.4.3

Given information:

- analogue seminar 24

Tasks:

1. Model the hydrographs and flood peak values of a 100-year storm event for the sub-catchments and for the total area for low or high initial soil moisture! Enter the model results in the tables Ü 25.1 and Ü 25.2! The model results for moderate soil moisture you can find in table Ü 24.3. Proceed methodically analogue to seminar 24:
    - identification of the design rainfall duration
    - modelling of the hydrographs for the design rainfall duration
    - wave superposition according run time effects (isochrone concept)

Table 25.1: Modelled flood peak discharges considering wave run times for dry conditions (low soil moisture, 100 years return period)

Calculation point	Flow path length [km]	Isochrone delay [min]	Flood peak discharge HQ [l/s]
1			
2			
1 + 2			
3			
4			
3 + 4			
5 (Catchment outlet)			

Table 25.2: Modelled flood peak discharges considering wave run times for wet conditions (high soil moisture, 100 years return period)

Calculation point	Flow path length [km]	Isochrone delay [min]	Flood peak discharge HQ [l/s]
1			
2			
1 + 2			
3			
4			
3 + 4			
5 (Catchment outlet)			

2. Interpret the model results according to design rainfall durations and flood peak discharges!

3. Compare the result uncertainty, results from initial soil moisture of the catchment with the uncertainties from rainfall distribution and rainfall tolerance ranges (see seminar 22)!

Hydrological seminar 26:

Flood analysis 12 – deterministic conception 11: quantitative assessment of the influence of land use changes in a catchment by means of rainfall runoff model

Objective:

Quantification of the influence of land use changes near village Münchenroda on flood peak discharge on example of a constant rain with a return period of 100 years using the conceptual rainfall-runoff model HQBEMESS for the unobserved catchment west of urban region of Jena (ditch Gollichsgraben including dry valley Münchenrodaer Grund up to the outlet to river Leutra)

Methodology:

→ see lecture notes "Hydrology III", chapter 4.3.4.3

Given information:

- analogue to seminar 24
- planning a business / industrial park outside the village Münchenroda → increase of sealed surfaces by 30 % to the account of grassland without the use of storage space (retention basins, etc.)

Tasks:

1. Model the hydrographs and flood peak values of a 100-year storm event for the sub-catchments and for the total area for the planning state! Proceed for a constant rainfall intensity and for moderate soil moisture. Enter the model results in table Ü 26.1! The model results for the initial state you can find in table Ü 24.3. Proceed methodically analogous to the seminars 24 and 25!

Table 26.1: Modelled flood peak discharges considering wave run times for the planned state (100 years return period)

Calculation point	Flow path length [km]	Isochrone delay [min]	Flood peak discharge HQ [l/s]
1			
2			
1 + 2			
3			
4			
3 + 4			
5 (Catchment outlet)			

2. Characterize the impact of the planned business / industrial park from the perspective of the rainfall duration, the flood rise time and the flood peak discharges! What are the effects for the sub-catchments and for the whole catchment (at catchment outlet)?

3. What activities do you propose to keep the flow increases due to land use change as small as possible?

Hydrological seminar 27: **Low water analysis**

Objectives:

- estimation of the deceeded low water discharges NQ(1m) with return periods of 5, 10 and 25 years for a summer month by statistical analysis of observed low-water events
- identification of reliability of water supply for the relevant month

Methodology:

→ see lecture notes "Hydrology III", chapters 5.2 and 5.3

Given information:

- deceeded low water discharges NQ(1m), time series 1951 – 1990 for July → see table Ü 27.1

Table Ü 27.1: Deceeded low water discharges NQ(1m) in m^3/s

July	NQ(1m) $[m^3/s]$	July	NQ(1m) $[m^3/s]$	July	NQ(1m) $[m^3/s]$	July	NQ(1m) $[m^3/s]$
1951	0.46	1961	0.10	1971	0.058	1981	0.60
1952	1.03	1962	0.077	1972	0.085	1982	0.055
1953	0.10	1963	0.25	1973	0.22	1983	0.095
1954	0.20	1964	1.25	1974	1.10	1984	0.67
1955	0.067	1965	0.74	1975	0.51	1985	0.26
1956	0.062	1966	0.080	1976	0.047	1986	0.40
1957	0.076	1967	0.10	1977	0.061	1987	0.81
1958	0.61	1968	0.12	1978	0.078	1988	0.075
1959	0.27	1969	0.11	1979	0.41	1989	0.058
1960	0.40	1970	0.90	1980	0.22	1990	0.13

Tasks:

1. Order the values (starting from the highest value) and enter the ordered values in table Ü 27.2!

2. Enter the low water flow rates NQ (1m) with the corresponding exceedance probabilities $P_{Ü}$ in the distribution function diagram (figure Ü 27.1) and determine the NQ (1 m) values with return periods of T = 5 a, 10 a und 25 a by free adjustment (curve of best fit)!

3. Indicate the NQ(1m) values for reliability degrees of water supply of 80, 90 and 98 % considering a minimum low water discharge from ecological point of view of 30 l/s!

4. Which reliability degree should be recognized when in case of drinking water supply? How many people could be supplied with drinking water in July (average water consumption is assumed)?

5. How would you proceed to determine NQ(1 m) values for a given reliability degree for each month of the year?

Table Ü 27.2: Exceedance probabilities of deceeded low water discharges NQ(1m), series 1951-1990

Order m [ ]	Low water discharge NQ(1m) [m^3/s]	Exeedance probability $P_Ü$ [%]
1		
2		
3		
4		
5		
6		
7		
8		
9		
10		
11		
12		
13		
14		
15		
16		
17		
18		
19		
20		
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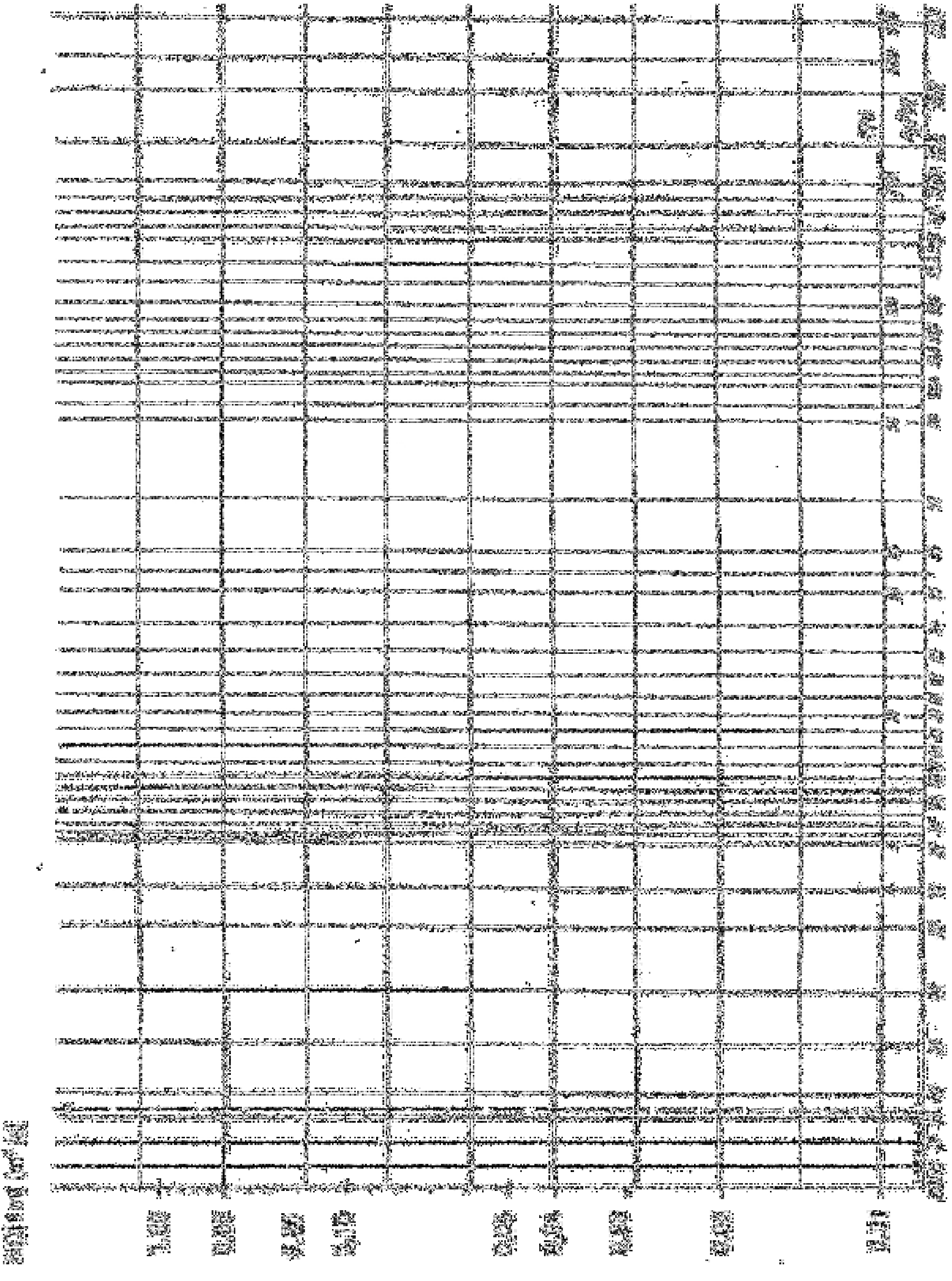

Fig. Ü 27.1: ***Probabilities of deceeded low water discharges*** *NQ(1m), series 1951 – 1990 and* ***adjusted probability distributions*** *in the EI distribution function diagram*

Hydrological seminar 28:

Hydrologic reservoir planning 1 – Application of empirical methods

Objective:

Application of empirical methods for hydrological design of a water retention basin near gauge Steinach (river: Steinach)

Methodology:

→ see lecture notes "Hydrology III", chapter 6.5.2

Given information:

- monthly average discharges for gauge Steinach, river Steinach in the period January 1998 to September 2009 → see table Ü 28.1
- catchment area: 37.2 km^2
- river-km: 43.2
- planning of a dam near gauge Steinach

Table 28.1: Monthly average discharges MQ [m^3/s]for gauge Steinach, river Steinach in the period January 1998 to September 2009

	J	F	M	A	M	J	J	A	S	O	N	D
1998	1.35	0.67	1.75	0.93	0.43	0.16	0.40	0.47	2.52	2.49	2.04	0.91
1999	1.17	0.86	2.43	0.99	0.32	0.27	0.17	0.15	0.17	0.59	0.79	2.47
2000	1.30	2.68	3.50	0.87	0.32	0.19	0.30	0.34	0.46	0.41	0.57	1.13
2001	1.56	1.56	2.26	1.08	0.60	0.45	0.46	0.26	0.62	0.67	1.30	1.44
2002	2.95	4.29	2.12	0.67	0.66	0.45	0.27	0.18	0.16	0.86	1.68	1.42
2003	2.32	0.55	0.97	0.54	0.46	0.20	1.05	0.11	0.12	0.25	0.32	1.18
2004	1.05	1.93	1.39	0.84	1.01	0.30	0.39	0.29	0.58	0.46	1.73	0.86
2005	1.98	1.87	2.03	1.19	0.72	0.36	0.26	0.38	0.39	0.38	0.26	0.96
2006	0.49	0.41	1.95	2.72	0.79	0.58	0.34	0.33	0.23	0.40	1.23	0.83
2007	2.88	1.86	1.95	0.42	0.33	0.49	0.98	0.63	0.52	0.58	0.99	3.21
2008	1.91	1.26	1.90	2.04	0.36	0.19	0.20	0.20	0.21	0.98	0.72	0.87
2009	0.44	0.39	1.86	2.60	0.42	0.34	0.29	0.21	0.26			

Tasks:

1. Calculate the inflow discharges (in hm^3) into the planned reservoir or the observation period 1998 – 2009! Develop for this an Excel spreadsheet!

2. Estimate by means of EXCEL total inflow volume, mean inflow volume and differential masses (each in hm^3) analogous to the example in the lecture notes "Hydrology III", chapter 6.5.2, table 6.2! Plot the differential mass curve!

3. Realize a storage volume design for the case that the regulated outflow out of the reservoir is equal to mean discharge MQ!

4. Determine graphically the storage volume which is necessary to realize a constant regulated outflow of 50 % of mean discharge!

5. Apply the method of the stretched thread and mark the periods in which for the storage volume determined in task 4 a regulated outflow > MQ, ≈ MQ and < MQ is necessary, to prevent a spillway!

Hydrological seminar 29:

Hydrologic reservoir planning 2 – Design of a rainwater retention basin by using simplified continuity equation

Objectives:

- design of a rainwater retention basin (RHB) with respect to the storage volume on basis of existing hydrographs (as a result of a rainfall-runoff model) in case of a constant outflow
- optimization of the storage volume of the RHB

Given information:

- landfill with two planned drainage sections to a RHB (→ see figure Ü 29.1):
  - → northern landfill drainage (output at the intersection of KP 4), drains the landfill over an surface drain in the RHB
  - → southern landfill drainage (output at the intersection of KP 5), drains the landfill and the surroundings (pinewood) directly into the RHB
- hydrographs for the two drainage sections for two design rainfall durations P_D → see tables Ü 29.1 and Ü 29.2:
  - → $P_D = 15$ min and
  - → $P_D = 30$ min (hydrographs contain both surface runoff and base flow)

Tasks:

1. determination of the total inflow hydrograph to the planned RHB:
   - indication of the relevant inflow peak to RHB
   - indication of the design rainfall duration (related to inflow peak)
   - indication of the rise time after the rain started
2. calculation of the required storage volume for the two rainfall durations:
   - indication of the required storage volumes for $P_D = 15$ and 30 min for the following two constant outflow discharges Q_R:
     - → $Q_R = 50$ l/s and $Q_R = 100$ l/s
   - indication of design rainfall duration (regarding storage design)
3. conclusions

Operations (proposal):

1. *superposition of the hydrographs respective isochrone concept (→ see lecture notes "Hydrology I", chapter 5.7):*
   - The hydrographs of intersection KP 5 (landfill and surrounding drainage) come together without any time delay (isochrone delay $t_{Iso} = 0$) and flow without any time delay to RHB
     - → direct summation of the discharge values for each time interval t
     - → recording the results in tables Ü 29.3 and Ü 29.4 (each column 2) for the two rainfall durations $P_D = 15$ and 30 min

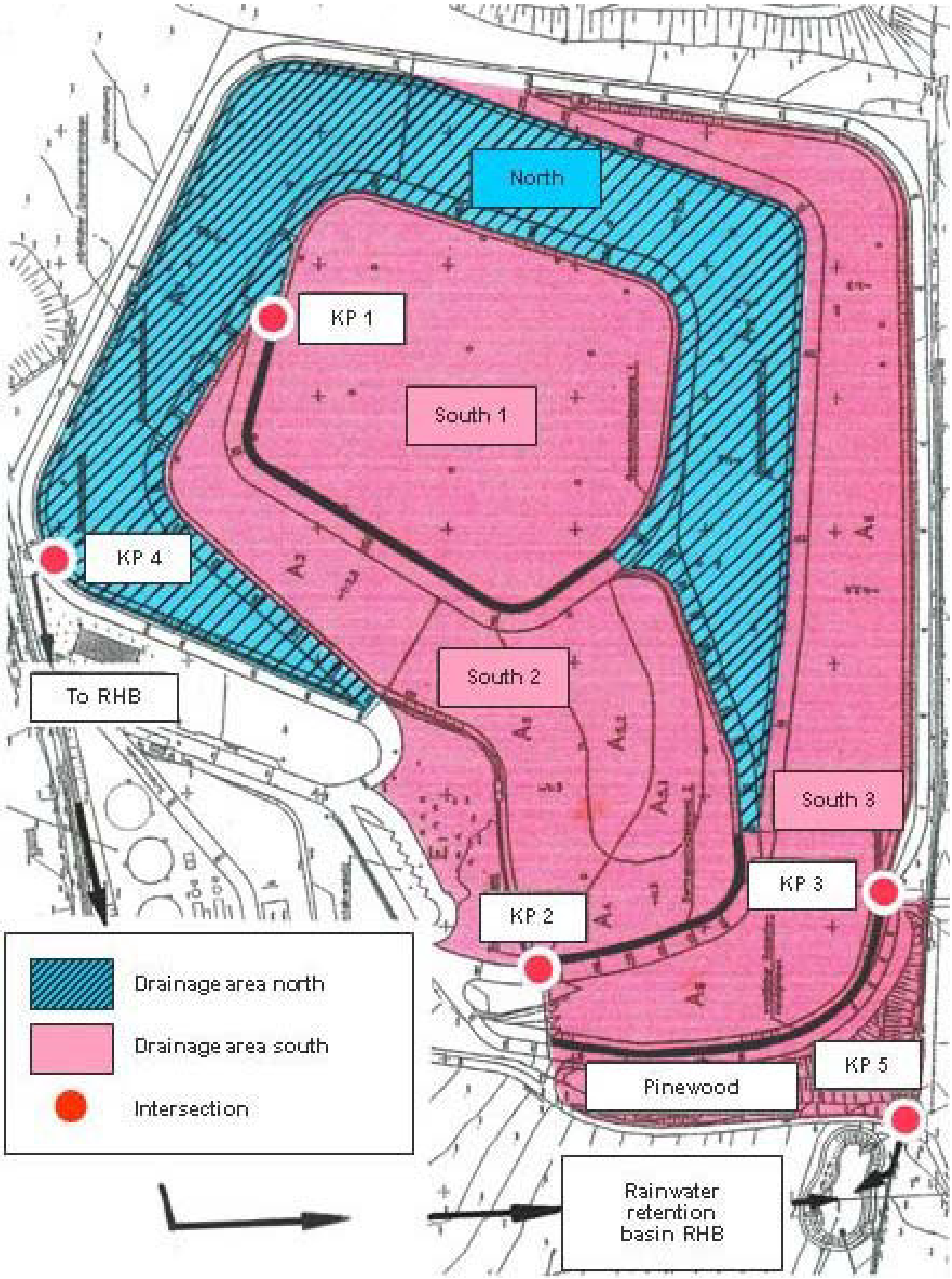

Fig. Ü 29.1: Planned surface water drainage concept fort he given landfill

Table Ü 29.1: Hydrographs for the intersections KP 4 (landfill drainage area north) and KP 5 (landfill drainage area south and surrounding) for design rainfall duration $P_D = 15$ min

Time inter-vall t [min]	Hydrograph KP 4 landfill (northern drainage area) Q [l/s]	Hydrograph KP 5 landfill (southern drainage area) Q [l/s]	Hydrograph KP 5 surroundings (southern drainage, pinewood) Q [l/s]
(Col. 1)	(Column 2)	(Column 3)	(Column 4)
1	2	3	1
2	5	12	3
3	10	28	5
4	17	53	8
5	26	87	11
6	36	128	13
7	48	173	16
8	62	219	18
9	77	265	20
10	93	308	22
11	108	345	22
12	122	373	22
13	133	387	20
14	140	386	18
15	144	372	16
16	145	348	13
17	143	315	11
18	137	279	9
19	129	240	7
20	118	203	6
21	106	169	5
22	93	138	4
23	80	112	3
24	67	90	3
25	55	72	2
26	45	57	2
27	37	46	1
28	30	37	1
29	24	30	1
30	20	25	1
31	16	21	1
32	13	17	1
33	11	15	1
34	10	13	1
35	8	12	1
36	7	11	1
37	7	11	1
38	6	10	1
39	6	10	1
40	5	9	1

Table Ü 29.2: Hydrographs for the intersections KP 4 (landfill drainage area north) and KP 5 (landfill drainage area south and surrounding) for design rainfall duration $P_D = 30$ min

Time inter-vall t [min]	Hydrograph KP 4 landfill (northern drainage area) Q [l/s]	Hydrograph KP 5 landfill (southern drainage area) Q [l/s]	Hydrograph KP 5 surroundings (southern drainage, pinewood) Q [l/s]
(Col. 1)	(Column 2)	(Column 3)	(Column 4)
2	1	4	1
4	3	9	1
6	5	16	2
8	8	26	3
10	13	40	4
12	19	58	5
14	26	78	6
16	35	101	7
18	44	124	8
20	53	146	10
22	63	168	11
24	72	189	12
26	80	208	12
28	88	225	13
30	96	241	14
32	102	254	14
34	107	265	15
36	111	270	14
38	113	269	14
40	113	263	13
42	111	252	12
44	106	237	11
46	100	219	10
48	93	200	9
50	86	181	8
52	78	162	7
54	70	143	7
56	62	126	6
58	55	110	5
60	48	96	4
62	42	84	4
64	37	72	3
66	32	62	3
68	28	54	3
70	24	47	2
82	21	40	2
74	18	35	2
76	16	31	2
78	14	27	1
80	13	24	1

- The superposition of the hydrographs at intersections KP 4 and KP 5 (=total inflow to RHB) must take into account the runtime of the hydrograph at intersection KP 4 to RHB.
- The runtime calculation is possible by application of GMS formula (→ see chapter 1.3.2). For this, use the following information for the design of the channel from intersection KP 4 to RHB (from planning documents):
  - → open drainage channel of dry-stone walls, coarse hewn (natural stone)
  - → rectangular cross-section: 1.0 m wide
  - → accepted design water level: 0.3 m above the bottom
  - → geodetic height of the channel (bottom) at the point KP 4: 475.20 m above sea level
  - → geodetic height of the channel (bottom) at the RHB-entry point: 467.80 m above sea level
  - → channel length: $l_{Channel} = 513$ m
- result of the flow velocity calculation: v = ______ m/s
- interposed question: Are the peak discharges Q_S (from tables Ü 29.1 and Ü 29.2) discharged by the planned channel? → drainage capacity → comparison of $Q_{Plan} = v * A$ with Q_S:

  result: Q_{Plan} = ______ m^3/s = ______ l/s

  Q_S für P_D = 15 min: Q_S = ______ l/s

  Q_S für P_D = 30 min: Q_S = ______ l/s

  drainage capacity O.K. / not O.K.: ______.
- runtime calculation (min):

$$t_{Iso} = (l_{Channel}/v) / 60\,s = \underline{\qquad\quad \text{min}} \qquad \textit{(Ü 29.1)}$$

t_{Iso} - discharge time [min]
l_{Kanal} - channel length [m]
v - flow velocity [m/s]

- calculation of the isochrone delay [min] by rounding the result to equation 1.28 for whole minutes:

  result: isochrone delay: ______ min
- temporal postponement of the hydrograph at intersection KP 4 about isochrone delay → Enter the values in the tables Ü 29.3 and Ü 29.4 (each in column 3)
- superposition of the hydrographs at intersections KP 4 and KP 5 (inflows to RHB) considering isochrone delay → enter the values (simple adding up the values from columns 2 and 3) in tables Ü 29.3 and Ü 29.4 (each column 4)

2. *RHB storage volume calculation for a given constant reservoir output control:*

- RHB design for a given constant reservoir outflow calculable through the application of the continuity equation (→ see chapter 6.5.4, equations 6.3 and 6.4 → requirement: inflow discharge $Q_Z > $ outflow discharge Q_A, where the outflow discharge Q_A is equal to output control Q_R)
- Use from the moment, where $Q_Z > Q_R$ for 2 consecutive time steps, the continuity equation in difference form (approximate solution → see equation 6.4)! The calculation ends with the moment, where $Q_Z < Q_R$.
- Enter the values in the tables Ü 29.3 and Ü 29.4 (each in columns 5 and 6)!

- The necessary storage volume is determined by the maximum value of column 5 (for 50 l / s output control) or 6 (for 100 l / s output control).

Table Ü 29.3: Determination of the total inflow hydrograph (superposed hydrographs by means of isochrone concept) to RHB for the design rainfall duration of P_D = 15 min

Time interval t [min]	Hydrograph at KP 5 Q [l/s] (landfill + surroundings)	Runtime corrected hydrograph KP 4 Q [l/s]	Total hydrograph to RHB Q_Z [l/s]	Storage volume RHB (Q_A: 50 l/s) S_{min} [m^3]	Storage volume RHB (Q_A: 100 l/s) S_{min} [m^3]
(Column 1)	(Column 2)	(Column 3)	(Column 4)	(Column 5)	(Column 6)
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Table Ü 29.4: Determination of the total inflow hydrograph (superposed hydrographs by means of isochrone concept) to RHB for the design rainfall duration of P_D = 30 min

Time interval t [min]	Hydrograph at KP 5 Q [l/s] (landfill + surroundings)	Runtime corrected hydrograph KP 4 Q [l/s]	Total hydrograph to RHB Q_Z [l/s]	Storage volume RHB (Q_A: 50 l/s) S_{min} [m^3]	Storage volume RHB (Q_A: 100 l/s) S_{min} [m^3]
(Column 1)	(Column 2)	(Column 3)	(Column 4)	(Column 5)	(Column 6)
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Hydrological seminar 30:

Hydrologic reservoir planning 3 – Design of a rainwater retention basin by means of rainfall runoff model

Objective:

Design of a rainwater retention basin near the village Münchenroda for a return period of 100 years by means of submodel SPEICHER (part of the rainfall-runoff model HQBEMESS) for ditch Gollichsgraben up to the outlet to dry valley Münchenrodaer Grund west of Jena

Methodology:

→ see lecture notes "Hydrology III", chapters 4.3.4.3 and 6.5.4

Given information:

- hydrograph of a 100-year flood event (constant rainfall intensity and moderate soil moisture in the catchment) for ditch Gollichsgraben (sub-catchment 4) in the planning state (after construction of the business / industrial park near the village Münchenroda) from seminar 26
- analogue: hydrograph for ditch Gollichsgraben for the present state from seminar 24

Tasks:

1. The discharge increase due to the business / industrial park near the village Münchenroda should be compensated by a retention basin. Calculate for the 100-year storm event the necessary storage volume of the basin for the case that the reservoirs output control is equal to the flood peak discharge, what has been modeled for the present state! Use for this the sub-model SPEICHER (in folder „Programm"). Is there any change of the flood peak discharge at catchment outlet (outlet to river Leutra)?

2. To reduce the peak discharge at the mouth of river Leutra, the rainwater retention basin at the mouth of ditch Gollichsgraben should be designed for a 100-year storm event for a maximum reservoir output of 1.5 m^3/s. How big the storage capacity of the reservoir has to be in this case? To which value the flood peak discharge at the catchment outlet to river Leutra is reducible by means of the retention basin? Is a retention basin of the calculated volume at the mouth of ditch Gollichsgraben realizable under the given geometric conditions?

Appendix 2

Instruction for project work:

Hydrologic evaluation of creek in a small catchment area regarding the situation in case of heavy rain

1. Introductory remarks

Object of study is a specially chosen catchment expediently (but not necessarily) near Freiberg which is not too large (maximal 1 km^2). Consider during the process of catchment selection that the creek should be big enough for discharge measurements in low water periods. In an independent group working parts of the catchment area have to be analysed by hydrological and hydraulic methods.

In the context of project work are carried out statements about the behavior of the study area in case of heavy rain events.

The subtasks are the following:

- mapping of the course of stream
- measuring of the longitudinal and cross sections
- measurement of water levels , the mean flow velocities and discharges at selected points in the longitudinal section of the stream
- creation of water level – discharge relationships for selected points of the stream
- calculation of flood discharges for different return periods
- Indication of maximum water levels for different return periods

To solve theses tasks, the conceptual rainfall-runoff model HQBEMESS and the DWD German Weather Survey heavy rainfall programme KOSTAB (part of KOSTRA atlas) are provided.

PLEASE NOTE IN COHERENCE WITH THE USAGE OF THE PROVIDED SOFTWARE:

THE USAGE OF ALL COMPUTER PROGRAMMES IS ALLOWED IN THE CONTEXT OF THE PROJECT WORK! IT IS NOT ALLOWED TO USE THE SOFTWARE FOR OTHER OBJECTIVES OR TO GIVE THE SOFTWARE TO OTHER PERSONS OR OTHER COMPANIES! THEREOF EXCLUDED ARE ALL FILES (INPUT FILES, OUTPUT FILES, TEXT FILES) YOU HAVE CREATED YOURSELFE. IN THE INTERESET OF A RAPID PROGRESS OF THE PROJECT WORK YOU CAN TRANSFER THE PROGRAMMES TO YOUR PRIVATE COMPUTER. AT THE END OF THE WORK YOU ARE COMMITTED TO DEINSTALL ALL COMPUTER PROGRAMMMES COMPLETELY!

You edit the sub-tasks within the team (usually about groups of four) and produce a report of **maximum 30 pages text**. The report must be submitted as a pdf or Word file. All input and output data have to be copied to an external data storage devise and have to attached to the report.

2. Working documents

Basis of project work are maps of the study area with information to morphology, land use, soil types and the course of the stream. You have to acquire the maps for yourself.

All data and parameters that you need to solve the tasks, you have to acquire independently. Heavy rain heights are determinable on the basis of the DWD KOSTRA Atlas. Flood peak discharges are computable by means of a rainfall-runoff model (for example programme HQBEMESS).

Each group organises independently all things which are necessary for the work (maps, measuring instruments, …).

3. Subtasks

- mapping the course of stream:
  - → mapping includes all the tributaries to the investigated creek
  - → comparison of the mapping results with existing maps
  - → mapping of critical points (stream narrows, bridges, sharp bends, ...)
- measurement of the mean flow velocities and discharges at the 5 in your opinion selected critical points (aim: if it allows the meteorological situation → measurements at least 3 hydrologically different days):
  - → cartographic documentation of all measurement points
  - → selection and explanation of discharge measurement method (if possible according to the discharge conditions → application of two different measuring methods)
  - → realization and documentation of the discharge measurements
  - → analysis and assessment (including the estimation of measurement errors and value uncertainties resulting from measurement errors)
- water level measurements in the exact places and exact times of the flow velocity and discharge measurements:
  - → definition of tide gauge zero if possible close to the river bottom
  - → provisorily fixing of level-zero, so that it will remain until the next measurement campaign
  - → measurement of water levels
  - → create a water level / discharge relationship for the 5 measuring points
  - → calibration of STRICKLER coefficients using the flow velocity and discharge measurements for the 5 measuring points
- create a photo documentation with an explanation for the areas of measurement points:
  - → documentation of the investigated points
  - → characterization of the hydraulic roughnesses
  - → characterization of vegetation

  - → deduction of STRICKLER coefficients from purely visual point of view (creek structure, vegetation)

- recording / measurement of the stream cross-sections of the 5 selected points:
  - → above the level where flooding starts → height in dependence on morphological conditions → up to the water assumed in case of 100-year flood event
  - → assessment of the hydraulic roughnesses of the 5 selected points
  - → estimation of d river gradients in the longitudinal section (own measurements in the area of the measurement points and comparison with maps)
  - → calculation of river cross-sectional areas for the situation where flooding starts

- calculation of the flow velocities for the selected points for the situation where flooding starts by means of GMS formula

- calculation of the maximum drainable discharge for the situation where flooding starts on base of the mean flow velocities and cross-sectional areas at the measurement points

- determination of flood peak discharges for return periods of 2 years (normal flood event) and 100 years (extreme flood event)
  - → determination of surface catchment boundaries and estimation of the catchment areas down to each of the 5 selected points
  - → development of catchment-specific parameters (morphometric parameters, use, soil types) on the basis of maps (topographic, land use and soil maps), validation of the soil information estimated on base of soil maps by means of infiltrometer measurements at at least 3 points in the catchment area
  - → development of KOSTRA heavy rainfall values for the 2 return periods using the DWD program KOSTAB
  - → quantification of the hydrological processes runoff formation and runoff concentration by means of a suitable rainfall-runoff model (for example model HQBEMESS) for the 2 return periods
  - → determination of the design rainfall duration (= rainfall duration, what produces a maximum discharge for otherwise identical conditions)
  - → superposition of the hydrographs, that have been estimated for the design rainfall duration by isochrone concept → flow velocity calculation for the sections between the 5 measurement points by application of GMS formula

- approximate determination of flood peak water levels for the 2 return periods (2 and 100 years)

- comparison of the maximum drainable discharges estimated by the hydraulic investigations with the calculated flood peak discharges modelled by means of a rainfall-runoff model and conclusions regarding the hydraulic drainage capacities of the 5 measuring points

Summarize the main results of the calculations (table or map with the maximum flow velocities, flood peak discharges and cross-sections for each drainage segment)! Discuss measures to defuse the most critical sections!